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FACULTÉ DES ÉTUDES SUPÉRIEURES  
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FACULTY OF GRADUATE AND  
POSTDOCTORAL STUDIES

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AUTEUR DE LA THÈSE / AUTHOR OF THESIS

Ph.D. (Civil Engineering)

GRADE / DEGREE

Department of Civil Engineering

FACULTÉ, ÉCOLE, DÉPARTEMENT / FACULTY, SCHOOL, DEPARTMENT

The Performance of Hybrid Long-span Cable Stayed Bridges Using Advanced Composites

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# **THE PERFORMANCE OF HYBRID LONG-SPAN CABLE STAYED BRIDGES USING ADVANCED COMPOSITES**

By

**Husham H Almansour**

B.Sc.(Honors), M.A.Sc.

A Thesis Submitted to  
The Faculty of Graduate and Postdoctoral Studies  
in partial fulfillment of  
the requirement for the degree of

**Doctor of Philosophy**

In Civil Engineering\*

The University of Ottawa  
Department of Civil Engineering  
Ottawa, Ontario, Canada

July 2006

\*The Ph.D. in Civil Engineering is a joint program with the  
Carleton University, administrated by the Ottawa-Carleton Institute for Civil Engineering

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395 Wellington Street  
Ottawa ON K1A 0N4  
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395, rue Wellington  
Ottawa ON K1A 0N4  
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*Your file    Votre référence*

*ISBN: 978-0-494-25851-4*

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*ISBN: 978-0-494-25851-4*

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## **ACKNOWLEDGEMENTS**

I wish to express my sincere gratitude and appreciation to my supervisor, Professor Moe Cheung for his immense support, guidance, valuable suggestions and encouragement.

My admiration and thanks also go to the Department of Civil Engineering of the University of Ottawa, and the Department Chair, Professor R. Narbitz, for continuous and kind support.

Great appreciation and thanks to Professor H. Tanaka for his valuable advices, encouragement and financial support, Professor M. Saatcioglu for his valuable discussions and unremitting support, and Dr. M. Mohareb for generosity in sharing his ABAQUS license.

I wish also to extend my appreciation and gratefulness to my wife, Radiah for her continuous and generous support, assistance and patience. My thanks are also due to my daughters Sura, Rula and Sana, for their love that has always given me strength during hard times.

## ABSTRACT

The main challenges in designing, building and maintaining cable-stayed bridges still remain the increase in the bridge central span, life cycle effectiveness and aerodynamic stability. The development of the cable-stayed bridge central span has almost reached the technical limit for traditional materials, construction technology and bridge system capability. An innovative solution has been proposed in this research, which introduces a new hybrid long span cable stayed bridge system utilizing advanced composite materials for the deck and the stay cables. The proposed hybrid cable-stayed bridge can be constructed for even longer spans, as the deck and cable stiffness and strength to weight ratios are significantly improved. This diminishes the critical compressive stresses of the traditional deck in the pylon zones, and enables the use of longer cables.

For such type of very complicated and expensive structure, a simplified ADP would not be adequate to optimize the structural parameters as well as the material parameters in the micro and macro levels. To overcome the complications resulting from the need to integrate the advanced composite material micro/macro design, and the resulting mechanical properties, to the structural modeling, an analysis and design procedure is proposed. The procedure is iterative and employs a Multi-Scale Modeling Technique. It assumes that the FRP micro and macro parameters and architecture, as well as the structural parameters of the hybrid bridge system, are variables. The final values of all the parameters at all levels, micro, macro and structure, are decided during the procedure. Satisfying serviceability and ultimate limit states and the aerodynamic stability bound the structural behavior of the bridge. The proposed Analysis and Design Procedure appears to represent an appropriate, consistent and systematic process for analyzing and designing the new Hybrid Long-Span Cable Stayed Bridge.

The Stochastic Finite Element method, using a two-phase stochastic process, is introduced to evaluate the mechanical properties of the chopped FRP material, with single or multi-fiber types. As the numerical efforts in the proposed method and the resulting accuracy are acceptable with the use of a reasonably fine meso-mesh, the method appears to be a capable tool and is employed as an integrate part of the analysis and design procedure. On the other hand, the “Laminas Dominant Fiber Alignment” principle, LDFA, is introduced in this

research. Consequently a simplified rule of laminate lay-up using the LDFA principle is proposed. It arranges the lamina orientations so that they are symmetrical on the middle surface of the laminate, which leads to an accurate simulation of the interlamina shear, using first order transverse shear flexible theory. It also separates the lamina orientations of the inner from the outer laminate by using low to medium orientation angles for the inner laminate, and medium to high orientation angles for the outer laminate. The laminate architecture based on the proposed lay-up rule gives better static and dynamic results than simple orientation arrangements for bridge decks with laminated box girders.

Four section models, together with CFRP cable stays, have been introduced to form the proposed Hybrid Long Span Cable-Stayed Bridge. The hybrid systems are scaled to match the bridge configurations and geometric characteristics of three of the world's longest span cable stayed bridges, of central spans 465 m, 628 m and 890 m, namely the Annacis Island Bridge, the Second Chang River Bridge and the Tatara Bridge respectively.

In order to adjust the deflection of the bridge under the self and the superimposed dead loads, according to the zero-deflection procedure, a simple empirical formula to calculate the required cable tension has been introduced. Using the proposed pre-tension formula, it has been found that almost zero deflection along the bridge longitudinal centerline can be achieved.

Comparing the results of the four models, it has been found that reducing the height of the deck at the section sides would increase the Tsai-Hill Failure Function in the vertical laminates, increasing the deflection at the central span, and decrease the critical flutter velocity. The use of the tubular inner cells increases the level of the longitudinal displacement of the deck, however it greatly improves the natural frequencies of the bridge system. The deck section that is arranged with most of the deck chopped FRP materials located in the two side "boxes", far from deck center of gravity, has very high torsional natural frequencies; hence, it has a high critical flutter velocity. It has also been found that increasing the CFRP cable radius and increasing the laminate thickness have significant effects on the deformations, stresses and natural frequencies. However, increasing LDFA would affect the hybrid bridge system slightly for most of the cases, and for some cases, it can significantly affect the stresses in some laminates. The increase of the pylon stiffness

improves the torsional to the vertical flexural ratio, but it also changes the stresses and deformations distributions.

With an appropriate choice of the micro/macro and structural parameters, the proposed hybrid bridge system gives quite acceptable maximum deflections, while the lateral and the longitudinal displacements are very small in general. For most of the cases, the Tsai-Hill Failure Functions have values much lower than the criterion limit. The stresses in the stay cables and the chopped FRP parts are far lower than the strength of their materials. The natural frequencies for most of the cases are within the known ranges of this type of structure for the discussed central span lengths. Considering the static and the aerodynamic results, it is apparent that Models III and IV are efficient and stable for extremely long spans (with central span lengths around 900m). On the other hand, Model I can bridge a very long central span (with a central span length around 600m), while Model II is only able to bridge shorter central spans of less than 400 m. The Critical Flutter Velocities of the proposed hybrid bridge is lower than those of the reference bridges in general. However, by improving the dynamic damping characteristics of the four models and evaluating their structural and aerodynamic damping ratios accurately, it is possible to improve the critical flutter velocities and extend the central span length limit of all the four models.

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# **Chapter 1**

## **Introduction**

# Chapter 1

## Introduction

### 1.1 General:

Bridging very long spans has always been a great challenge for engineers throughout history. The pre-eminence of cable supported static systems, in addition to the use of higher strength materials, enables engineers to move ahead to the limits of what is technically possible. Cable-stayed types of bridges are becoming more and more popular in the construction of long span bridges due to their many advantages, i.e. light in weight, efficient in load resistance, and of smaller cross sections. The cable-stayed bridge can be constructed for even longer spans, if the deck and cable stiffness and strength to weight ratios can be improved. This could significantly diminish the critical compressive stresses of the deck in the pylon zones, and increase the apparent stiffness of the stay-cables, as their sag action is reduced due to a huge drop of the weight per unit length. Nevertheless, the major target in the design, building and maintenance of longer span cable-stayed bridges remains the reduction of the deck stresses, life cycle effectiveness and the aerodynamic stability.

FRP materials have superior structural characteristics, such as high strength and stiffness to weight ratios, high resistance to corrosion and favorable fatigue characteristics. These properties, together with recently reduced material cost-to-weight ratio, encourage the use of FRPs in long span cable-stayed bridges. Consequently, this research proposes a new hybrid long span cable-stayed bridge, which utilizes advanced composite materials for the deck and the stay-cables. The new cable-stayed bridge system is scaled to match the general geometrical shapes of three of the world longest cable-stayed bridges, the Annacis Island bridge of Vancouver (central span of 465 m), the Second Chang River Bridge, China (central span of 628 m), and the Tatara Bridge, Japan (central span of 890 m). Specifically, the use of uni-axial, laminated and chopped Fiber Reinforced Polymer (FRP) was investigated as major materials for the new cable-stayed bridge system.

Advanced composites have complicated anisotropic mechanical properties, which result from the material micro/macro architecture. While the only coupling of an isotropic material is the extension-extension coupling, the stresses and strains of an anisotropic material are all coupled, and the failure states cannot be related to a single stress or strain. The deformation response of an anisotropic material, even to simple stress states, can literally be in every direction and in every plane. Consequently, with the minimum use of the material, it is very challenging to develop proper advanced composite structural sections. Structures formed from such sections should be capable of holding the static and dynamic loads and give acceptable static deformations and vibrations in the desired planes, while in other planes the deformations and vibration amplitudes are negligible.

A long span cable-stayed bridge system is a very sensitive nonlinear system. Balancing the loads and stiffness contributions of different components in the bridge system, and the cable-stay configuration, pretension and sagging play important roles in the stress levels and distributions in the cable stayed bridge deck and stay-cables. On the other hand, as the central span of the proposed bridge system is very long, aerodynamic stability is of major concern. The dynamic characteristics of the bridge system, such as the natural frequencies and the system damping, have important effects on the bridge response to the aerodynamic excitation.

Hence, using advanced composite materials for the deck and stay-cables would impact the sensitive relations of the bridge system unless a proper micro/macro design is

introduced. The traditional cable stayed bridge system is designed using a well-defined list of typical materials. In contrast, the advanced composite material has to be designed in the micro/macro scale to give the desired characteristic to the bridge structure. The advanced material architecture parameters should then be integrated to the bridge system structural parameters to give a single iterative process. The aim of such a process is to get an optimum static and dynamic response of the proposed hybrid bridge. Consequently, the present research suggests an analysis-design procedure involving Multi-Scale Modeling Techniques and following the Limit-States Design philosophy. The procedure starts with an assumed micro/macro design for the advanced composites, and accordingly the mechanical properties of the material are evaluated. The results represent an input for the structure level modeling stage. Then the structure level parameters are evaluated by the limit states and the structural stability bounds. According to the results, further modifications (or major changes) to the micro/ macro level parameters are expected. Re-evaluation of the material properties starts a second cycle of the analysis and design procedure. The other steps are repeated successively and the procedure cycles are repeated till an optimum bridge system behavior is achieved.

In the present research, the focus is on the performance of the proposed bridge system. Therefore, the influence of the major micro/macro and structure parameters on the bridge response is investigated. Consequently, to view the effect of using advanced composite material on the new cable-stayed system performance, three sets of parameters are examined. The *first set* includes the micro level parameters such as fibers and matrix properties, fiber fraction, and randomness or periodicity of the microstructure of the advanced composites. While the *second set* includes the macro-level parameters such as the lamina alignments and thicknesses, lamina sequence in the laminate or the laminate lay-up, and the laminate thickness. The *third set* is the structure level parameters, which includes the bridge central span length, the CFRP radius and the pylons stiffness. Many other structure-level parameters are not discussed in this research, such as the pylon height, the stay system arrangements and the boundary conditions, as those of the reference bridges are used for comparison purposes.

## 1.2 Objective:

The development of long central spans for cable-stayed bridges has almost achieved the technical limit for traditional materials, construction technology and bridge system capability. Further increase of the central span and decrease of the maintenance cost requires introducing an innovative and comprehensive solution to the main conflicts in the design and construction of such very long span cable stayed bridges. Consequently, the main objectives of the proposed research are:

- To develop a new hybrid long span cable stayed bridge system using advanced composite materials for the deck and the cables, which are capable to bridge extremely long spans, and are cost effective, durable and aerodynamically stable.
- The use of three dimensional nonlinear modeling techniques is compulsory to overcome (i) the three dimensional complicated response results from the advanced composite material anisotropy, where the stresses and strains are coupled, and (ii) the effect of the deformations on the system stiffness of the proposed huge long span bridge, where accounting for the geometric non-linearity is essential. On the other hand, the appropriate advanced composite material micro/macro architecture for this type of bridge structural behavior is not known. Hence, it is an absolute necessity to integrate the material architecture design-evaluation stage to the structural modeling-performance stage. The structural behavior of the system has also to be bounded by ultimate and serviceability limit states and aerodynamic stability. Therefore, a major aim of the research is to develop an Analysis-Design Procedure (ADP) using a Multi-Scale Modeling Technique that interactively involves the material and the structural parameters as variables. For such type of very complicated and expensive structure, a simplified ADP would not be adequate to optimize the structural parameters as well as the material parameters in the micro and macro levels.
- Using the structural and the aerodynamic achievements regarding the most successful deck sections and stay system arrangements, the most economical available advanced composite material and the proper automated manufacturing process, innovative deck section models have to be proposed. These deck section models, together with the cable stay system arrangement with the new material, have to be geometrically and structurally equivalent to some of the most successful very long span cable stayed

bridges around the world. The research objective therefore comprises investigating the structural characteristics of the new bridge system through a detailed performance study. This includes a comprehensive study of the micro-macro as well as the structure level, parameters influence on the structure static and dynamic outputs.

### 1.3 Scope

The proposed research will be fulfilled, with the following scope:

- A selection of three of the major very long span cable stayed bridges around the world as reference bridges. For comparison purposes, the general configuration and geometry of the three bridges are to be as close as possible, but their central span lengths are to be range between 450m and 900m (the current maximum constructed very long span cable stayed bridge).
- Introduction of four different deck-section models using laminated and chopped FRP materials: (i) From the static consideration, a shallow box girder with thin sides is proposed (Model II), (ii) to increase the deck stiffness and to match the reference bridges' deck sections, a uniform depth box girder with aerodynamically smoothed sides is introduced (Model I), (iii) to reduce the possible stress concentrations at the corners and keeping the boundary shape of Model I, a cylindrical tubular laminated box girder is introduced (Model III), and (iv) to increase the torsional to the vertical natural frequencies and to reduce the use of the laminated FRP, a section with two “boxes” and intermediate “slab-grid” part formed from chopped FRP in addition to two tubular girders and stiffening laminate is introduced.
- Consequently, the four proposed deck sections are rescaled to approach the general boundary dimensions of the reference bridges deck sections. On the other hand, the CFRP cables are introduced as the material for all the cables, keeping the stays configurations the same as in the reference bridges.
- Development of an interactive analysis and design procedure, which integrates the material micro/macro design and properties' evaluation to the structural modeling. The material and the structural parameters of the hybrid system are to be optimized through matching the ultimate and serviceability limit states and the aerodynamic stability. Each



stage of the procedure is to be developed using available evaluation or modeling methods or introducing new methods if they do not exist or the existing methods are inappropriate.

- Introducing a simplified lay-up rule to obtain the optimum laminate macro design which is appropriate for the proposed hybrid bridge.
- Development of the Multi-Scale Modeling Technique (MSMT) in two phases: (a) Micro-Macro level phase that evaluates the advanced composite properties through homogenization processes. Two micro-mechanical approaches are involved here; (i) Stochastic Finite Element method using two-phase stochastic process for the composites of random nature, which is developed in this research, and (ii) Luciano analytical approach to evaluate the advanced material properties with periodic nature in the micro-scale. (b) Structure Level Modeling Phase which is three dimensional, and simulates the Hybrid Cable-Stayed Bridge using the Nonlinear Deterministic Finite Element Method.
- Introducing a simple method to adjust the deflection of the hybrid bridge deck under self and superimposed dead loads through determining the pretension force.
- Properly define ultimate and serviceability limit states according to the composite materials failure criteria and the bridge deformations and vibrations service limits.
- Examine the aerodynamic instability of the bridge through evaluating the critical flutter velocities, while other types of instability are considered less critical. Because of the nonlinear nature of the proposed hybrid bridge system, the aerodynamic stability is to be investigated through the time domain flutter analysis technique.
- As the proposed hybrid bridge system is entirely new, the major efforts of this research are directed in exploring its structural behavior, capability and limitations. A thorough performance study to investigate the influence of three sets of parameters; micro, macro and structural levels parameters, on the bridge static and dynamic characteristics, is to be conducted.

## **Chapter 2**

### **Literature Review**

# **Chapter 2**

## **Literature Review**

### **2.1 Introduction**

The proposed research involves investigation of the potential of a new generation of long span cable-stayed bridges that utilize a hybrid design of laminated with chopped advanced composites for the deck, CFRP cables and concrete pylons. The research suggests a consistent Multi-Level Modeling procedure and proceeds to examine the new bridge structural properties. This involves the effect of micro/macro design of advanced composite materials on the overall bridge structural static and dynamic characteristics, in addition to its stability. Consequently, it proposes a compact analysis-design procedure for the new hybrid long-span cable-stayed bridge, in parallel with the state of the art analyses in long-span cable stayed bridges. Hence, the literature review aims to explore the background of this work through the published research in the last three decades, which are employed as bases of the three roots of this study; (i) the state of the art in the field of long span cable stayed bridges design and structural modeling techniques. (ii) advanced composite materials and present techniques in modeling their properties, and their present use in the bridge structures, and (iii) aerodynamic forces, modeling and long span bridge stability.

## **2.2 Long-Span Cable Stayed Bridges:**

### **2.2.1 Cable-stayed Bridge History, Development of their Design, Layout and Structural Characteristics:**

Bridging very long spans is the domain of cable-supported structures. The superiority of their static systems, as well as the use of high strength tensile elements, enables them to advance to the borderline of technical feasibility. For a given system, this borderline for maximum span is determined by the strength, stiffness, and density of the utilized materials [Starossek 1996]. The following is a brief review of the development of cable-stayed systems, bridge layout and structural characteristics.

Cable-stayed bridges have been around for the last couple of centuries but have become more prevalent in the last fifty years. Even before 1840 (Hartley System) there was evidence that primitive bridges had decks stayed from above by ropes or vines. Modern cable-stayed bridges have been experiencing a revival since the mid-1950s, although the concept of supporting a bridge girder by inclined tension stays can be traced back to the seventh century [Podolny et al, 1972]. In the same way that engineers invented the concept of cable-stayed bridges, they also began the development of suspension bridges; however, with the collapse of Tweed and Saale bridges, which had been built over rivers at the beginning of the nineteenth century, the idea of cable-stayed bridges was abandoned [Troitsky, 1988]. Roebling and others later introduced some cable-stays in suspension bridges to reduce deformability, such as for the Brooklyn Bridge, and some innovative suspension systems, very close to those of cable-stayed bridges, were invented, such as those by Gislard [Virlogeux, 1999]. The first “modern” cable-stayed bridges were built using concrete by Eduardo Torroja in the 1920s (Tampul aqueduct) and by Albert Caquot in 1952 (Donze`re canal bridge; Fig. 2.1); but the real development came from Germany with papers published by Franz Dischinger and with the famous series of steel bridges crossing the river Rhine [Dischinger, 1949].

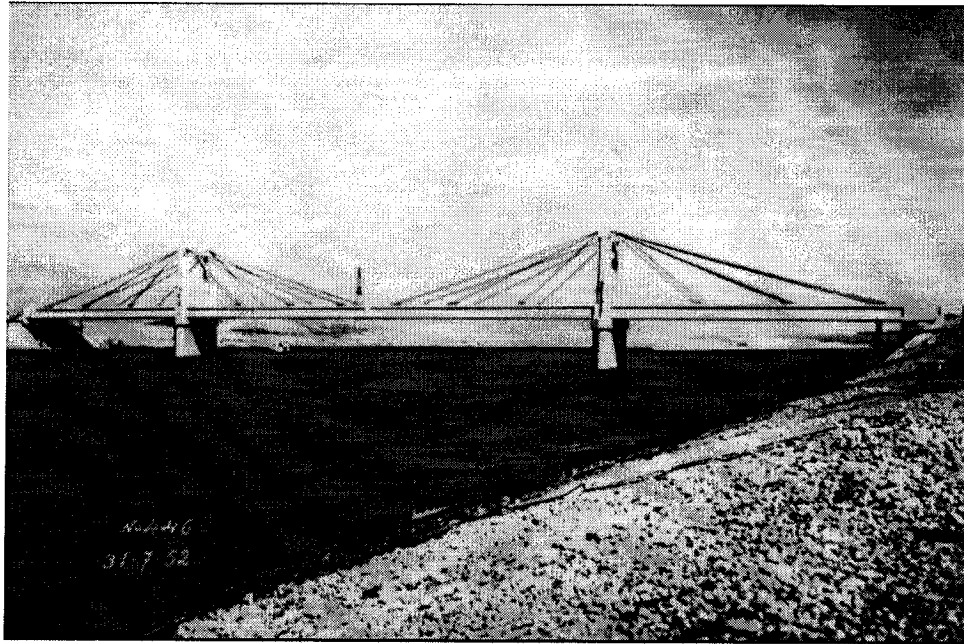


Figure 2.1 The bridge over the Donze`re canal in France (1952). Photo: J. Kerisel.

In 1970s, international progress on this bridge type first began, however a very big step forward took place in the 1990s, when cable-stayed bridges entered the domain of very long spans, which was previously the preserve of suspension bridges [Walther, 1999].

Recent years show that the evolution has been towards slender and flexible decks for medium spans: i) Composite decks, with two steel I-shaped beams of limited depth, even for rather long spans, up to 602 m for the Yangpu Bridge. Limits only arise with aeroelastic stability, due to the rather unfavorable shape of the deck, which calls for some aerodynamic amendments. In these bridges, the main problem is the possible development of cracks in the concrete slab; prestressing the slab where cables do not introduce high compressive forces can prevent this happening. ii) Prestressed concrete slabs stiffened by two rectangular edge beams. Very long spans can be built with such designs, such as the Penang Bridge in Malaysia or the Dames Point Bridge in Jacksonville, Florida (central span of 400 m, opened to traffic in 1988). iii) However, the greatest flexibility is reached with the rectangular slabs designed by Rene' Walther; the pioneer bridge is at Dieppoldsau over the Rhine (built in 1985 with central span of 97 m). The idea

was re-used by Jorg Schlaich for the Evripos Bridge, in Greece (215 m, in 1993), with a rectangular slab 0.45 m thick (Figs. 2.2–2.3) [Virlogeux 1999]. iv) Finally, ribbed slabs with streamlined shapes to avoid fairings and baffles, which were considered to be modification of imperfect designs: the depth of the ribs is limited and their width is enlarged, and the transverse cross-beams which connect them and support the top slab, preferably steel I-shaped beams for lighter decks, are of variable depth and follow the streamlined shape. An example of such design is the Ting Kau Bridge in Hong Kong that proved extremely efficient for aerodynamic stability, (Figure 2.4) [Walther 1999].

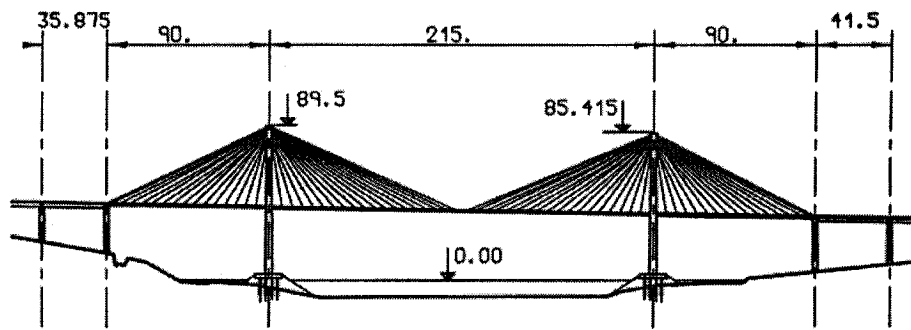


Figure 2.2. The Evripos Bridge: longitudinal view, (dimensions in m), [Virlogeux 1999].

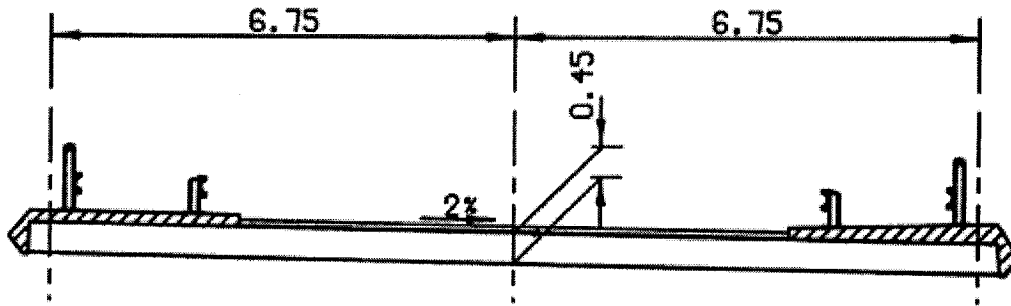


Figure 2.3. The Evripos Bridge: cross-section, (dimensions in m), [Virlogeux 1999].

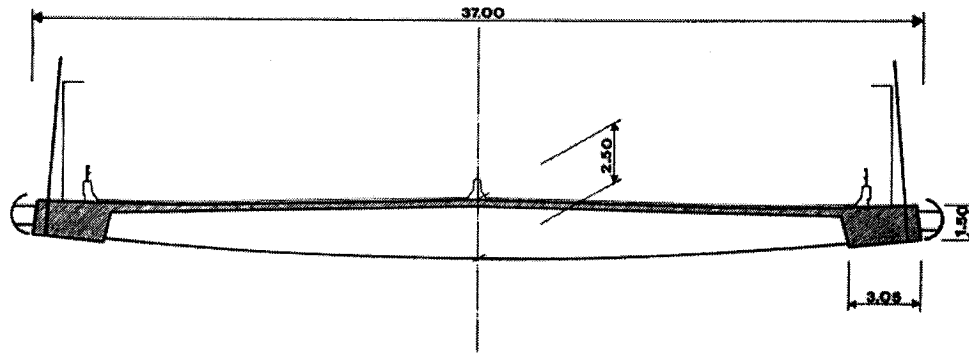


Figure 2.4. Cross-section of a project for the Ting Kau Bridge, (dimensions in m), [Walther 1999].

A typical cable-stayed bridge, as reported in the literature, is a continuous girder with one or two towers (pylons) erected above piers at intermediate points of the span. From these pylons, cables are attached diagonally to the girder. The increasing popularity of contemporary cable-stayed bridges among bridge engineers can be attributed to: (1) the appealing aesthetics, as the only part of the structure that extends above the road is the towers and “transparent” mostly harp shaped cables; cable stayed bridges have a simple and elegant look. As a matter of fact, they offer outstanding architectural appearance due to their small diameter cables, minimum overhead structure, and wide choice of design methods. This includes using single or multiple cables, in a fan, harp, bundle or star design. The pylon design can also vary significantly; (2) the full and efficient utilization of structural materials; (3) the increased stiffness over suspension bridges. Cable-stayed bridges offer greater stiffness, torsional and lateral rigidity over that of suspension bridges as their cables are tauter in comparison with the very flexible cables of the suspension bridges. This could help to make cable-stayed bridges even more stable against the wind and offer better performance with respect to aerodynamic effects; (4) the efficient and fast mode of construction; and (5) the relatively small size of the bridge elements [Ren 1999]. Cable-stayed bridges are very price competitive in the 150-600 m span length range. This creates stable point supports in the main span, which minimizes deflections. In addition, cable-stayed bridges have a low centre of gravity, which makes them strong against

earthquakes, but at the same time makes them vulnerable to uneven sinking of the ground [Abdel-Gaffar et al, 1984, Ali et al, 1995, and Ni et al 2005].

Cables are extremely well suited for axial tension, however they are weak against compression and bending forces. As a result, long span cable stayed bridges, though strong under normal traffic loads, are vulnerable to the forces of winds. Special measures are taken to ensure that the bridge does not vibrate or sway under heavy winds [Pfeil, 1994 and Saito, 1999]. The increase in span length of long span bridges results in a remarkable decrease in their natural frequencies and the ratio between the fundamental torsional and vertical mode frequencies. This renders long span bridges very susceptible to the actions of strong wind. The wind load effects generally become the most critical external loads that need consideration in the design of long span bridges [Chen et al, 2002].

### **2.2.2 World Record of Cable-stayed Bridge Central-Span and its Relation with Construction Materials:**

It is extremely interesting to analyze the progress in the world record for cable-stayed bridges, since it provides keys to understanding the evolution of their design and construction materials. It has been proved that very long span limits are predominantly controlled by (i) the materials strength to weight and stiffness to weight ratios, (ii) the bridge configuration that involves the ratio of the central span to the side spans, (iii) the stay-cables pattern, and (iv) the dynamic response to turbulent wind in addition to the cable vibrations. The optimization and control of these aspects now appear to be the major problems in very long span cable-stayed bridges [Virlogeux, 1999]. Figure 2.5 Shows world longest cable stayed bridges and the world record of cable-stayed and suspension bridges in comparison with other types of bridges.



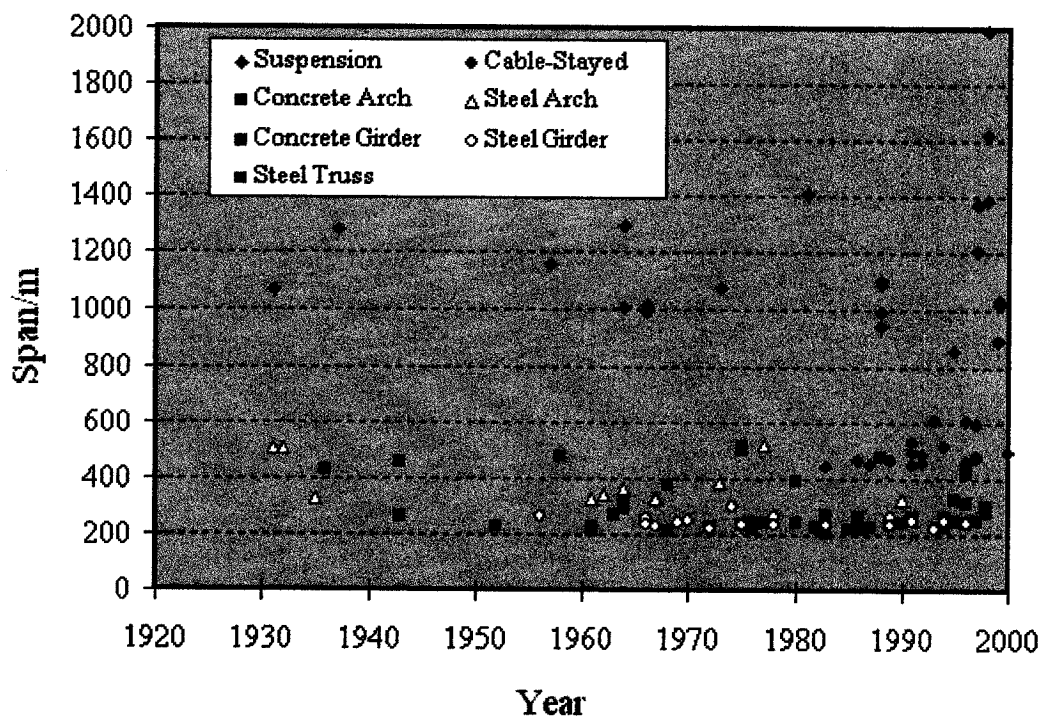
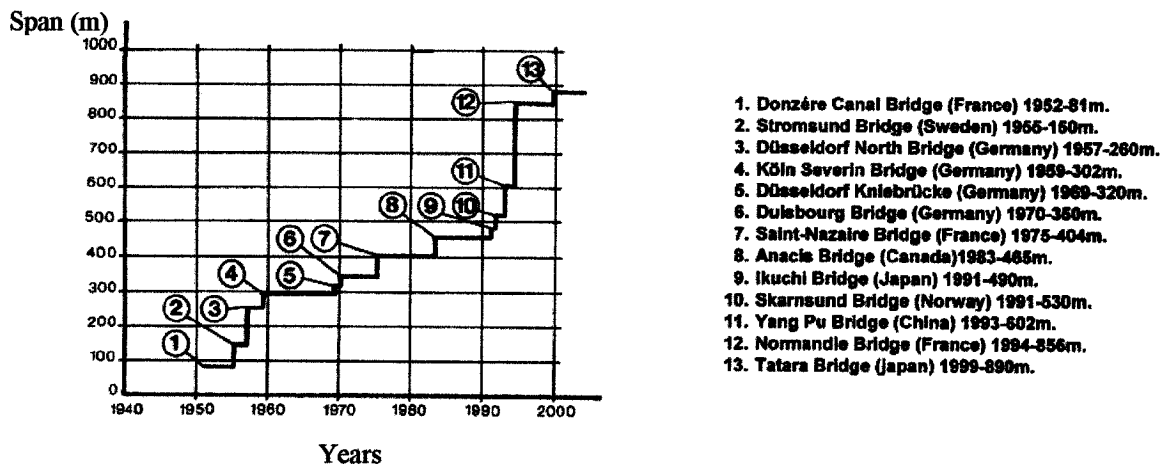


Figure 2.5. The world longest span cable stayed bridges in comparison with other types of long span bridges.

Two jumps in the development of cable stayed bridges central span lengths are shown in Figure 2.5; (i) The 1950s period when the modern bridge system was introduced, and (ii) the 1990s with the Normandy bridge of France and the Tatars bridge of Japan when a big jump in the central span has occurred. The second huge advance can be referred to: (a) the successful use of high strength steel and

the development of related construction technologies, (b) Balancing the bridge weight in the most effective way, using the steel for the central span and the prestressed concrete for the side spans, (c) the use of the advanced analysis methods together with the advances in the field of wind engineering.

The first cable-stayed bridges in modern times (Strösund Bridge in Sweden and North Bridge in Düsseldorf) were designed with steel decks. This is no longer the standard but is used in cases where weight is a concern. At the present time, decks and cables are usually made from steel. Concrete is frequently used for shorter spans. Composite sections made from steel and concrete, are also competitive with the steel sections for long spans. Figure 2.6 shows the growth of cable-stayed bridge spans using deferent deck materials, during the second half of the twentieth century.

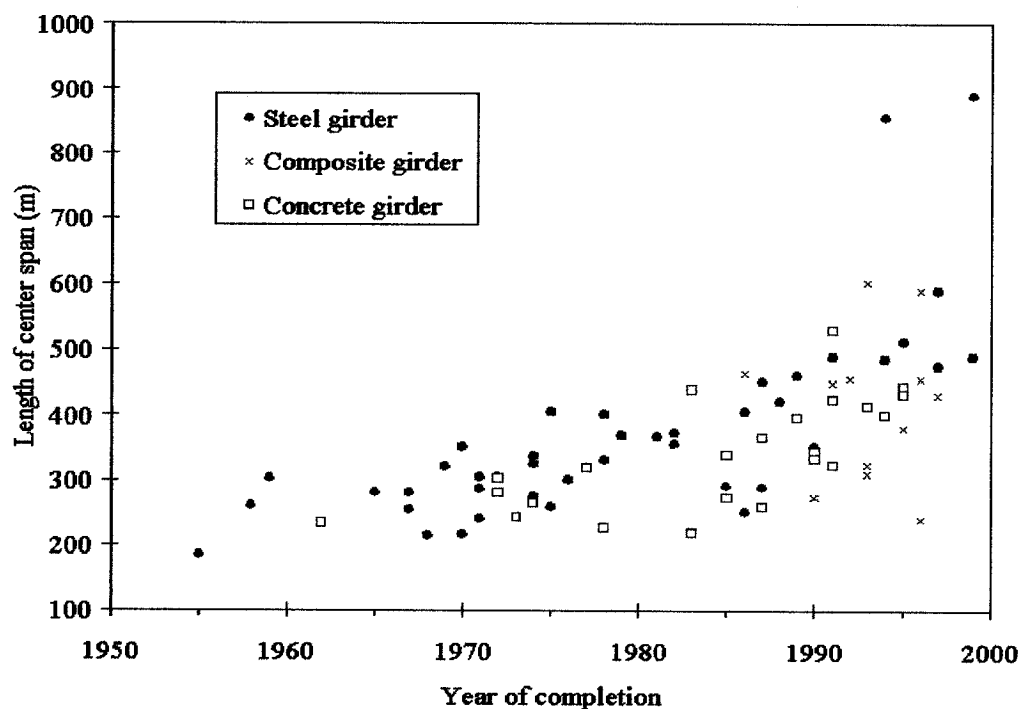


Figure 2.6. Cable-Stayed Bridge span for different designs.

### **2.2.3 Dynamics of cable-stayed bridges:**

With the rapid increase in span length, combined with the trend to use high strength materials and therefore more slender section for the bridge deck, concern has been raised about the dynamic behavior of cable-stayed bridges [Au et al 2001]. An accurate analysis of the natural frequencies and mode shapes of a cable-stayed bridge is fundamental to the solution of its dynamic response due to seismic, wind and traffic loads. Cable-stayed bridges exhibit complex behavior in which the translational and torsional modes are often strongly coupled. This coupled dynamic behavior increases the challenges involved in modeling the bridges for analysis and design purposes. In particular, the deck design may play a significant role in the modes coupling. Specifically, when the deck mass center does not coincide with the shear center of the section, an effort must be made to ensure that these couplings of the translational and torsional modes of the deck are accurately reproduced.

The stay cables have been modeled as single truss elements in most previous dynamic analysis studies, such as those by Morris [1974], Fleming and Egesli [1980], Wang and Huang [1991], Abdel-Ghaffar and Nazmy [1991], and Wilson and Gravelle [1991]. This approach is simple but inadequate for accurate dynamic analysis of cable-stayed bridges, because it essentially precludes the transverse cable vibrations and the resulting mode shapes account for the vibration of the deck and towers, while neglecting the dynamic interaction among the cables, the deck and the towers. This dynamic interaction has been studied by many researchers, such as Abdel-Ghaffar and Khalifa [1991], Tuladhar et al [1995] Caetano et al [1997 and 1998], and Au et al [2001]. They investigated the contribution of the stay cables in free vibration analysis of cable-stayed bridges using a variety of modeling methods. Their research results indicate that the modeling method of stay cables has a significant effect on the prediction of the dynamic characteristics of cable-stayed bridges. They proved that a single-element modeling of stay cables by the finite element method, even with sag allowance, would not be able to account for the cable vibrations. Accordingly, discretization of each stay cable into a series of elements helps to predict the cable

vibrations and the accuracy of the results generally improve as the number of elements used increases. On the other hand, the dynamic stiffness method is also valid for analyzing free vibration of cable-stayed bridges.

Chang et al [2001] investigated the dynamic characteristics of a 430 m main span length cable-stayed bridge in Hong Kong using finite-element analysis and ambient vibration measurements. A “three-dimensional” finite-element model was first established for the bridge based on design drawings. The dynamic characteristics were then analyzed from the statically deformed configuration. Ambient vibration measurements were also conducted to obtain the dynamic characteristics of the bridge. Comparison between these two results showed that for the most part a total of 31 modes could be correlated with a reasonable agreement. However, the frequency differences of the higher modes ranged between 15 and 30%. They suggested that if the measurements were more reliable, a finite element model updating is necessary in order to achieve better correlation.

#### **2.2.4 Methods of Analysis, Parametric Studies, Stability Studies and nonlinearity:**

During the last two decades, many parametric and stability studies, using linear and nonlinear models, have been conducted on cable-stayed bridges to demonstrate their promising capability to compete against suspension bridges in the very long span range. With the possible variation of design concepts, for example the use of twin girder systems, multi-cable planes, new spread-pylons and partial earth anchoring of the cable systems, cable stayed bridges should be seriously considered as an alternative to the suspension bridge system for even a most extreme span of 3200 m [Gimsing et al, 1987, and Starossek et al 1996]. Since the span is large and the cable stays are long and under high pretension force, the non-linearity due to cables sag, compression effect in pylons and girders in addition to the deck large deflections, have to be taken into account.

A cable stayed bridge is treated classically as a statically indeterminate structure with large degree of redundancy. The girder is approximating as a continuous beam, elastically supported at the points of cable attachments and supported on rollers at the towers. If non-linearity due to large deflections, catenary action of cables, and beam-column interaction of the girder and tower elements are neglected, the structure can be assumed to be linearly elastic. In such a case, the linear methods of analysis can be applied and the principle of superposition is admissible. Troitsky and Lazer [1970] developed a load-balancing method of analysis of cable-stayed bridges suitable for hand computations. A mixed force-displacement method was developed by Smith [1967], which he extended [Smith 1968] to the analysis of double plane cable-stayed bridges and treated the deck as a plate. Other analysis techniques have also been proposed such as; Lazer et al. [1972] used a flexibility approach for theoretical analysis, Tang [1971] illustrated the use of the transfer matrix approach and Kajita and Cheung [1973] illustrated use of the finite-element procedure. The well-known stiffness approach, and more general finite element method, seems to be the dominant tool employed for analyzing cable-stayed bridges.

Nazmy and Abdel-Ghaffar [1990] presented a static three-dimensional nonlinear analysis of long-span cable-stayed bridge under its own dead weight and a set of initial cable pre-tensions. All sources of geometric nonlinearity, such as cable sag, axial force-bending moment interaction in the bridge deck and towers, and change of the bridge geometry due to large displacements are considered in the analysis. The study results show that cable-stayed bridges are highly non-linear, especially for very long spans.

Adli and Zhang [1994], following a non-linear model of prestressed concrete cable stayed-bridge by Saif and Dilger [1990], presented a “fully” non-linear analysis of composite girder cable-stayed bridges. Three types of stay arrangements; fan, semi-fan and harp were considered. By incorporating material and geometrical nonlinearities, the effect of various nonlinearities on the failure load and near-collapse behavior of composite girder cable-stayed bridges is investigated. They concluded that under dead or dead and live loads, the effect of

geometric nonlinearity is small (the difference is within 5% of the linear analysis results); however, material nonlinearity has a more significant effect, especially on the maximum value of the negative bending moment at the tower region of the deck. In addition, they found that the material and geometric nonlinearities have a significant effect on the near collapse behavior and failure load of a composite girder cable-stayed bridge. The maximum bending moment for the case of a fully non-linear analysis is about 40% of the corresponding value of the linear analysis. In addition, different cable arrangements change the initial force distribution of the structure and its failure load substantially. They also found that the semi-fan composite girder cable-stayed bridge provides the highest failure load capacity, followed by the fan type bridge. The fan composite girder bridge, however, is the stiffest bridge, with the least amount of deflection. The harp composite girder cable-stayed bridge has the smallest failure load capacity and therefore is not recommended for long span bridges.

Argawal [1997] used a classical linear stiffness method in his parametric study on steel cable-stayed bridges. The models for this linear analysis were assumed to be the equilibrium configuration of the bridge under the effect of the dead loads and prestress providing that the initial configuration was not affected by the deformations. The cables were assumed to be straight members and perfectly flexible, the flexural rigidity of the cables was neglected and the cable-girder and cable-towers joints were assumed pinned. The effect of all nonlinearities was ignored in the analysis. The research concluded that i) the maximum cable tension decreases rapidly with the increase in the number of cables, ii) in the case of side span ratios of 0.35 and 0.40, the cable tension increases with decrease in the length of the central panel. For side to main span ratios of 0.45 and 0.5, for the smaller number of cables, it increases with decrease in the length of the central panel, but for a larger number of cables, the trend reverses, iii) the effect of length of the central panel on the sagging moment is significant, while with a hogging moment, the effect is not appreciable, iv) Both the sagging and hogging moments increase with increase in the number of cables, and v) For a smaller number of

cables, the total weight of steel in the harp arrangement is larger compared to that of the radiating arrangement.

Wang and Yang [1996] presented a two-dimensional finite element procedure for the initial shape and static deflection analysis of cable-stayed bridges. They had to take into account the non-linearity induced by cables sag, beam-column and large displacement effects. They investigated the individual influence of such sources of nonlinearity on the analysis of cable-stayed bridges. They concluded that since pretension forces are highly variable according to the loading stage, the prestress and the geometry of cable-stayed bridges are dependent on each other. For the initial shape analysis of cable-stayed bridges (under dead loads and cable pretension, before the application of the live loads) the inclusion of cables' sag nonlinearity has a significant effect on the displacement. At this stage, the cables are under minimum tension forces and the cables' sag is at its maximum value. The beam-column and deflection nonlinearities are the smallest and they have negligible effect on cable tension calculation. For the final stage, in what they called the "static deflection analysis", the contributions of the different nonlinearities were revised and it was shown that the large deflection and beam column non-linearities have significant effects on the results divergence with the exact solutions for the cases they studied. On the other hand, the cable sag nonlinearity has a less, and generally insignificant, effect. They finally concluded that a linear analysis of the final static deflection stage of cable-stayed bridges is meaningless and is not acceptable, since large axial forces exist in all the structural members causing large deformations. In this research, the effect of the unsymmetrical loading in the third dimension, the nonlinearities influence on the aerodynamic stability analysis and the use of different materials for the deck are not considered.

### **2.3 Advanced Composite Materials:**

Throughout history, it is evident that progress in many fields is closely linked to the development of new materials. This is apparent in structural engineering; for example, in the ancient history, the invention of brick reinforced by straw enabled

the Sumerians and Babylonians to build their great towers and temples, without using stone. The inventions of structural steel and reinforced concrete also changed the face of our modern civilization.

Originally developed for the aerospace and defense industries, advanced composite materials are increasingly being used for the construction of bridges, railway infrastructure, offshore structures, building frames etc [Vinson et al, 2002]. The importance and impact of new materials on export competitiveness, job growth and general prosperity is significant [Jones, 1999]. Advanced composites form one class of new materials that have a major impact on the aerospace industry and many other industries, and is starting to have a strong influence on research in civil-structural engineering, especially bridge engineering.

Extensive research has been conducted in the field of advanced composite materials, both in modeling their complicated anisotropic properties, starting from the micro level to the structural level, and in their application to innovative design of civil-engineering structures. The focus here is on research in two fields: i) Homogenization or Micro/Macro level modeling of advanced composites, and, iii) The recent applications in structural engineering, specifically bridge engineering.

### **2.3.1 Homogenization:**

An important goal of research in continuum micro-mechanics is the modeling of the macroscopic overall response of composites in an averaged or “homogenized” sense. The conventional mechanics of materials is to a large extent, based on the concept of Representative Volume Element (RVE). Mathematically, the RVE is an infinite length scale limit relative to the micro-scale, or the length scale of a single heterogeneity, in which the material appears uniform and the continuum concept may be applied. Consistent with this uniform continuum setting, the methods of analysis, particularly the boundary value problems, can successfully be applied. However, many problems of mechanics of materials require a careful consideration of dependent fields over scales not infinitely larger than the



microscale. This requires the introduction of meshes with elements that are finite relative to the microscale, rather than infinite. In effect, the constitutive response of elements displays a non-zero statistical scatter and the entire simulation procedure should be then be modified to a stochastic finite element method (SFE), rather than a deterministic finite element method (DEF). A careful observation shows a conflict between the DFE, in which the refinement of element meshes controls accuracy and stability, and the postulated RVE concept, which requires elements to be much larger than a single heterogeneity.

Ostoja-Starzewski et al [1999] presented a stochastic finite element method aimed at the introduction of micro-structural material randomness below the level of a single finite element. A consideration of the dependence of effective moduli on scale, and on the defining boundary conditions, leads to an identification of a finite element as a mesoscale window (or a mesoscale finite element) in a stochastic finite element method. An estimation of the global response can be obtained through bounds stemming from minimum potential energy and complementary energy principles. While in the classical case of homogeneous materials, these two bounds converge as the finite element mesh becomes sufficiently fine, and an optimal mesoscale with respect to the difference between both bounds may exist in the case of heterogeneous material.

Pellegrino et al [1999] developed a numerical method to evaluate the homogenized constitutive relation for the global behavior of periodic composite materials with elasto-plastic components. The algorithm presented is general and can be applied to any kind of non-linear material behavior.

In previous research shown earlier, the stochastic models were originally derived to model composites with random inclusions of very fine microstructure (paper, duplex steel, particulate composites with very fine insertions). In the present research, the approach is extended to be applicable on less fine microstructure with fiber dimensions in the range of the regular glass, steel, or carbon fibers for regular mechanical- or civil- advanced composite structural application. A simplified random system relating the random fiber fraction in the mesoscale to the size of the element is available through introducing a variable “probability

band width". The use of multi-type-fibers in the chopped FRP by introducing a multi-layer (or transparent layers) of separated random distribution generators is proposed. In addition, a filed problem, which is the stress/strain analysis, is involved in the SFE program development. Such an application can be experimentally performed for further adjustment and comparison.

Luciano and Barbero [1994] analyzed the mechanical behavior of composite materials with periodic microstructure. The corresponding elastic problem is solved using the Fourier series technique, assuming the homogenization strain to be piecewise constant. The coefficients of the overall stiffness tensor of the composite material were expressed analytically in terms of elastic properties of the constituents (fibres and matrix) and as a function of nine triple series, which take into account the geometry of the inclusions. In the case of composite materials reinforced by long fibers, simple formulas for evaluating these series were derived. Closed-form expressions for the elastic moduli of the fiber reinforced composite with periodic microstructure, and for the equivalent transversely isotropic material, were obtained. The model outputs were compared with experimental results.

### **2.3.2 Advanced Composites in Bridge Structures:**

In recent years, attention has been focused on FRP shapes, as alternative bridge deck materials, because of their high specific stiffness and strength, corrosion resistance, lightweight, and potential modular fabrication and installation that can lead to decreased field assembly time and traffic routing costs. Despite the overall benefits of using FRP sections, they are as yet not widely used. Besides concerns with material costs, in comparison with conventional materials, the complexity of material properties and structural behavior has deterred the rapid development of design codes for practicing engineers, [Mosallam et al 1992].

#### **2.3.2.1 Analysis and Design of Advanced Composite Bridge Decks:**

A critical obstacle to extensive use and applications of FRP structures in construction is the lack of simplified and practical design guidelines. Unlike

standard materials, FRP composites are typically orthotropic or anisotropic, and their analyses are much more difficult. For example, while changes in the geometry of FRP shapes can be easily related to changes in stiffness, changes in the material constituents do not lead to such obvious results. In addition, shear deformations in pultruded FRP composite materials are usually significant and, therefore, modeling of FRP structural components should account for shear effects. For applications to pedestrian and vehicular FRP bridges, there is a need to develop simplified design equations and procedures, which should provide relatively accurate predictions of bridge behavior and be easily implemented by practicing engineers. In the design of FRP structural members, both stiffness and strength properties are equally important and depend on the material system and the geometry of the cross-section [Mosallam et al 1992].

Davalos et al [1996] presented a comprehensive approach for the analysis and design of pultruded FRP beams in bending. The research was based on Barbero's [1991] proof that the material architecture of pultruded FRP shapes can be efficiently modeled as a layered system. Using the information provided by the material producers, a detailed procedure was offered for the computation of fiber volume fraction ( $V_f$ ) of the constituents, including fiber bundles or roving, continuous strand mats, and cross-ply and angle ply fabrics. By means of the computed  $V_f$ , the ply stiffness is evaluated from selected micromechanics models. The research showed that wall or panel laminate engineering constants can be computed from the ply stiffness and macro-mechanics. The predictions were well correlated with coupon test results. The bending response of various H and box sections was studied experimentally and analytically. The mechanics of the laminated beams (MLB) model, which was used in the study, accurately predicts displacements and strains. It had also been shown that the experimental results agreed closely with the MLB predictions and finite element verifications.

Salim et al [1997] presented a comprehensive study on the analysis and design of a fiber reinforced plastic (FRP) composite deck-and-stringer bridge. The FRP decks considered (Figures 2-7, 2-8) consist of continuous thin-walled box sections and were fabricated by bonding side-by-side pultruded thin-walled box sections

and were placed transversely over FRP composite stringers. In this research, the modeling and experimental verification of FRP structural beams [Davalos, 1996], including micro/macro-mechanics predictions of ply and laminate properties, beam bending response, shear lag effect, and local and global buckling behaviors, were reviewed. A simplified design analysis procedure for cellular FRP bridge decks was developed.

Qiao and Davalos [2000] presented a systematic approach for the analysis and design of all-FRP-deck-stringer bridges. To verify the accuracy of orthotropic material properties, an actual deck was experimentally tested and also analyzed by a finite element model. An approximate series solution for orthotropic plates, including first-order shear deformation, was applied in order to develop simplified design equations, which account for load distribution factors under various loading cases. The bridge systems were tested under static loads for various load conditions, and the experimental results were correlated with those by an approximate series solution and a finite element model.

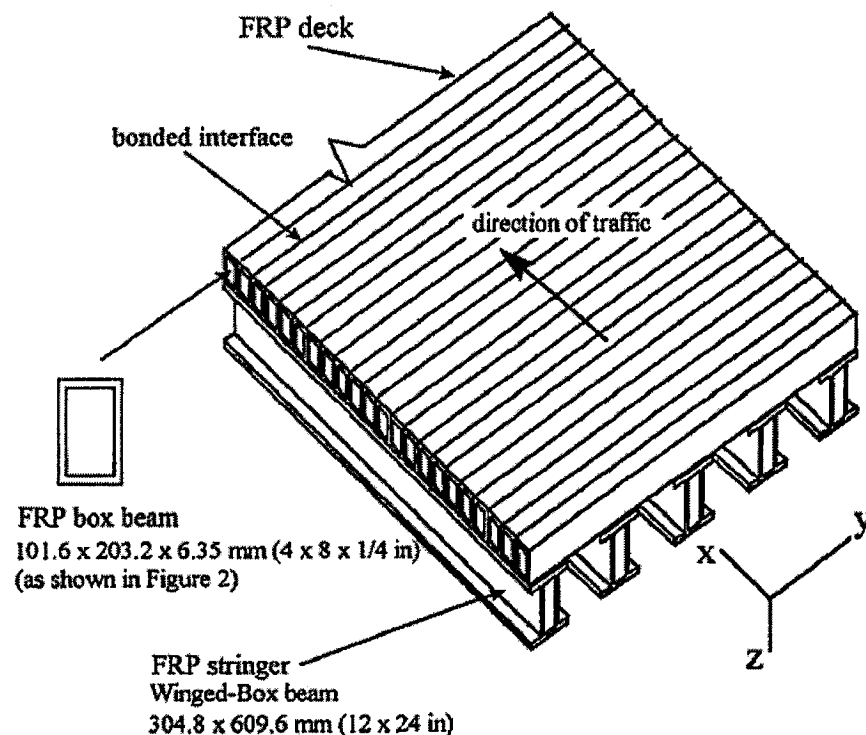


Figure 2.7 Schematic of an FRP deck on FRP stringers.

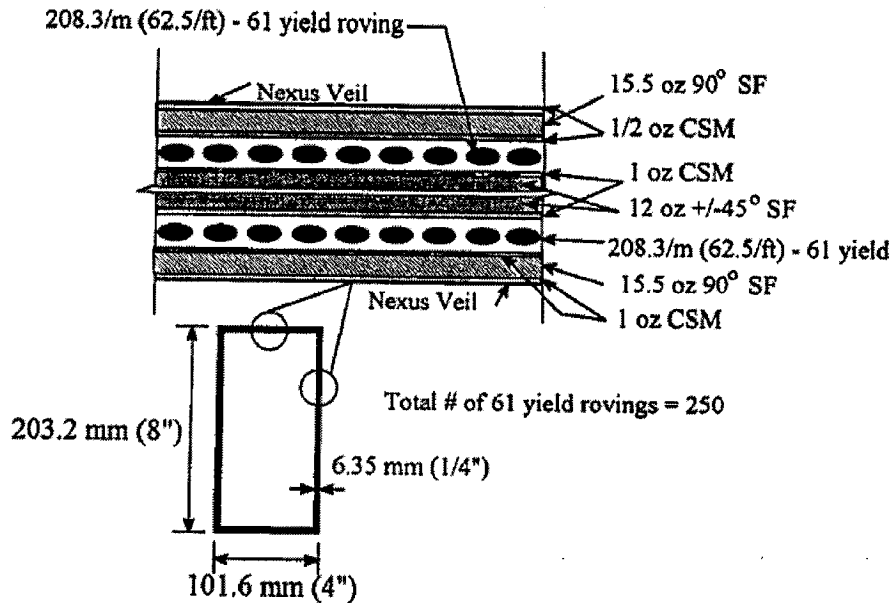


Figure 2-8.a. Material lay-up of typical FRP box beam section.

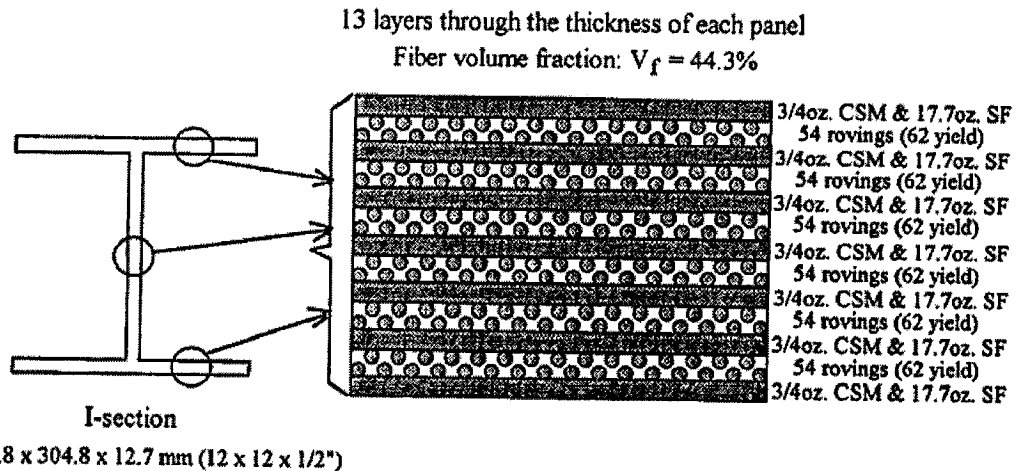


Figure 2-8.b. Dimensions and panel fiber architectures of WF-beam.

Aref and Parsons [2000] presented an integral fiber reinforced plastic bridge structural system. Figure 2-9 shows their bridge design, which consists of a series of inner cells surrounded by an outer shell. The bridge was manufactured using the filament winding process. The researchers showed that several types of reinforcement could be used, including fibers, prepreg tape and rovings. The given design had several important features. The circular shapes, used in the inner cells and the outer shell, avoid stress concentrations that are present at corners of

an I- or box-section. Effective transfer of shear through the bridge was achieved by providing large contact areas between the inner cells and the outer shell. Diaphragms were used at the end of the cells to provide stiffness to the cells at the supports. It had been shown that the stiffness and strength of this conceptual design were sufficient to support vehicular loads, and compared favorably with other fiber reinforced plastic bridge designs.

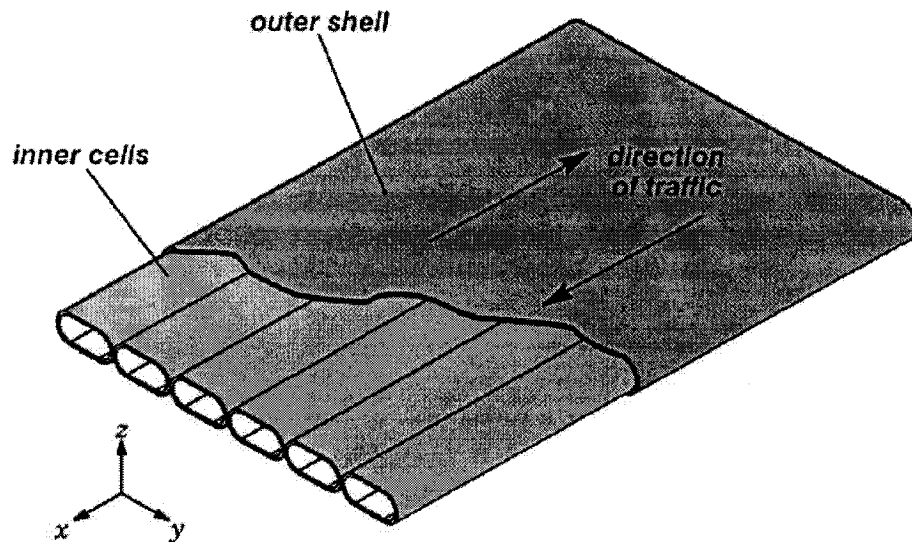


Figure 2-9 Component of Aref & Parsons bridge deck.

The development of optimization design procedures has emerged as an essential component primarily to overcome the challenge of high initial construction costs associated with the fiber reinforced polymer (FRP) bridge deck systems. He and Aref et al [2003] presented a genetic algorithm-based optimization procedure to minimize the weight of FRP web core sandwich bridge deck systems

Burgueno et al [2001] tested, at large scale, a system comprised of concrete filled, filament wound, circular carbon/epoxy girders and an E-glass/polyester deck (Figure 2.10), representative of a bridge section in the positive moment region in order to assess the overall and component structural response. The system was characterized for stiffness and overall response under monotonic and cyclic fatigue loads.

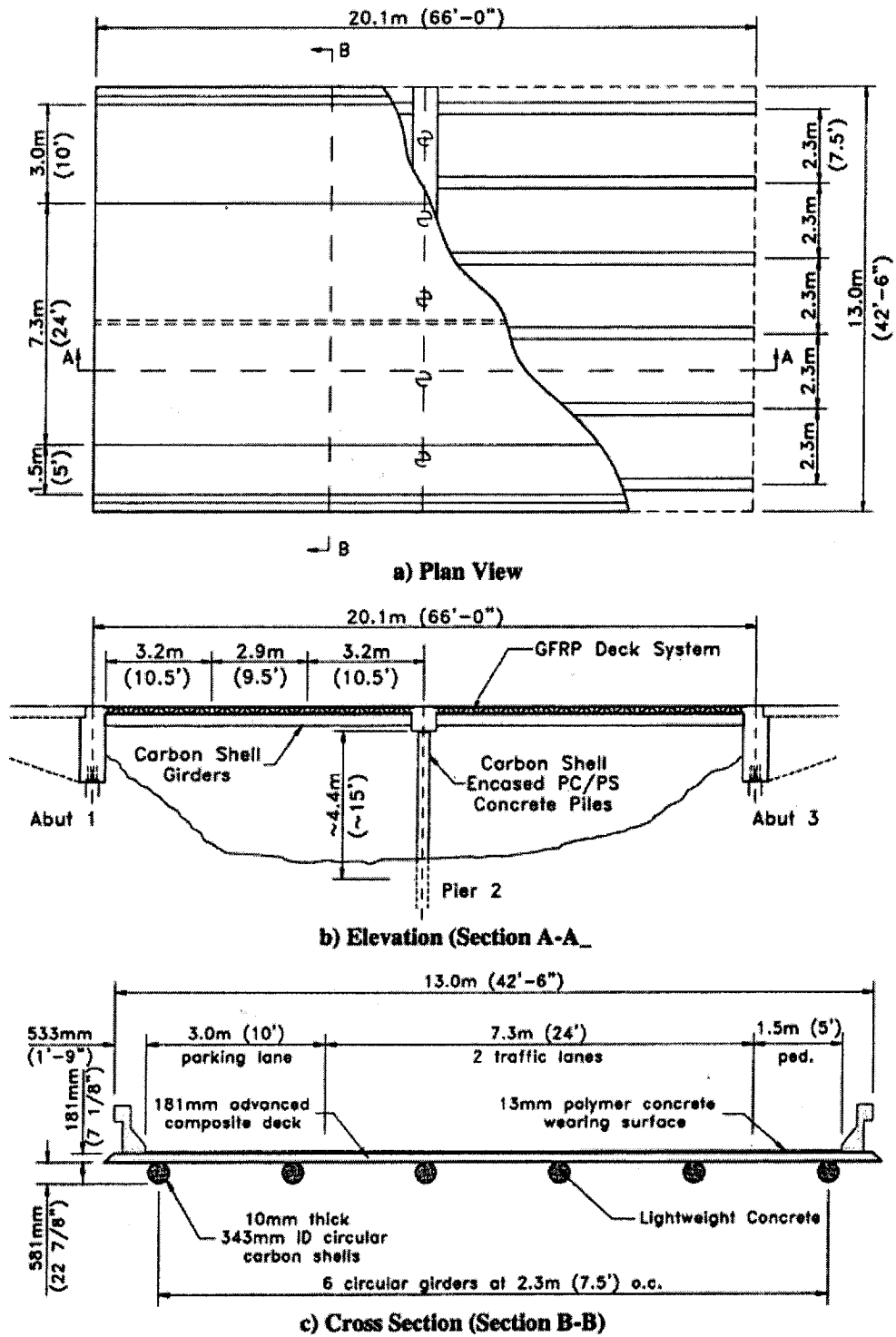


Figure 2.10 Geometry of the Kings Stormwater channel bridge, [after Burgueno et al 2001].

### **2.3.2.2 CFRP Cables:**

Stay cables are the main structural elements of cable-stayed bridges. Unlike many other bridge types, these components are completely exposed to environmental conditions. It is thus of general interest to make such cables as corrosion resistant as possible. Advanced composites such as carbon or aramid fiber reinforced plastics offer properties like high specific strength and stiffness, and high fatigue and corrosion resistance. They can be an excellent alternative to conventional steel by extending service life, reducing maintenance and simplifying installation. Their outstanding corrosion resistance, their light weight and their high specific strength make them prime candidates for bridge applications such as Stay Cable or conventional box girder structures. Furthermore, due to their high strength to weight and high stiffness to weight ratios, they allow the construction of more efficient long-span bridges [Meier et al, 1996].

BBR Company (EU) in collaboration with EMPA (Swiss Federal Laboratories for Materials and Testing) pioneered their introduction in the twin span "Storchenbrücke", which is a Cable Stay Bridge near Winterthur in Switzerland. Two out of the 24 stays of this Bridge were substituted with CFRP cables, consisting of parallel wire bundles, each having 241 wires of 5mm dia. (as shown in Figure 2.11). The extensive monitoring of both CFRP cables since traffic opened in April 1996 produced excellent results [BBR Ltd. 1996]

The CFRP wires for the Storchenbrücke cable-stayed bridge (Figure 2.12) with diameter slightly over 5 mm contain about 24,000 single carbon fibres, totaling 400,000 fibers. Each single fiber has a diameter of about one tenth of a human hair. The total section area of a CFRP wire consists of 70% carbon fibres and 30% epoxy matrix [Wernli, M. et al, 1999].



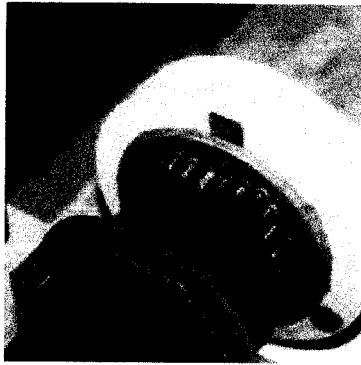


Figure 2.11 CFRP cable for cable stayed bridge

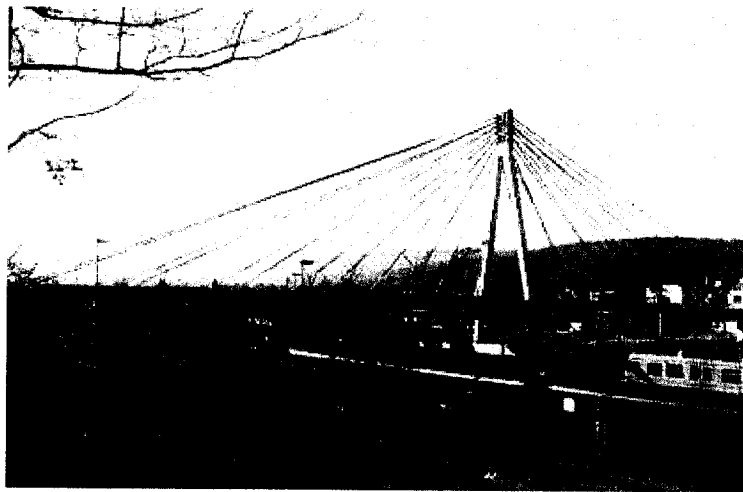


Fig. 2.12. Storchenbrücke Cable-Stayed Bridge (Winterthur, Switzerland).

### **2.3.2.3 All-Advanced Composite Cable Supported Bridges:**

#### **2.3.2.3.1 Constructed FRP Bridge:**

The first major application of an All-Composite-Cable-stayed-System was the Aberfeldy Footbridge (Fig.2.18) in the UK, which has been extensively reported in the literature. It joins two halves of a golf course on either side of the River Tay. The main structure is a cable-stayed bridge that consists of a GFRP deck, suspended by Parafil-Aramid ropes from GFRP towers. Although originally designed only for pedestrians, it has recently been strengthened locally by the addition of CFRP at some highly stressed locations in order to deal with golfers' motorized "buggies". The very lightweight allowed mass to be added locally to the structure, by filling some of the tubes with concrete, so that the flexural and

torsional natural frequencies could be separated. The lightweight of composites offers advantages when moveable bridges are required, since significant savings in the machinery can be realized.



Fig. 2.13. Aberfeldy Bridge

#### **2.3.2.3.2 Proposed Bridge Projects:**

Meier [1986] proposed a carbon fiber reinforced composite bridge across the Straits of Gibraltar. The mid-span of the bridge would be 8400 m with side spans of 3100 m and 4700 m. The height of pylons would be 850m and 1250m, with foundation depth of 300m. Although huge dimensions of the proposed cable-stayed bridge raise a great number of problems (for example, resistance against wind loads and aerodynamic stability, long term reliability, etc.) remain to be solved, the researcher proved that such a bridge appears possible from the technical standpoint within the next thirty to forty years.

To demonstrate the application of advanced composite materials in the cable-stayed bridge field, The University of California, San Diego and The Federal Highway Administration (1993-2004) presented a hybrid single pylon short span cable stayed bridge, which is designed utilizing advanced composites. The bridge caters for 4-lane traffic (18.3 m total width) and is 137.2 m long, using cable-stays supported by 57.9 m high A-frame single pylon. The (I-5/Gilman) advanced composite cable-stayed bridge system is a dual plane, fan type, cable-stayed bridge with an eccentric type pylon (Figures 2.14 and 2.15), [Zaho et al 2004].

The design concept consists of a dual cable plane system supporting concrete filled carbon/epoxy tubes, connected in the transverse direction by hybrid E-glass-

carbon reinforced FRP hollow cross-beams spaced 2.4 m on center, which in turn support a longitudinally spanning fiber reinforced concrete deck. The system includes a set of cables every 4.9 m. It has a fixed support condition at the west abutment, for torsional rigidity, and a roller support condition with torsional restraint at the east abutment that allows for thermal expansion movement. The single A-frame type pylon system is located on the west side of the freeway, allowing a dual set of stays radiating outward towards the edge beams. The design uses conventional concrete abutments and foundations. The primary structural components in the I-5/Gilman Advanced Composite Cable Stayed Bridge are the deck system, the longitudinal edge girders, the transverse beams, the pylon, and the cable stays.

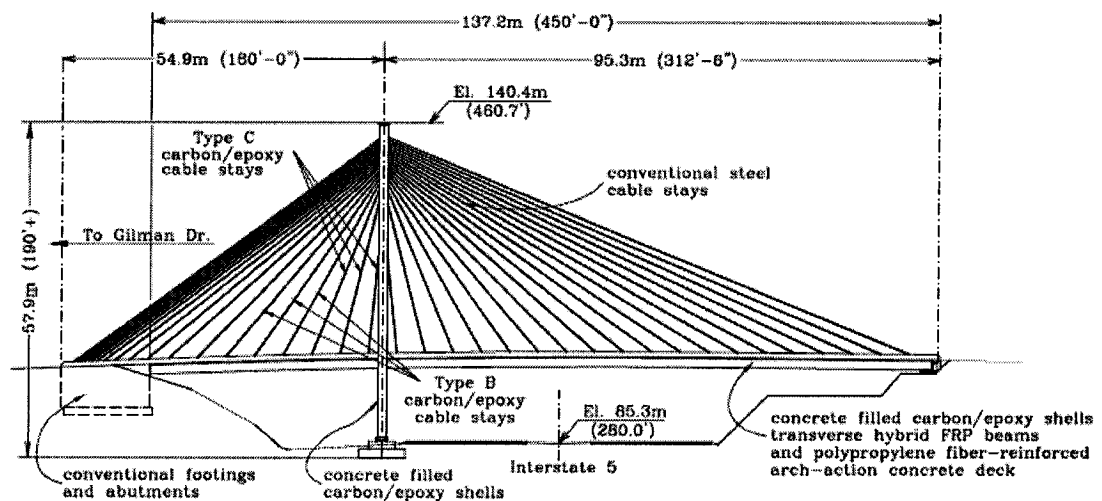


Figure 2.14. Elevation of the I-5/Gilman advanced Technology Bridge Geometry and Components



#### **2.4.2 The Field of Advanced Composite Bridges:**

As shown in the preceding section, many researchers have proposed, tested and constructed short span bridges using FRP and have proved their capability to support traffic loads effectively, achieve high fatigue and deterioration resistance, which adds to their light weight and fast construction. The present research extends the achievements in the field of advanced composite short span bridges and proceeds to propose a new system capable of bridging very long spans.

The application of advanced composite materials to short span pedestrian cable stayed bridges appears to be arbitrary. The various proposed or constructed bridges have not been developed to a standard bridge style. In addition, no analysis or design procedure has been proposed for such bridges. GFRP materials were used in all the bridges components, including the pylons, and the bridges were design to support the dead and live loads only. Accordingly, the very low stiffness of those bridges made them very flexible, resulting in very low and coupled natural frequencies, and hence their dynamic stability is suspicious. On the other hand the I-5/Gilman Bridge proposal represents a major advance towards the use of FRP materials in short span cable stayed bridges under regular traffic design loads. However, the system is similar to the traditional slab-girder cable-stayed bridges with partial use of FRP material in the bridge components. The deck section is basically a concrete slab stiffened by lateral FRP beams (perpendicular to the traffic direction) and supported by two major longitudinal FRP pipes filled with concrete. The Cable stays are combined CFRP cables and Steel cables. The bridge span is relatively short and the aerodynamic stability has not been investigated.

Although many proposals predicted the possibility of building a long span cable stayed bridge using advanced composite materials in the next few decades, none of them have proposed a structural system or investigated its behavior, modeling or performance. The discussions among these proposals were focused on the life cycle, cost effectiveness and the lightweight of the material.

The present research proposes and investigates, in detail, the performance of a hybrid cable stayed bridge system capable to bridge very long spans. It takes into

account the anisotropic properties of the material and the resulting complication in the sectional design and modeling and proceeds to model the bridge system with all its geometrical, loading and stability complications. The following points summarize the contribution of the present research to the existing knowledge in the field of advanced composite bridges:

- i) Introduction of multi-fiber type FRP material as a new material for use in the bridge deck together with SFE procedures to evaluate the material properties and strength,
- ii) Introduction of four deck sections entirely formed from FRP materials to economically overcome the long span cable stayed bridges deck requirements through: (1) minimizing the material in the section, (2) minimizing the stress concentration and the related possible delamination, (3) maximizing the ratio of the torsional to the bending stiffness of the deck and hence increase the design flutter velocity, and (4) sizing the deck to match the boundary dimensions of very long span reference bridges and use the available aerodynamic section test result to investigate the aerodynamic stability.
- iii) For the first time, CFRP has been investigated as the only material for stayed cables of long span cable stayed bridges.
- iv) A proposal for a systematic and iterative analysis and design procedure involving the different stages of the design, modeling, and stability.
- v) The system was investigated in parallel with three of the longest cable stayed bridges in the world. This would show its behavior, capability, limitation and expectation.

## **2.5 Aerodynamic Stability of Long-Span Cable-Stayed Bridges:**

The consecutive uses of new materials as well as the improvements in the traditional materials and construction techniques have resulted in a new generation of flexible suspension and cable-stayed bridges. As the flexibility increases with span length, so does the susceptibility to wind-induced problems. The most renowned bridge collapse due to wind action is that of the Tacoma

Narrows suspension bridge linking the Olympic Peninsula with the rest of the state of Washington. While other bridges had been destroyed by wind action, the failure of the Tacoma Narrows Bridge shocked engineers and led to urgent scientific investigations of bridge aerodynamics. Engineers still debate the exact cause of its collapse, however there is a concurrence that the bridge collapse can be considered as an aerodynamic instability due to excitation of a torsional mode. The aerodynamic instability is a self-excited vibration and in this case, the alternating force that sustains the motion is created or controlled by the motion itself. It is important here to mention that the aerodynamic instability oscillation is not a resonant oscillation since the wind does not have a forcing frequency at, or near, the bridge's torsional mode frequency. The resonance phenomenon requires the driving force frequency to be at, or near, one of the system's natural frequencies in order to produce large oscillations. The turbulent wind pressure, however, would have varied randomly with time. Thus, turbulence would seem unlikely to have driven the observed steady oscillation of the bridge.. Hence, aerodynamic instability is a phenomenon caused by the interaction of wind and bridge vibration. When the input wind energy cannot be dissipated, vibration accumulates and the bridge becomes aerodynamically unstable [Cai and Albrecht 2000].

Structural instability appears when the deformed shape of the structure produces an increase in the excitation force to further increase structural deformation. Other classes of instability, such as compression buckling or lateral torsional buckling, very well illustrate this phenomenon. In the field of aeroelasticity, instability occurs when aerodynamic forces induced by wind velocity are modified as the structure deforms. If the new values of the forces act to further increase the displacements, this process leads to unstable behavior of the structure. In the following redrawn, the main classes of aeroelastic instability are outlined with more attention given to those problems related to long span bridges.

### **2.5.1 Aerodynamic Forces:**

Jurado and Herbández [2000], based on the book of Dyrbye and Hansen [1997], summarized the most important wind loads on structures as: **i) Vortex Shedding:**

A bluff body immersed in a moving fluid sheds, under certain conditions, alternate vortices, the appearance of which would create a vertical, or lift force.

**ii) Galloping:** This instability is typical in structures with rectangular or D-shaped geometry. It can originate, for example, in a circular cable with ice fixed to it and subjected to strong winds. Structures showing this instability present large oscillations in the across wind direction, with amplitudes many times larger than the characteristic dimension of the structure. Power transmission lines are the most common structural example for this type of behavior. **iii) Torsional Divergence:** The state when the torsional stiffness of the deck is equal to the static pressure of the wind times the first derivative of the aerodynamic moment coefficient with respect to the angle of rotation of the deck, in the deck original static configuration (i.e. angle of rotation equal zero). **iv) Flutter:** This class of instability is of significant importance in bridge design. It can be produced by a single degree of freedom as, namely vertical movements or torsional rotation, but more frequently it happens with the coupled effect of both degrees of freedom. Flutter analysis is usually carried out considering a typical cross section of the bridge and takes into account the set of aerodynamic forces. **v) Buffeting:** Buffeting is a three dimensional aeroelastic problem due to turbulent behavior of the wind flow. Wind turbulences create flow lines with different attack angles along bridge length. This circumstance produces a variable distribution of pressure that needs to be formulated by stochastic theories. According to the spectral distribution of velocity angles, some degrees of freedom of the bridge can be excited giving way to instabilities. Bridge response is regulated by the square of the mean wind velocity, velocity fluctuations, turbulence distributions, deck geometry, natural frequencies, and damping coefficient of deck. Buffeting behavior of bridges may be evaluated experimentally by wind tunnel testing. Modeling procedures of wind speed parameters were discussed by Tanaka [1992].

### 2.5.2 Modeling Aerodynamic Forces:

The aerodynamic performance under the action of strong winds is of major concern because it serves as a governing criterion for the design and construction



of long span bridges. Understanding of the aero-elastic response of bridges has been significantly improved through experimental and analytical studies. Remarkable developments in analytical approaches have been made since the pioneering studies by Davenport [1962] and Scanlan [1978], among others. These analytical approaches utilize aerodynamic forces linearized at the statically deformed position of the bridge, which are commonly separated into static, self-excited, and buffeting force components. Advances in identifying force parameters such as static force coefficients, flutter derivatives, aerodynamic admittance functions, and spanwise coherence functions, utilizing scaled bridge models in wind tunnels have led to remarkable improvements in accurately modeling aerodynamic forces and predicting the overall aeroelastic response of bridges [Walshe and Wyatt 1983; Davenport et al. 1992; Scanlan 1993; Sakar et al. 1994; Larose and Mann 1998; Chen and Kareem 2002]. In conjunction with experimentally identified force parameters, analytical approaches have been widely used in the design of many bridges. Conventional analytical techniques, which rely on the assumption that inter-modal coupling is negligible and flutter is dominated by a single torsional mode, have adequately predicted the response of typical cable supported bridges built thus far. However, experience has shown that this may not be a valid assumption for exceptionally long span bridges, which require a multimode-coupled analysis framework [e.g., Miyata et al. 1995; Jones et al. 1998; Katsuchi et al. 1999; Chen et al. 2000a; Xu et al. 2000; Chen et al. 2001].

Aeroelastic analyses have been predominantly conducted in the frequency domain because it facilitates the modeling of frequency dependent characteristics of unsteady aerodynamic forces. However, it is constrained by the assumptions of linearity in both structural dynamics and aerodynamics, and stationarity of wind fluctuations. Challenges in analytical approaches remain in the areas of modeling aerodynamic forces excited by non-stationary wind fields, such as hurricanes and thunderstorms, for bridges located in complex topography [Davenport and King 1993], consideration of nonlinearities in both structural dynamics and aerodynamics, and ubiquitous issues related to turbulence. Clearly, these

challenges can only be adequately addressed in a time or time frequency domain analysis framework.

Most of the studies concerning time domain analysis of buffeting response were based on quasi-steady aerodynamic forces, due to their inability to model frequency dependent unsteady aerodynamic characteristics in the time domain [e.g., Kovacs et al. 1992; Miyata et al. 1995]. Therefore, its application should be limited to those cases where the reduced velocity is sufficiently high such that the unsteady fluid memory effect may be ignored. Chen et al. [2000b] proposed a time domain analysis framework in which frequency dependent aerodynamic forces can be modeled akin to the frequency domain approach. The frequency dependent unsteady aerodynamic forces are represented by the convolution integrals involving the aerodynamic impulse functions and structural motions or wind fluctuations. The aerodynamic impulse functions are derived from experimentally measured flutter derivatives, aerodynamic admittance functions, and spanwise coherence of aerodynamic forces using rational function approximations. A significant feature of this approach, which is the frequency dependent characteristics of unsteady aerodynamic forces and the nonlinearities of both aerodynamic and structural origins, can be modeled in the response analysis. The flutter and buffeting response of a long span suspension bridge has been analyzed using the proposed time domain approach and the results showed a good agreement with those from the frequency domain analysis.

## **Chapter 3**

# **Development of Hybrid Advance Composite Long-Span Cable-Stayed Bridge**

## **Chapter 3**

### **Development of Hybrid Advance Composite**

### **Long-Span Cable-Stayed Bridge**

#### **3.1 Introduction:**

Development of a hybrid long span cable-stayed bridge, together with the analysis and design procedure, is presented in this chapter. The general geometric, materials and structural characteristics of the proposed new hybrid long-span cable-stayed bridges, in parallel with three of the world-longest span bridges having central spans ranging from 465 m to 890 m that were built during the last two decades, is also shown. The general architectural-geometrical designs of the reference bridges are approached using advanced composite materials. The focuses of this chapter are on the new hybrid bridge analysis, design requirements and procedures, the four proposed models section sizing, the structural compatibility and convergence.

### 3.2 Deck Sections:

Four advanced composite deck section models are proposed in this research. The general shape and detailed dimensions of the deck section are decided by two major considerations: (i) the long span cable stayed bridge requirements and (ii) the advanced material properties and macro/structural design requirements. It has been shown that a very thin deck section can support the static and traffic loads applied to a cable-stayed system [Virlogeux 1999 and Walther 1999]. The deck section dimensions in such cases are only dependent on the material strength and the level of compression in the pylon region. However, as the central span increases, the serviceability requirements and the aerodynamic stability entail further increase of the deck stiffnesses. Consequently, increasing the ratio between the torsional to the vertical bending natural frequencies, and reducing the possible coupling of these types of frequencies are becoming more essential. The boundary dimensions of the deck sections (the deck width, depth and the sides slope) of deck section models are approaching the boundary dimensions of the reference bridges. On the other hand, the advanced composites properties decide all the internal dimensions of the section including the inner cells dimensions, the minimum laminate thickness, the laminate design, the lamina micro design ...etc. All the major parameters of the bridge system are investigated in performance study of Chapter 6. The four deck section models are developed and introduced according to the following:

- Under low aerodynamic loads, the design of a cable-stayed bridge deck leads to a thin deck section, like a uniform thick slab or a very shallow box girder. This would results in reducing the deck vibrations due to the vortex shading. For such basic situation, the “Model II” deck section is introduced (Figure 3.3).
- As the bridge central span increases the bridge natural frequencies decrease and the deck deformations and vibration together with the aerodynamic stability of the bridge system become more critical. Accordingly, the deck stiffnesses should be increased, hence a constant depth deck section and “aerodynamically smoothed sides”, or Model I, is introduced (Figures 3.1 and 3.2). It is named “Model I” instead of “II” as the logical sequence assumes, because its boundary dimensions match those of the reference bridges deck sections.

- In order to reduce the possible stress concentrations and the resulting delamination due to the internal boxes of “Model I” deck section’s sharp corners, and to further increase the torsional to the vertical flexural stiffness ratio, the “Model III” is introduced (Figure 3.4). The internal boxes now replaced by “opened circular tubes”, which also simplify the manufacturing process of the deck.
- Recently, and referring to many long span bridge designs and design proposals, it has been proved that using a deck section with “two boxes” apart and an intermediate grid system results in very high torsional to vertical flexural stiffness ratio. Hence further separation and divergence of the torsional from the vertical natural frequencies is expected. On the other hand, the use of laminated FRP is more expensive than the use of randomly oriented FRP or “chopped FRP”; hence minimizing the laminated FRP is a benefit. Accordingly, Model IV is introduced (Figure 3.5) where the laminated FRP part is limited to the two side tubes and a stiffening laminate for the intermediate part of the section.

### 3.2.1 Model I:

The proposed cable-stayed bridge deck section, called “Model I”, consists of a series of inner box cells surrounded by an outer FRP binding box. A filament winding process is to be used in manufacturing this deck section. This would deliver a high-speed and accurate lay-up of fiber reinforcement around a mandrel. In this process the fiber winding angles are machine controlled to form the required stacking sequence specified by the design. Since large volumes of material are involved in the construction of the proposed bridges, the choice of filament winding is an attractive solution for automating the manufacturing process. The design has several important features. The designer can readily choose the material, winding angles and thickness of plies to comply with appropriate strength and stiffness criteria. Effective transfer of shear through the deck components is achieved by providing large contact areas approaching the full contact between the inner cells and the outer box in addition to the full contact between the top surface of the binding box and the deck top surface plate or “deck slab”.

The boundary dimensions of the deck section are variable and are defined by the traffic volume, the architectural and structural design, the aerodynamic stability requirements and the overall structural behavior of the bridge. In the present research, the dimensions of the deck section will be adjusted according to the reference bridges section-dimensions. Figure 3.1 shows Model I dimensioned for a bridge with the same dimensions as the Second Chang River Cable-Stayed Bridge in China, with a 628 m central span. The internal proportions and thicknesses of the section are complicated functions of the material properties, together with the stress and strain distributions resulting from the overall bridge structural analysis. The cable-stayed bridge is a stability-sensitive and complicated system; the analysis-design process is excessively convoluted and will be investigated throughout the different static loading stages and the dynamic analysis.

In contrast with the relatively simple stage of deciding the material type from the traditional well-defined list, more complications are added through a secondary-process of the advanced composite micro/macro scale analysis and design. The thickness of the “deck slab” or the “top surface plate” is a function of the punching stress due to the maximum wheel load of the design vehicle, the level of the maximum compressive stress in the pylon zone and the stress concentration related to the cables anchorage. On the other hand, its thickness is also dependent on the chopped FRP material properties, which are depending on the matrix and the fiber properties and the volumetric percentage of the fiber. The connection between this plate and the rest of the section is supposed to completely transfer the stresses; as such the section is assumed to behave integrally in its contribution to the overall cable-stayed bridge performance. In addition to its ability to distribute the concentrated load, the other leading function for this plate is smoothing the transfer of the stresses to the laminated material of the box girder to minimize punching failure occurrence, local buckling or delamination.

The stiffness and strength of the laminated box girder are other complicated multi-tasks, including the laminate structure and thickness, the FRP laminated box-girder cell dimensions, stress concentration at the cell corners, and the outer bonding box-inner cells interaction. The laminate behavior is a function of the number of the unidirectional-fiber-laminas, the volumetric content of the fiber in the lamina, the orientation of those

laminas, and the properties of the resin. The laminated cells and outer box dimensions, together with the different laminates thicknesses and lay-up, are the complex variables contributing to the stiffness of the overall section.

In the traditional materials, steel or reinforced concrete, the geometry of the structural section and the two simple isotropic material properties, the modulus of elasticity and Poisson's ratio, are the only properties to be defined before the structural analysis stage. However, in advanced composite structures, any change in the lamina's properties such as fiber type or its volumetric percentage or its orientation or the resin type ...etc, or the laminate structure, like the periodic sequence of the laminas or their orientation or thicknesses ...etc, would change the overall material properties and the three-dimensional deformation resulting from the anisotropy. More complicated failure criteria are also required to select the laminate properties and thicknesses for the laminated box girder part of the section. The Tsai-Hill criterion [Jones 1999] for anisotropic materials is used to check for the first ply failure. Figure 3.2 shows an example of the final macro design of the internal cells and outer binding box laminas, which are formed from twelve laminas each; the thickness and the angle of orientation of each lamina, are also demonstrated.

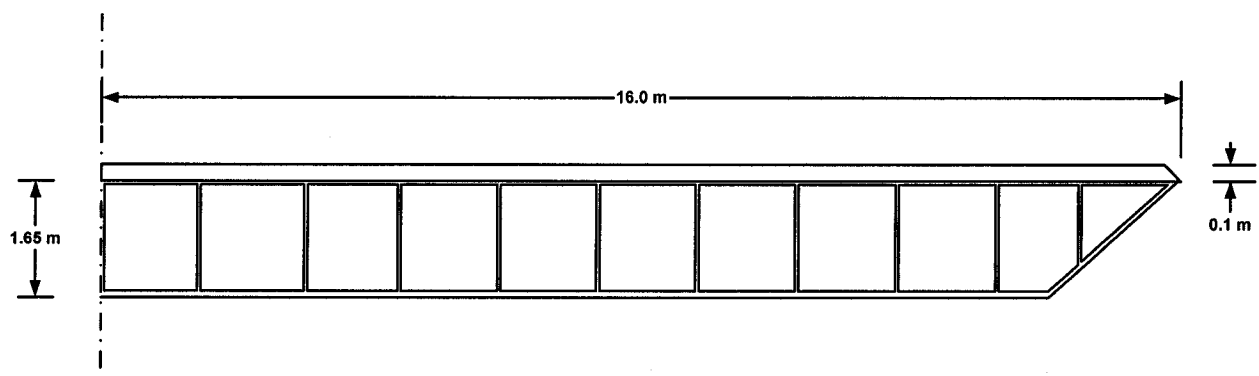


Figure 3.1 Model I dimensioned for Second Chang River Cable-Stayed Bridge.



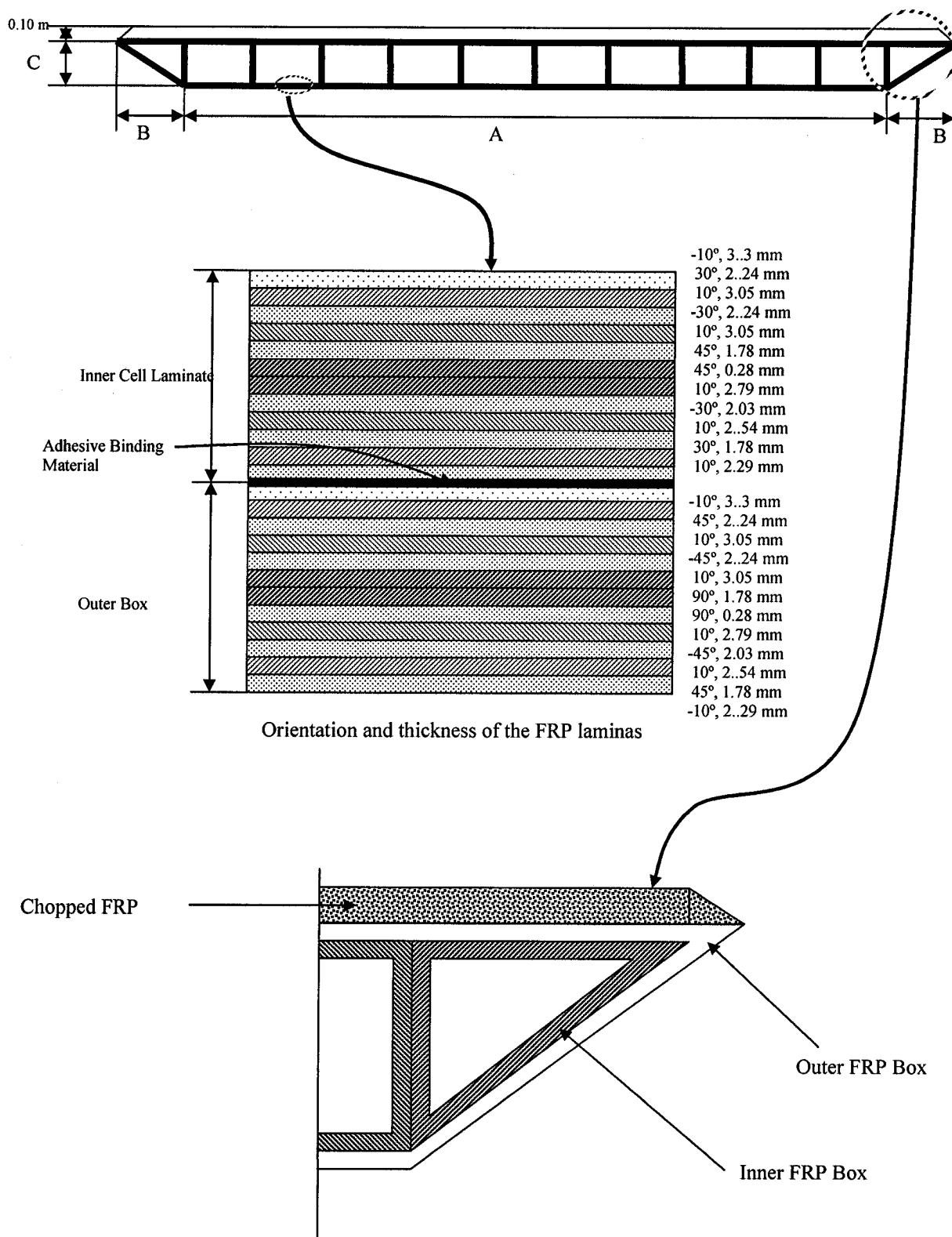


Figure 3.2 Model I: General geometrical and material properties

### 3.2.2 Model II:

In order to minimize the deck weight in cases where the aerodynamic stability is not the dominant concern, slender and flexible decks are frequently used in medium span and many long span bridges; for example, the Yangpu Bridge with a central span of 602 m [Virlogeux 1999]. Many deck sections are used such as slab sections, composite sections formed from two major steel girders and concrete deck slabs or shallow concrete or steel deck sections. As a consequence, “Model II” is introduced as the most basic section in the investigation, and is called “II” instead of “I” because its depth does not match the reference bridges deck section depths.

The same principal configuration is employed as the section is again formed from a set of inner cells and an outer binding box. The section is composed of a series of trapezoidal or triangular inner box cells surrounded by an outer binding box shaping the final pattern. The section is again assumed to be manufactured using the filament winding process. The top face of the laminated final box girder is again fixed to the “deck slab”, formed from chopped fiber reinforced composites.

Figure 3.3 shows “Model II” dimensioned to approach the width of Second Chang River Bridge deck section. The sharp edges in the inner cells, as well as the outer binding box, would dramatically amplify the stress concentration and hence delamination occurrence. These corners should desirably be rounded during the manufacturing process in order to overcome such a serious problem. If one defines the “laminated panel” (Figure 3.3) as any “double-laminate” formed from two inner laminates or inner and outer laminates joined together and bounded by two double-laminates at different angles, then the deck section can be looked as if it is formed from horizontal, vertical and lateral panels. As the cable-stayed bridge’s central span increases, the compression stresses in the deck are increased in the pylon zone. Hence buckling of thin laminated panels becomes more probable. As a result, it is imperative to reduce the inner cells dimensions in either the horizontal or the vertical direction.

The section is more aerodynamically smooth with less mass and less torsional stiffness than that of “Model I”. Accordingly, much less vortex-shedding tribulations are expected. However, from the research results, the “Model II” section has less critical flutter velocities for all bridge spans. This could be due to its low ratio of torsional to vertical

bending natural frequencies than those of the other models. On the other hand, the anisotropic nature of the laminated box girder plays a major role in the three dimensional deformation of the bridge system. Hence, the micro/macro level design of the lamina and the laminates could excessively control the natural frequencies, vertical or torsional, coupling or separation of different modes of deck oscillations.

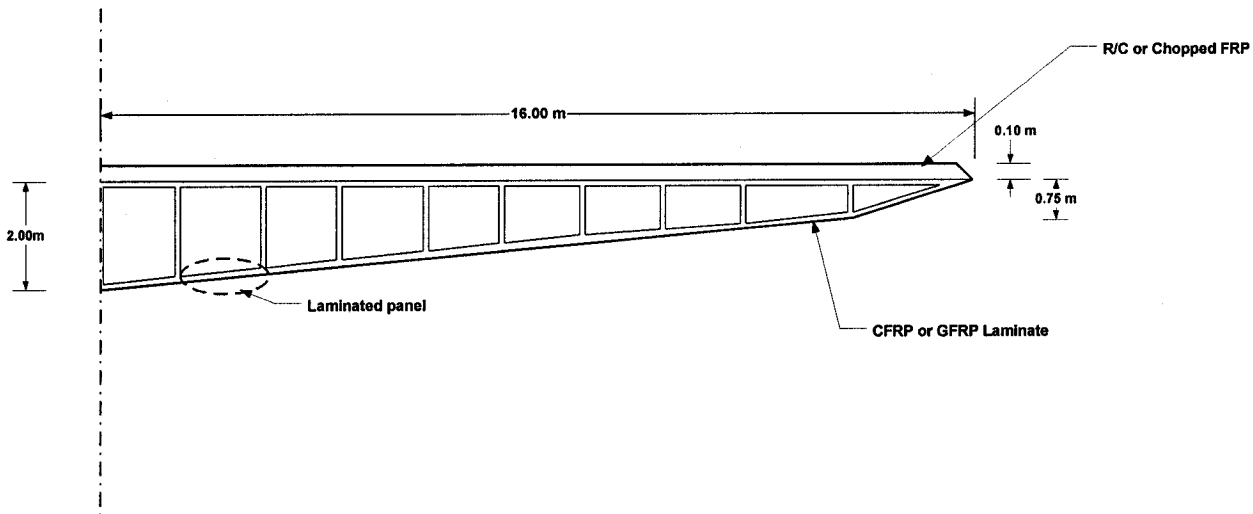


Figure 3.3 Model II with dimensions approach Second Chang River Cable-Stayed Bridge.

### 3.2.3 Model III:

In order to eliminate the corners' stress concentrations in the inner cells and outer binding boxes of models I and II, a tubular cell configuration, Figure 3.4, is introduced for "Model III". The inner cell sections are of semi-elliptical shape with upper and lower horizontal sides connected by two semi-circular sides in the vertical direction. The outer boundary-dimensions of this model are the same as those of "Model I", except the corners at the two lower edges are rounded. The section is again composed of a series of inner cells surrounded by an outer binding box shaping the final pattern. The section is also assumed to be manufactured using the filament winding process. The top face of the

laminated box girder is continuously attached to a “deck slab” formed from chopped fiber reinforced composites.

The structural analysis results related to this section confirm high torsional stiffness and much less risk of stress concentrations, delamination or buckling, except the rounded part of the bottom outer laminate, where the stress state is sensitive. The contact between any two contiguous cellular tubes could reduce the torsional stiffness but it less affects the longitudinal bending stiffness. Hence, it is assumed that any two adjacent vertical laminates have a common tangential line in the longitudinal bridge direction. Theoretical and experimental investigation on a simple span bridge using a similar arrangement showed that the system deformation is considerably reduced when the round sides of the inner tubes are joined using adhesive [Aref & Parsons 2000]. It may be also possible to improve the static and dynamic behavior of the model by filling the space between the circular sides of the inner cells by a chopped fiber reinforced polymer, with less fiber content than the material used for the deck slab, however this case is not studied in this research. The bridge system structural behavior under static load cases and its natural frequencies are highly related to the laminate lay up and thickness. More detailed discussions are presented in Chapter 6.

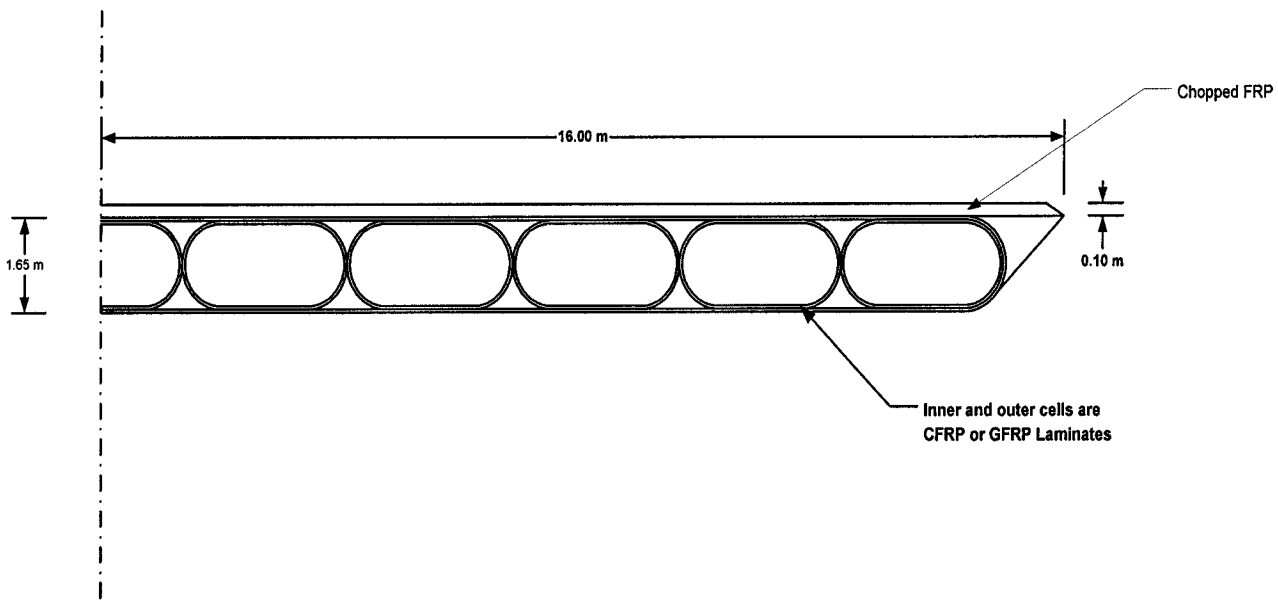


Figure 3.4 Model III dimensioned for a bridge in the same dimensions of Second Chang River Cable-Stayed Bridge.

### **3.2.4 Model IV:**

In order to increase the torsional stiffness of the deck section, reduce the use of the laminated FRP materials and increase the volume of the chopped FRP material, “Model IV” is introduced. It has a completely different configuration from the preceding three models. For economical design, the use of laminated GFRP material is limited to the two longitudinal tubular shells at both ends of the section and the stiffening GFRP laminated sheets for the slab-grid system. The two sides of the bridge deck in this model serve as huge girders supporting the intermediate slab-girder system. The isotropic nature of the dominant material, that is the chopped FRP, would simplify the analysis process.

The outer boundary of the cross-section does not completely match those of the reference bridges and hence the aerodynamic behavior is expected to be different. Wind tunnel section tests are important to evaluate the aerodynamic characteristics of this model, such as the flutter derivatives and the aerodynamic admittance function, which are needed for the aerodynamic stability analysis. The section is expected to be less aerodynamically efficient in low service wind speeds. Much more vortex shedding tribulations are expected than the first three models because of the rather big two sided “girders” and the relatively narrow intermediate part.

The section of this model has very high torsional stiffness and similarly extremely high torsional natural frequencies. The coupling of the vertical and torsional modes due to the anisotropy, which are expected in the first three models, vanishes for this model. “Model IV” is expected to have a higher flutter speed as the natural vertical, lateral and torsional modes are separated and far apart from each other. However the vortex shedding phenomena is of concern.

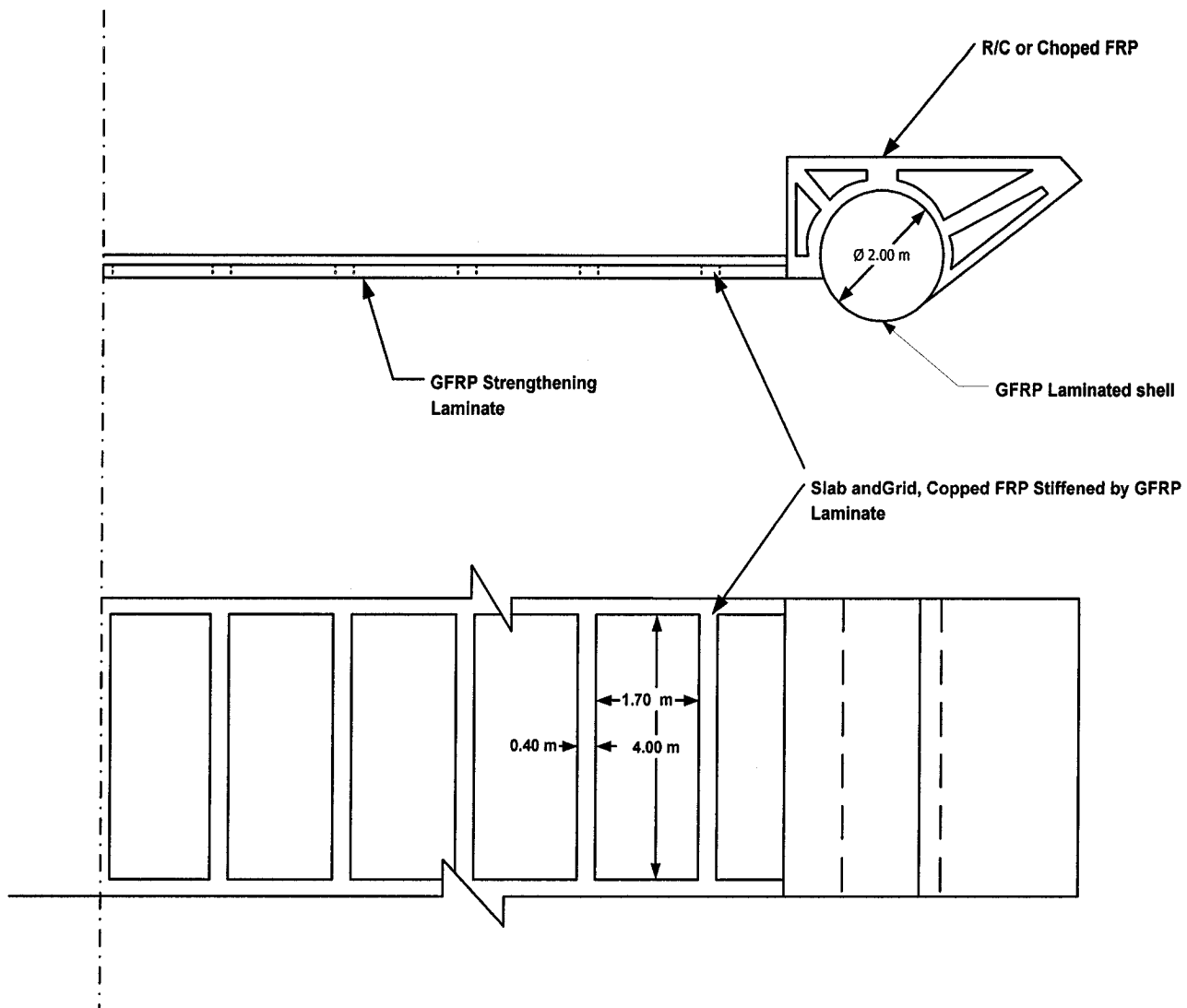


Figure 3.5 Model IV dimensioned for a bridge in the same dimensions of Second Chang River Cable-Stayed Bridge.

### 3.3 Stay Cables:

Carbon fiber reinforced polymer cables are proposed as an alternative to the traditional high strength steel cables for all the four models mentioned earlier. The mechanical properties used for those cable are the same as those given by BBR Ltd in collaboration with EMPA (Swiss Federal Laboratories for Materials and Testing) [Meier, 1996, and BBR review, 11-1996]. The stays have CFRP cables consisting of parallel wire bundles,

each having many wires, 5mm diameter each (as shown in Figure 3.5), containing about 24,000 single carbon fibers. The total sectional area of a CFRP wire consists of 70% carbon fibers and a 30% epoxy matrix. The mechanical properties of these cables are linear elastic isotropic with strength much higher than high strength steel and a modulus of elasticity slightly less than that of the steel (165 GPa versus 200 GPa for steel) and a similar Poisson's ratio of 0.29. However, for a cable horizontal projection length of 400 m and stress level ranging between 10 to 20% of the material strength, the apparent or Ernst Modulus of CFRP is much higher and less sensitive to the applied tensile stress than that of the steel (162.8-164.7 GPa versus 49.3 – 144.8 GPa for steel). This is primarily due to the CFRP cables lightweight per unit length, which is about one sixth of the steel ( $13.5 \text{ kN/m}^3$  versus  $75 \text{ kN/m}^3$  for the steel), while the strength is much higher than the steel (3100 MPa for CFRP versus 1000 – 1870 MPa for the steel). For fatigue and strength considerations, the limit of the maximum permissible stress in general is 45% of the strength for steel cables but it can exceed 90% for CFRP cables [Walter, 1999, and BBR Ltd., 1996]. However the relaxation and creep strains of CFRP Cables are higher and need to be adjusted repeatedly in the construction to reach the design tensile stress levels, [Wernli et al, 1999]. Figure 3.5 shows the FRP cables and Figure 3.6 shows their anchorage system developed by BBR Ltd. and EMPA.

The use of CFRP cables is anticipated to greatly improve the cable-stayed bridge performance in different ways. First, their very low weight would reduce sagging and consequently increase their stiffness, which means that the Ernst modulus is higher than that of traditional steel cables. Second, the very high strength, which is almost three times the steel cable strength, enables one to apply a higher initial pre-tension force taking into account controlling the strain by successive adjustment of the cable length. This could further reduce the required cable diameter and the initial sagging and reduce the effect of the geometric non-linearity. Third, the natural frequencies for both the vertical and torsional modes of similar configuration cable-stayed bridges are expected to be higher than those with steel cables. The expected cable vibration is also less due to the high stiffness to weight ratio and the less curvature under gravity loads.

## Manufacturing of the wires and technical data:

### General properties of CFRP:

- high corrosion resistance
- high tensile strength or high elastic modulus
- outstanding fatigue performance
- high relative equivalent modulus
- lightweight
- low thermal expansion

### Data of selected carbon fiber cables:

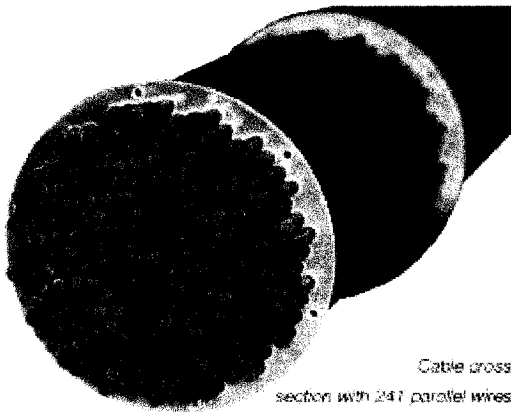
- wire diameter: 5 mm
- tensile strength of a wire: 3100 MPa
- modulus of elasticity: 165 GPa
- fiber volume fraction: 68 %
- nominal breaking load cable: 94 % of the sum of the single wires
- matrix: epoxy based

| number of parallel wires | nominal breaking load [kN] |
|--------------------------|----------------------------|
| 1                        | 60                         |
| 19                       | 1'070                      |
| 91                       | 5'100                      |
| 241                      | 13'600                     |

Each wire is made from 400'000 single filaments



Figure 3.6-a CFRP wires for cable stayed bridge



Cable cross section with 241 parallel wires



Figure 3.6-b CFRP Cable for cable stayed bridge (BBR Ltd, 1996)



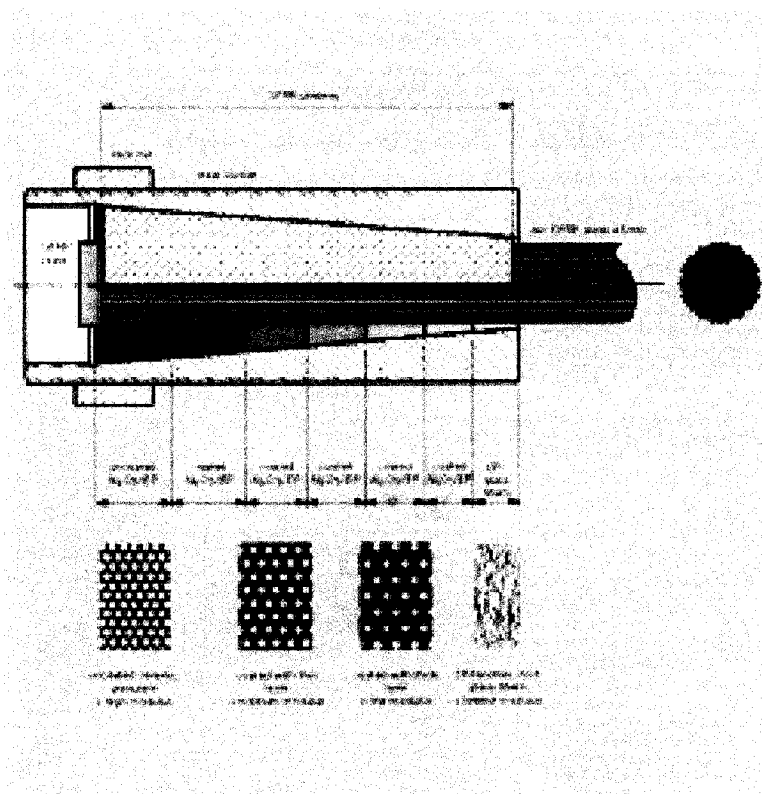


Figure 3.6.c Anchorage system for a CFRP Cable, (after Meier 1996).

### 3.4 Pylons:

The pylon stiffness has an important effect on the deformations and the level of stresses in the cable stayed bridge deck, pylons and the level of tensile stress in the cable stays. Most of the parametric studies on cable stayed bridges focus on the effect of the system nonlinearity and analysis methods on the results accuracy, or optimization of the cable diameter or the stay arrangements (harp, semi-harp or fan) or the deck internal dimensions and the resulting changes to the distribution and magnitude of deck, cable and pylon stresses. Walther et al [1999] performed a comprehensive parametric study using a second order two-dimensional analysis of cable-stayed bridges with different stay arrangements under ultimate static loads. They showed that the effect is significant when the pylon stiffness is very low, i.e., the stiffness that can only support the cable-stayed bridge self loads, and the maximum superimposed static and traffic loads, without buckling. As the pylon stiffness increases, sudden reduction in the deck deformations and stresses, and slight increase of the pylon

stresses are reported. The effect of the pylon stiffness on the bridge natural frequencies is not reported in the literature.

As the pylon is connected to the deck by the cable stays, the cable stays stiffness together with the tensile stress levels in the cables are important factors affecting the pylon stiffness influence on the deck deformations and stresses. Hence, and referring to (section 3.3), it is apparent that the cables material mechanical properties and their weight per unit length have significant effect on their apparent modulus and therefore their transferring-efficiency of the pylon stiffness to the bridge deck. CFRP cables are much more efficient and hence the elastic support given by the cables are more capable of controlling the deformation of the deck than traditional steel cables. The combined effect and the interaction between the pylon stiffness and stay cables stiffness and the cable tensile stress level on the bridge system performance are also not reported in the literature. Consequently, optimization of the pylon stiffness together with the cable diameter and the internal dimensions of the FRP laminated box girder seem to be a very complicated process in the proposed three dimensional modeling procedure and is considered to be out of the scope of this research and has to be addressed as an important step for economical and high performance hybrid cable stayed system in the future. In this research, the pylons of the proposed models are assumed to be reinforced concrete. Three alternatives are to be investigated, Stiff, Medium Stiff ( $2/3$  of Stiff) and flexible ( $1/3$  of Stiff) pylons of the same Vertical dimensions but of different depths in the direction of the traffic. A “Stiff” pylon is defined in the present investigation according to a very simple criterion; that is the maximum displacement at the top of the pylon under the effect of the self loads and any of the combinations and load cases of the superimposed dead and traffic loads should not exceeds the pylon height above the deck top surface divided by 500. In the performance study of this research, this criterion is applied with the worst self-load case related with the heaviest deck sections and medium examined cable diameter, without pretension. The dimensions of the pylon that performs this criterion for any of the tested bridge models and any of the tested span lengths is stated to be the “stiff pylon” for that model and span length. An example is shown in (Figure 3.7 a, and b) dimensioned to support a bridge of similar geometrical shape as the Second Chang River Bridge.

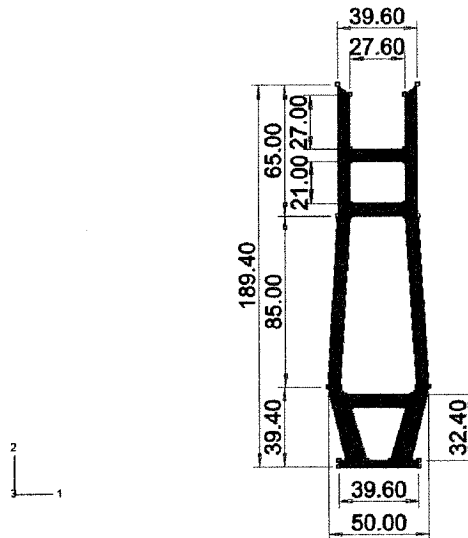


Figure 3.7-a Vertical section of a Pylon dimensioned for The Second Chang River Cable-Stayed Bridge

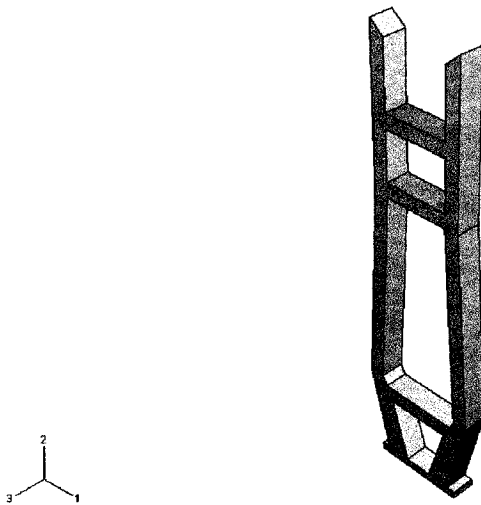


Figure 3.7-b Stiff Pylons dimensioned for The Second Chang River Bridge.

### 3.5 Reference Bridges:

Three of the longest span cable stayed bridges are employed as general reference bridges. These bridges are the Annacis Island Bridge (also called the Alex Fraser Bridge) of Vancouver, Canada; with a central span of 465 m and opened to traffic in 1986, the Second Chang River Bridge, China; with a central span of 628 m and opened to traffic in 2001, and the Tatara Bridge, Japan; the longest cable-stayed bridge in the world with a

central span of 890 m opened to traffic in 1999. Brief descriptions of these three bridge structural characteristics are as follows:

### **3.5.1 The Annacis Island Bridge:**

Annacis Island Bridge had two alternative designs that were completely evaluated; a composite deck (steel-concrete), which is the one that was constructed, and the second a concrete box girder (Figure 3.8). The bridge has a central span of 465m and two side spans of 240.25 m, with a distance of 45 m close to the bridge end of each side not connected to the stay cables. The bridge structure and supports are symmetrical on the bridge centre in the longitudinal direction. The deck is simply supported from either direction at 0 m and 45 m (Figure 3.8) and also simply supported on both pylons. The cable stays are arranged in a semi-harp configuration with pylon height of 102.1 m above the deck top level and a total height of 142.3 m from the base. The pylons are opened from the top in the planes perpendicular to the traffic direction, so as to allow the two “columns” of each pylon to hold the stay cables in one vertical plane.

### **3.5.2 The Second Chang River Bridge:**

Due to the impact on long span cable stayed bridges design and construction, the general outline of the Tatara Bridge has been used recently as a standard configuration for long span cable stayed bridges. The Second Change River Bridge is a resized version of the Tatara original design, with some modifications to the ratio of the main to side spans and the deck material. The bridge has a central span of 628m and two side spans of 305 m. The bridge structure and supports are symmetrical on the bridge centre in the longitudinal direction. The deck is simply supported from either direction at 0 m and 58.5 m (Figure 3.9) and also simply supported on both pylons. The cable stays are arranged in a semi-harp configuration of 20 cables each, with pylon height of 160.3 m above the deck top level and a total height of 195.4 m from the base. The pylons are closed from the top in the planes perpendicular to the traffic direction, so as the cables of each stay do not lie in one vertical plane.

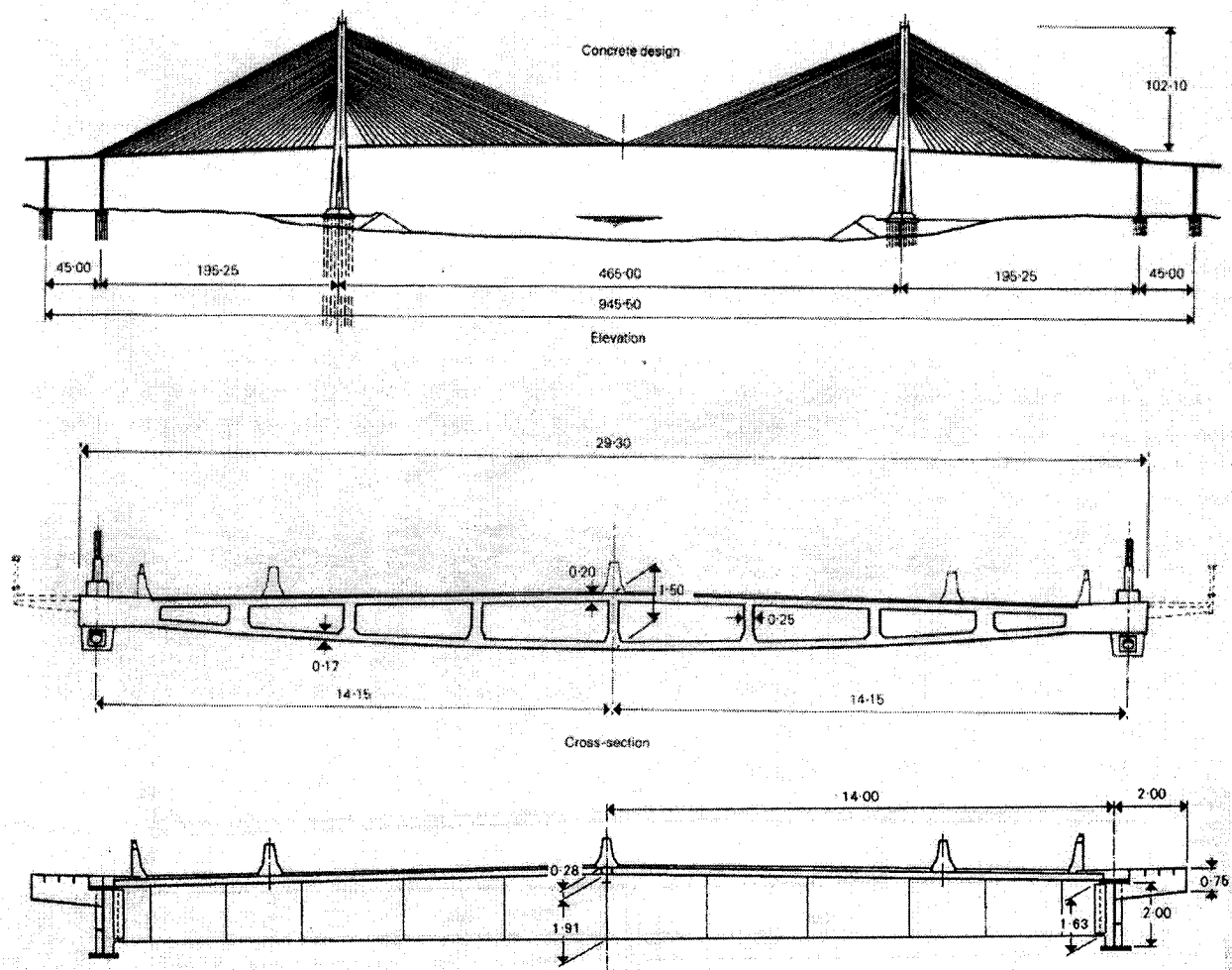


Figure 3.8 Annacis Island Cable Stayed Bridge, Elevation, Concrete Deck Section Design and Steel Section Design alternatives. [Walter et al 1999]

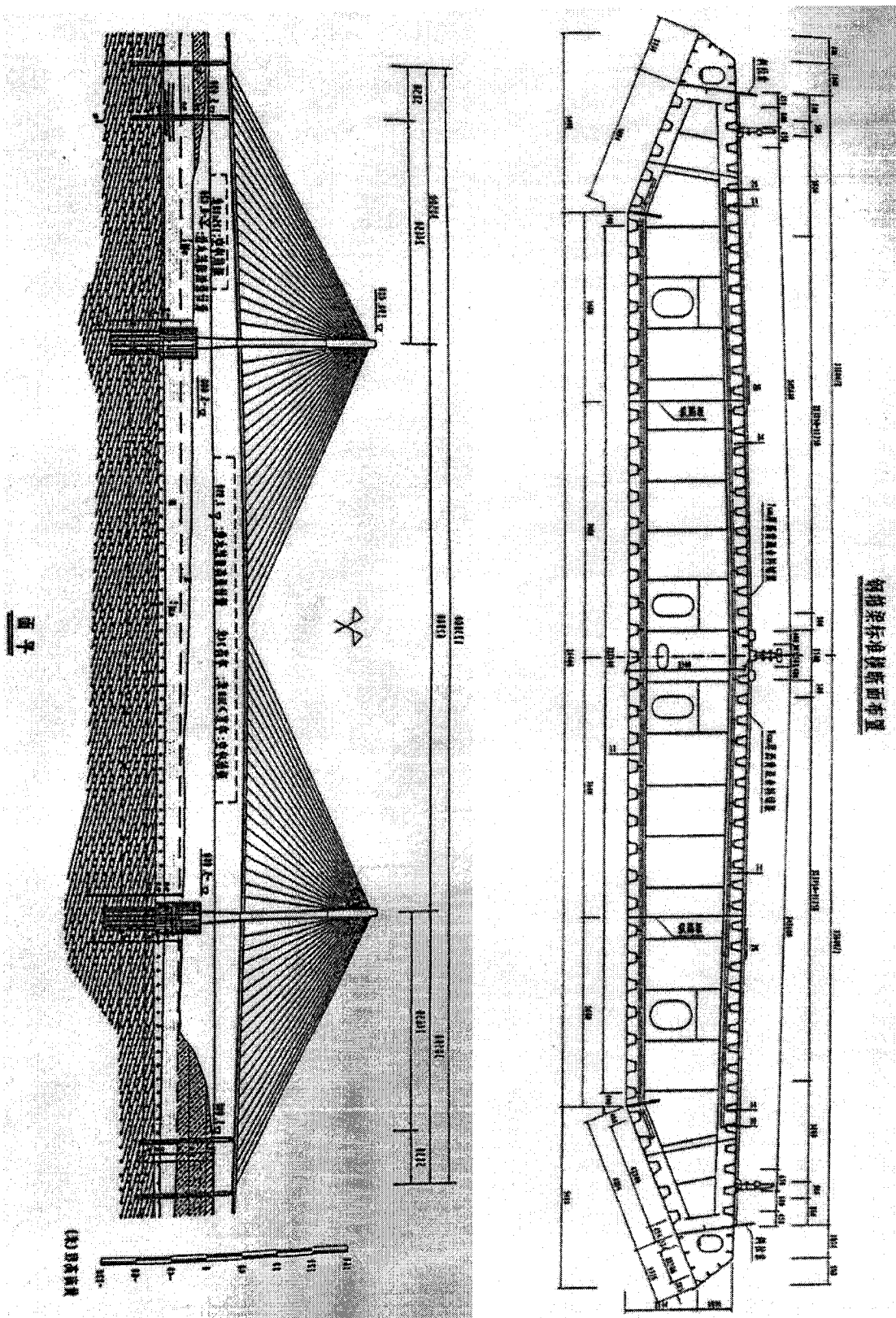


Figure 3.9 Nanjung Second Chang Bridge, China, Elevation and deck section, (Tongji University, Section Model Wind Tunnel Test Report 1999)

### **3.5.3 Tatara Bridge:**

The Tatara Bridge is the currently world longest cable stayed bridge, with a hybrid deck of steel section for the central span and prestressed concrete for the side spans [Yabuno 2003]. The bridge has a central span of 890m and two side spans of 320 m and 270m. The bridge structure and supports are unsymmetrical on the bridge centre in the longitudinal direction. The deck is simply supported for the longer side span (320 m) at 0 m and 50 m and 100m measured from the bridge end, and the shorter side span is supported at 0 m and 50 m measured from the bridge end (Figure 3.10), and the deck is also simply supported on both pylons. The cable stays are arranged in a semi-harp configuration of 21 cables each, with pylon height of 180.6 m above the deck top level and a total height of 220 m from the base. The pylons are closed from the top in the planes perpendicular to the traffic direction, so the cables of each stay do not lie in one vertical plane. Figure 3.10 shows some design details of the Tatara Bridge.

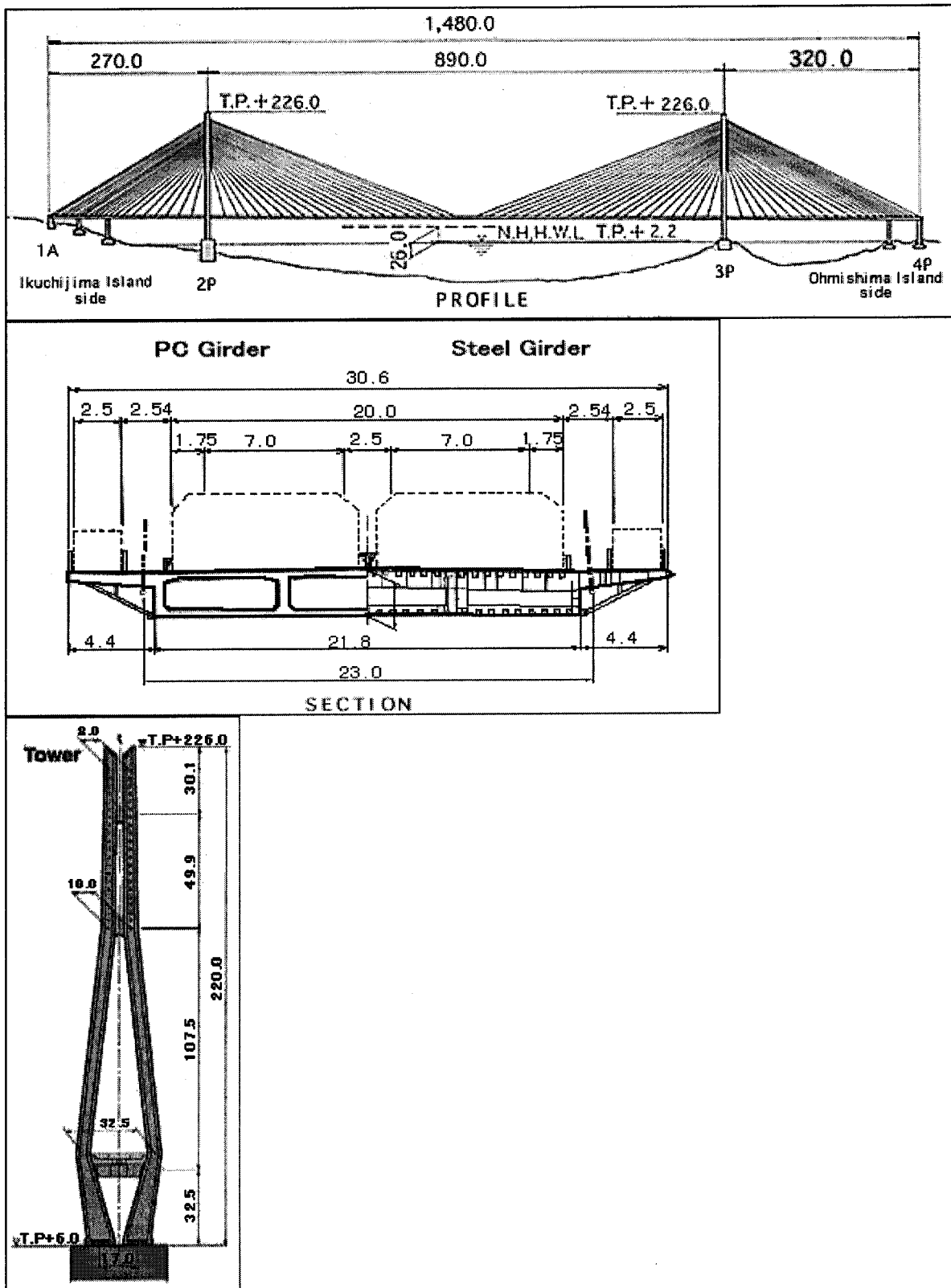


Figure 3.10 Tatara Bridge, Japan, Elevation and deck section [Yabuno et al, 2003].



### **3.6 General Lay-Out of the Proposed Cable-Stayed System:**

The proposed hybrid cable-stayed bridge systems have the same general geometrical shapes as the three reference bridges (section 3.5). For comparison and prediction purposes, minor changes have been made to accommodate the differences in the design of the three reference bridges in an attempt to unify their general geometrical proportions in the proposed bridge system. The details of the vertical plane of the proposed pylon is the same as the Tatara Bridge with the exception of separating the top “columns” in order to place each cable set in one plane normal to the deck surface. This would simplify the bridge geometric design, and consequently the finite element model. In addition, such a change in the pylon has a negligible effect on the deck static and aerodynamic behavior; however the aerodynamic characteristics of the pylon itself, which is out of the scope of this research, are affected more and should be studied in future research.

The proposed deck section models are sized to match the boundary dimensions for each of the three reference bridges. The deck longitudinal dimensions and supports (boundary conditions) of the proposed hybrid cable-stayed system are exactly the same for Annacis Island and the Second Chang Bridges equivalent systems. However, a slight change to the configuration of the hybrid cable stayed system that proposed to match Tatara Bridge is made, that is the side spans are both made equal to 320m, with only two simple supports at 0m and 150m from either bridge sides. This would make the three bridge systems comparable and have fewer differences in their geometric characteristics and configurations, and hence can be compared directly with each other. Figures 3.11 -3.13 show the proposed hybrid system resized to match the three reference bridges.

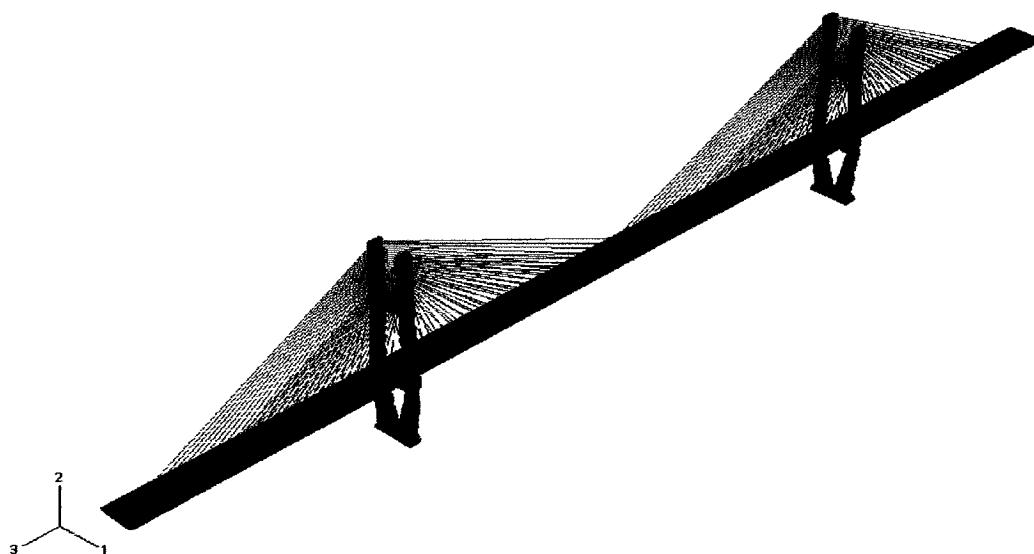


Figure 3.11 General lay out of Model I dimensioned to match Annacis Island Cable-Stayed Bridge (465 m central span).

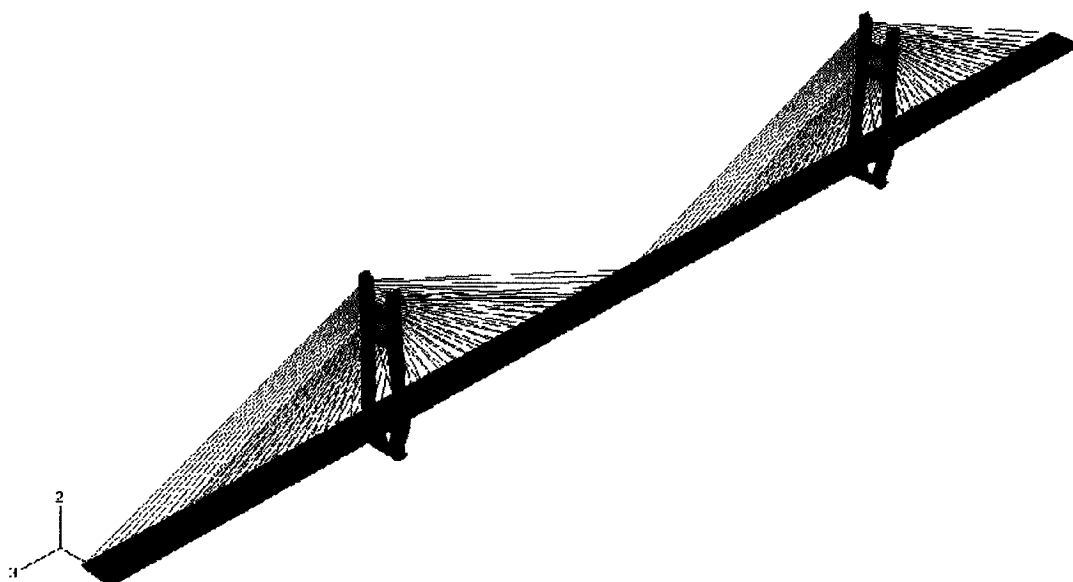
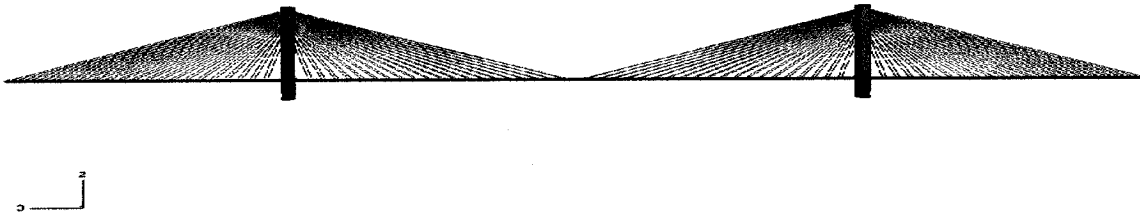
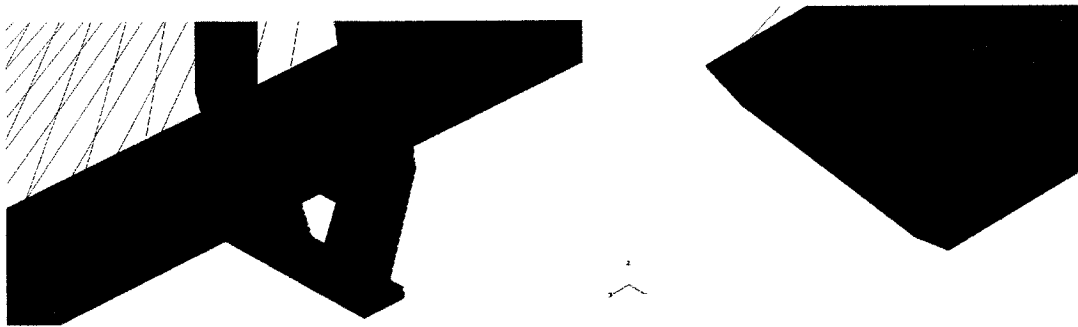


Figure 3.12-a General lay out of Model I dimensioned to match The Second Chang River Cable-Stayed Bridge (628 m central span).



3.12-b Longitudinal projection of Model I-The Second Chang River Cable-Stayed Bridge.



3.12-c Some details of Model I- The Second Chang River Cable-Stayed Bridge.

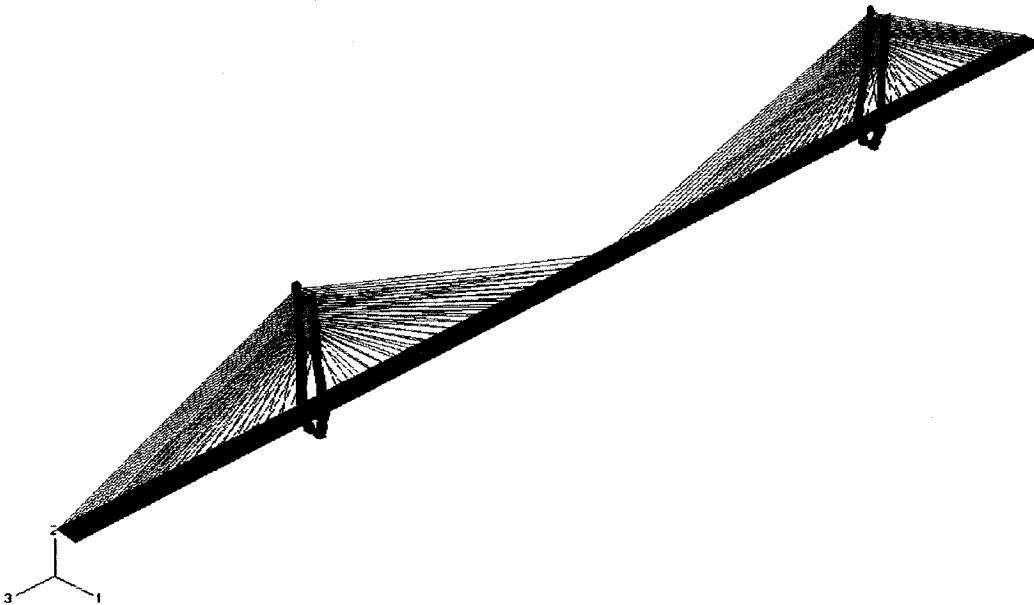


Figure 3.13 General lay out of Model IV dimensioned to match Tatara Cable-Stayed Bridge  
(890 m central span)

### **3.7 Development of the Analysis-Design Procedure:**

#### **3.7.1 Introduction:**

In addition to the necessity for including the changes in structural behavior of the hybrid cable-stayed system, due to the use of advanced composite deck sections together with CFRP cables in the analysis-design process, it is also required to define clearly the ultimate and serviceability limit states for such a new structural system. The analysis and design procedure for the new proposed hybrid cable-stayed bridge includes the micro/macro-level design of the chopped FRP, the laminas and laminates, a thorough evaluation of their resulting properties through a homogenization process, and the estimation of their equivalent characteristics in the structural level. The next stage is the numerical simulation of the proposed structural system in three dimensions using a non-linear anisotropic finite element model, which results in consistent representation of both the material and the geometrical characteristics.

As the laminated FRP material involved in the proposed deck sections has anisotropic properties, the procedure should include proper material failure criteria that have to be examined in different stages of the procedure. The serviceability requirements include examination of whether the proposed bridge system models do match acceptable deformation and vibration limits under extreme service loading situations. Finally, the proposed system is a long span cable-stayed bridge and the aerodynamic stability has to be assured in the analysis-design procedure and this step position in the course of action has to be defined.

On the other hand, an optimization process of the bridge system detailed dimensions and materials design is to be integrated with the proposed analysis and design procedure. This involves: (i) the material properties of the fibers and matrix and the fiber fraction in the chopped FRP material, (ii) the micro/macro level design of the composite laminas and laminate, (iii) the section internal dimensions in the structure level, (iv) the stay cables configurations (assumed to be the same as those of the reference bridges throughout this research), cable diameter, level of pretension at each cable and the cable anchorage. The multi-level optimization process is bounded by: (i) the required ultimate static load capacity of the bridge system, and the allowable deformations and vibration under service loads, (ii) separation and divergence of the torsional from the bending natural frequencies

and avoiding any coupling of different oscillation modes. Hence, achieving the optimum hybrid system design and performance is a very complicated task; however it could lead to a long span bridge system with better structural and aerodynamic characteristics than the existing bridge systems (using traditional materials). For example, the coupling of the oscillation modes of the traditional very-long-span cable stayed bridges could be reduced for the case of the proposed bridges through suitable composite material micro/macro design and deck cross-section configuration. The following sections show, in brief, the steps of the analysis-design procedure for the proposed hybrid cable stayed bridge system.

### **3.7.2 Micro/Macro Level Design and Evaluation of Advanced Composite Materials:**

At this step in the analysis-design procedure, the micro/macro design of the material is determined. This could include deciding on the types of fiber and matrix to be used for each lamina, the volumetric ratio of the fiber, lamina thickness, orientation of the fiber in the lamina, the periodicity or randomness of the fiber in the micro-scale structure of the advanced composites, sequence of the laminae in the laminate and the resin used. The final decision is an output of a complicated interactive process the boundary of which is the anticipated characteristics of the bridge structural systems and their performance under extreme loading conditions. Although following general material and sectional design regulations adopted from experience with traditional cable-stayed bridge systems is supposed to simplify and accelerate the analysis-design procedure, it does not guarantee that the structural system will pass all the limit states and the aerodynamic stability requirements. Unfortunately, such considerations are apparently oversimplified analogies and are incapable of dealing with the complicated behavior of the proposed system.

Two micro-mechanical approaches have been used in this research for the simulation and use of advanced material at the micro-scale level, depending on the nature of the material in the micro level. Both methods represent the material properties in the micro-level and transfer them to the macro level. Consequently, the individual properties of the advanced composites components and the random or the periodic nature of their existence are

characterized. The engineering properties in the macro-level are the output of this important step in the analysis design procedure that leads to homogenization of the material properties in the micro-level to the macro-level. Then, with these resulting properties, the basic assumptions of the mechanics of homogenous materials are valid and a structure built using advanced composite materials can be analyzed using continuum mechanics, without unnecessary and unrealistic over-simplification.

The first approach is simulating the chopped FRP material with random orientation of relatively short fibers using the stochastic finite element method. In this approach, the random existence of the fiber in composite materials is dependent on the element (or window) size and the fiber volumetric fraction in the meso-window is assumed to follow a natural distribution (Chapter 4). The meso-window represents a transition window between the micro-scale and the macro scale where the material properties of the composite components can be averaged. The micro scale is the scale where the material is treated as heterogeneous and only one of the composite components (or one phase; fiber or matrix) are represented as a unique phase in the element, and the macro scale is the scale where the material is treated as homogeneous, and the composite material properties are the homogenized properties of its components. Hence the lower limit of the meso-window is the micro-scale and its upper limit is the macro-scale, and its dimensions are decided by those two limits, which are related to the inclusion (fiber) size. After generating the material properties through a random process, the method proceeds to perform plane stress analysis to evaluate the material mechanical properties. The method can also simulate the more complicated case of multi-fiber chopped FRP involving a multi-random process.

On the other hand, and for the simulation of the laminated box girder part of the deck, which assumes an automated construction process, it is more practical to use an advanced composite of periodic nature in its micro-scale structure. Hence a second analytical approach has been adopted to evaluate the lamina material properties. This approach relates the stiffness and strength properties of the laminated advanced composite material to the basic general properties of its components, the fiber and the matrix, and the volumetric percentage of the fiber in the composite. Both approaches are presented in Chapter 4.

### **3.7.3 Design and Modeling the Bridge System in Structural Level:**

In this step the proposed hybrid cable stayed bridge system is simulated using three dimensional non-linear finite element modeling. The input for this step is the detailed three-dimensional geometry of the bridge system components and configurations, and the material properties resulting from the micro/macro design and evaluation step. The outputs are the deformations, strains, stresses and failure criteria distributions for every component of the structure.

Many researchers [e.g. Jones, 1999, and Reddy, 2004] showed that the stiffness of advanced composite material is coupled to the geometry of the structure. The deformation response of an anisotropic material even to simple stress states, can literally be in every direction and every plane. It has also been proved that orthotropic materials could exhibit apparent anisotropy when stressed in non-principal material coordinates, which is the stress state of laminas of the deck. Hence, accurate simulation of the composite structure geometry in three dimensions is a necessity although it is a very expensive and long process. Consequently, it is not possible to simplify the deck using an equivalent simple structural system. For instance, the cable-stayed bridge deck designed using the traditional materials are frequently modeled by an equivalent grid or orthotropic plate. In this research, the deck section geometry is simulated exactly in all its details. This step on its own is very complicated, notwithstanding that the structure dimensions are so huge for all the four section models and the three reference bridges.

The material properties resulting from the micro/macro homogenization process are of two types: (i) averaged linear isotropic for the random oriented FRP (or the chopped FRP), (ii) orthotropic material properties of the periodic FRP (of the lamina). The cables and the pylon materials are also linear elastic materials in the present research modeling.

In the finite element modeling, the hybrid deck is discretized using isoparametric laminated shell elements for the first three models (i.e. Models I, II, III), while Model IV is discretized using the same shell elements for the two-sides advanced-composite laminated tubular parts and the stiffening laminate for the mid slab-grid part, and a solid isoparametric quadrilateral element for other parts of the section. Furthermore, the cables are represented using general isoparametric beam elements so that all the properties of

the cables are characterized. The pylons are also represented using solid isoparametric quadrilateral elements.

In order to optimize the finite element mesh, in the discretization process a deformation convergence criterion has been used. That is, the difference between the maximum deck deflections of two successive refined mesh arrangements should not exceed  $10^{-5}$ , provided that the distributions of the deformations of different bridge components have no apparent changes.

As the bridge systems, particularly the deck and cables are expected to have high deformation; the geometric non-linearity is taken into account in the finite element model. It is also noted that the material anisotropy generates another type of non-linearity due to anisotropic material properties, usually called anisotropic-material-nonlinearity. In addition to that, it is also important to note the unsymmetrical nature of the deformations and stresses resulting from any symmetrical loading applied on the proposed symmetrical bridge models. Indeed, the loading in any plane or direction on the system generates deformations in all directions, as the deformations of the presented anisotropic system cannot be related to the direction or the plane of the externally applied stresses. The ABAQUS advanced finite element program has been employed in the finite element modeling.

#### **3.7.4 Material Failure Criterion:**

Different materials are used in the proposed system and have to be examined for any possible failure under extreme loading conditions. Two major types of materials are involved in the system, isotropic or anisotropic materials homogenized to isotropic, and anisotropic materials homogenized to orthotropic. Since the failure states of isotropic materials are understood and well defined, the concentration here is on the orthotropic material failure criteria. Indeed, this critical and very important step is also internally included in the next analysis-design steps (i.e. matching the reference bridges properties and ultimate and serviceability limit states), as the material failure is a break point in any structural design.



As a result of the micro/macro homogenization process, the laminas of uni-directional fibers that are used in the laminate design have orthotropic mechanical properties. Hence using related failure criteria is essential since the failure criteria of isotropic materials are not applicable. The material failure criteria used in this research are based on the stress failure theories. Two stress-based failure theories are followed, the maximum stress theory and Tsai-Hill theory. Denoting orthotropic material directions by 1 and 2, with the 1-material direction aligned with the fibers and the 2-material direction transverse to the fibers. The input data for both stress-based failure theories are tensile and compressive stress limits,  $X_t$  and  $X_c$  in 1-direction; tensile and compressive stress limits,  $Y_t$  and  $Y_c$ , in 2-direction; and shear strength (maximum shear stress),  $S$  in the  $X - Y$  plane.

#### 3.7.4.1 Maximum Stress Theory:

In this criterion, each and every one of the stresses in the principal material coordinates must be less than the respective strengths; otherwise failure is said to have occurred. That is for tensile and compressive stresses:

$$\begin{aligned} \sigma_{11} &< X_t, \quad \sigma_{22} < Y_t \\ \sigma_{11} &> X_c, \quad \sigma_{22} > Y_c \end{aligned} \quad (3-1)$$

For shear;

$$|\sigma_{12}| < S \quad (3-2)$$

If any one of the above inequalities is not satisfied then the assumption is made that the material has failed by failure mechanisms associated with  $X_t, X_c, Y_t, Y_c, \text{ or } S$  respectively.

#### 3.7.4.2 Tsai-Hill Failure Criterion:

Hill [1950], proposed a yield criterion for orthotropic materials as an extension to von Mises' yield criterion:

$$\begin{aligned} (G + H)\sigma_{11}^2 + (F + H)\sigma_{22}^2 + (F + G)\sigma_{33}^2 - 2H\sigma_{11}\sigma_{22} - 2G\sigma_{11}\sigma_{33} - 2F\sigma_{22}\sigma_{33} + \\ 2L\sigma_{23}^2 + 2M\sigma_{13}^2 - 2N\sigma_{12}^2 = 1 \end{aligned} \quad (3-3)$$

where F,G,H,L,M, and N are failure strengths. Tsai [1968] relates Hill failure strength to the usual failure strengths X, Y and S for a lamina. If only  $\sigma_{12}$  acts on the body, then, because its maximum value is S;

$$2N = \frac{1}{S^2} \quad (3-4-a)$$

Similarly if only  $\sigma_{11}$  acts on the body then;

$$G + H = \frac{1}{X^2} \quad (3-4-b)$$

and if only  $\sigma_{22}$  acts on the body then;

$$F + H = \frac{1}{Y^2} \quad (3-4-c)$$

if the strength in 3-direction is Z and only  $\sigma_{33}$  acts on the body then;

$$F + G = \frac{1}{Z^2} \quad (3-4-d)$$

Then combining equations (3-4) gives the following relations between F,G,H and X,Y,Z:

$$\begin{aligned} 2F &= \frac{1}{Y^2} + \frac{1}{Z^2} - \frac{1}{X^2} \\ 2G &= \frac{1}{X^2} + \frac{1}{Z^2} - \frac{1}{Y^2} \\ 2H &= \frac{1}{X^2} + \frac{1}{Y^2} - \frac{1}{Z^2} \end{aligned} \quad (3-5)$$

For plane stress in 1-2 plane of uni-directional lamina with fiber in (1-direction),  $\sigma_{33} = \sigma_{13} = \sigma_{23} = 0$ . However for the cross section of such lamina, Y=Z from the obvious geometrical symmetry of the material structure, thus, Hill equation (3-3) simplified to

$$\frac{\sigma_{11}^2}{X^2} - \frac{\sigma_{11}\sigma_{22}}{X^2} + \frac{\sigma_{22}^2}{Y^2} + \frac{\sigma_{12}^2}{S^2} = 1.0 \quad (3-6-a)$$

The inequality results from limiting the left hand side of equation (3-6) to (1.0) and is called the Tsai-Hill failure criterion. The left hand side of the criterion is defined as a function and will be referred as Tsai-Hill Failure Function (THFF) for the use in the performance study:

$$\phi = \frac{\sigma_{11}^2}{X^2} - \frac{\sigma_{11}\sigma_{22}}{X^2} + \frac{\sigma_{22}^2}{Y^2} + \frac{\sigma_{12}^2}{S^2} \quad (3-6-b)$$

The appropriate values of  $X_t$  or  $X_c$  and  $Y_t$  or  $Y_c$  must be used depending on the signs of  $\sigma_{11}$  and  $\sigma_{22}$ . Figure 4.3 shows the difference between the two criteria.

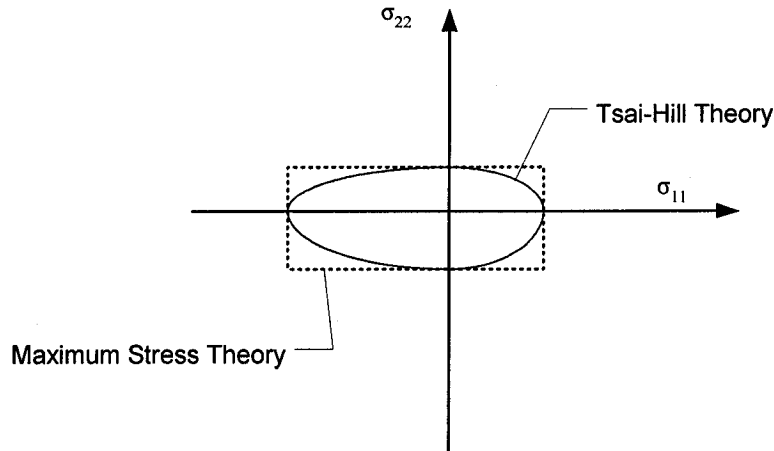


Figure 3.14 Maximum and Tsai-Hill Failure Theories.

### 3.7.5 Adjustment of Cable-Stays Pre-Tension According to the Zero-Deflection Procedure:

Once the finite element model is set for optimum discretization and numerical performance, the investigation of the structural behavior of the bridge system is possible. The first step of this investigation is the adjustment of the pre-tension forces of the stay cables. The procedure used in this research, which is called the zero-deflection cable-stays pre-tension procedure, is mainly dependent on the application of an appropriate pretension stress for every cable. As the proposed bridge system is symmetrical on the longitudinal deck centerline, the pretension cable stress is applied equally to each pair of cables (one from each of the pylon columns) but differently from the next pair of cables and so on. In order to overcome the large deflection in the mid-central span of the bridge, the pretension is increased according to the distance measured on the deck surface from each cable anchorage to the pylon. The major function of this variable pretension is to reduce the deck deflection due to self-weight and dead loads down to zero along the deck longitudinal centerline.

The level of this variable pretension at every cable is a complicated function of the deck weight per unit length, the deck stiffness, stay configuration, the cable stiffness and length, bridge central span, the ratio of the central span to the side spans, the pylon stiffness and height, and the deck supports. For the proposed bridge system, and for each hybrid system of the same general characteristics of each reference bridge, the stay configuration, the ratio of the central span to the side spans, the pylon stiffness and height, and the deck supports can be considered as constants. Hence the cable tension function is dependent on the cable stiffness and length, and the deck stiffness and weight. The stiffness of the advanced composite deck itself is a very complicated function of its material properties, the deck section and the bridge length; also the cable stiffness is a function of its material properties, length and weight. However, the change in the CFRP cable weight is small, and variations in the deck stiffness and weight are functions of the bridge central span length. Hence, the variable cable tension can be simply related to three variables, the central span length, the cable position in the stay (which measures its length and is linearly proportional to the bridge central span length), and the cable axial stiffness (which is a function of the cable length). Therefore, the cable pretension is linearly proportional to the cable modulus and cross sectional area (the cable axial stiffness without the cable length, which is combined with major variable; the central span length), and nonlinearly proportional to the deck central span length and the cable position in the stay. Consequently, the cable tension function can be introduced using a simple nonlinear function of the central span and the cable position in the stay, which is also a linear function of the cable modulus and area. The function can be adjusted to find the degree of nonlinearity by trial and error. For the configurations of the hybrid cable-stayed bridge discussed in this research, it has been found that the following empirical expression for the pre-tension force in any cable perfectly matches the zero-deflection procedure requirement:

$$T_i = E_i \cdot D_i^2 \cdot \frac{i^{\alpha/3}}{L^\alpha} \quad (3-4)$$

Where:

$T_i$  : the tension at cable  $i$ , (N).

- $i$  : cable number starting from the face of the pylon as 1 and the maximum number is for the cable positioned far from the pylon in both sides.
- $E_i$ : the modulus of elasticity of cable( $i$ ) material (Pa), and assumed equal for all cables.
- $D_i$ : cable ( $i$ ) diameter (m), assumed equal for all cables.
- $L$ : central span of the bridge (m).
- $\alpha$ : a constant depend on the bridge configuration, central span length, deck stiffness and weight and pylon height, (e.g.  $\alpha = 1.575$  for 2<sup>nd</sup> Chang River Bridge).

Figure 3-15 shows a three-dimensional view of Model I dimensioned for the second Chang River cable stayed bridge deformed under self and dead loads. Figure 3-16 shows the same model after the proper pre-tension forces are applied. Figure 3-17 shows the deflection of the bridge centerline at the surface level of the deck slab for the two cases shown in Figures 3-15 and 3-16.

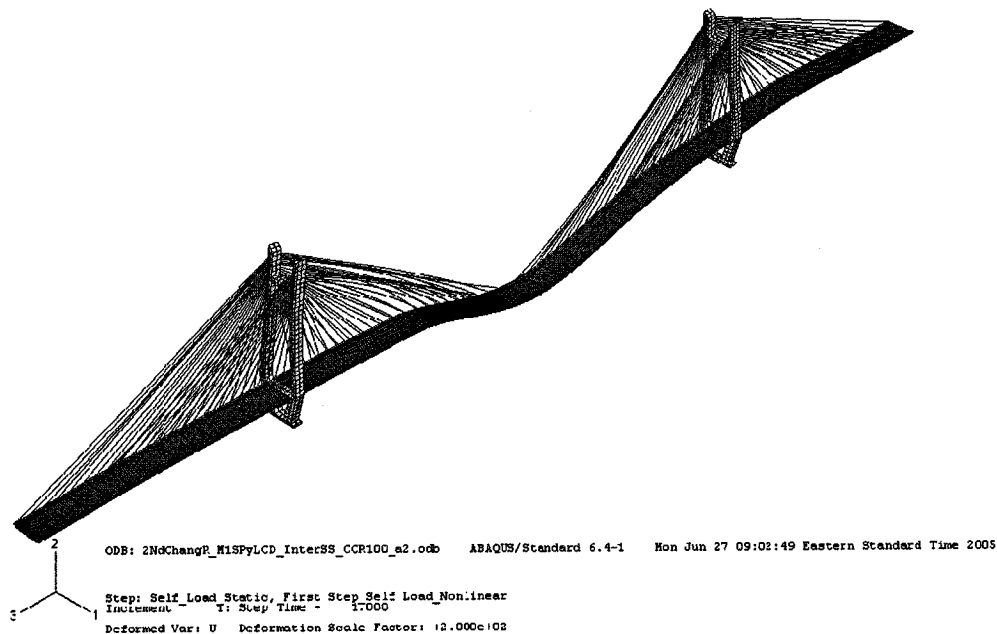


Figure 3-15 3D view of Model I-2<sup>nd</sup> Chang River Bridge deformed non-linearly under the self and dead loads

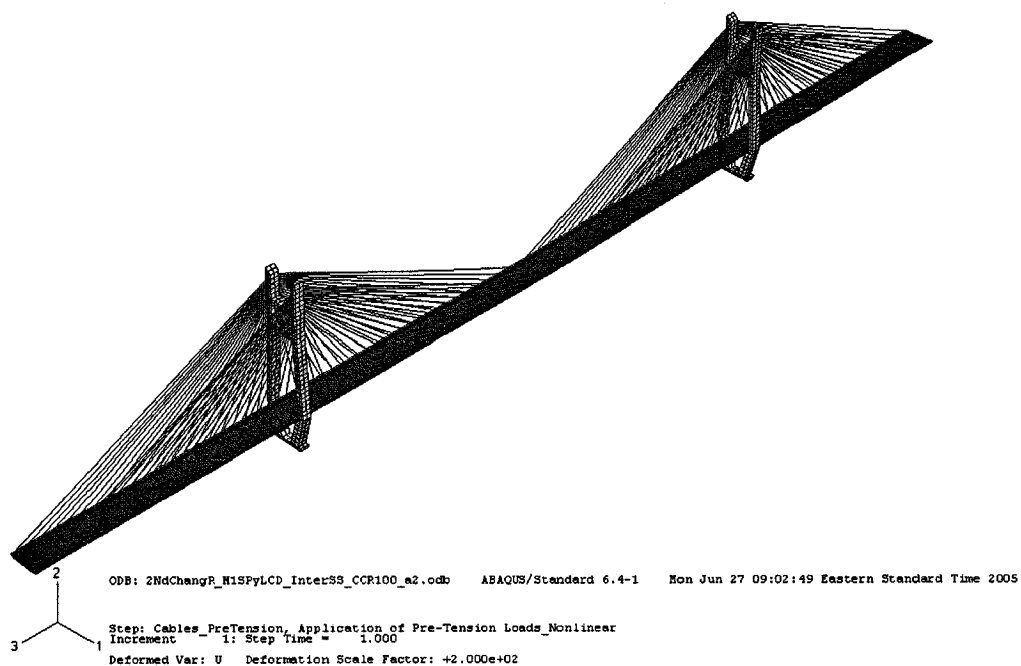


Figure 3-16; 3D view of Model I-2<sup>nd</sup> Chang River Bridge deformed non-linearly under the self and dead loads and pretension in the cable-stays

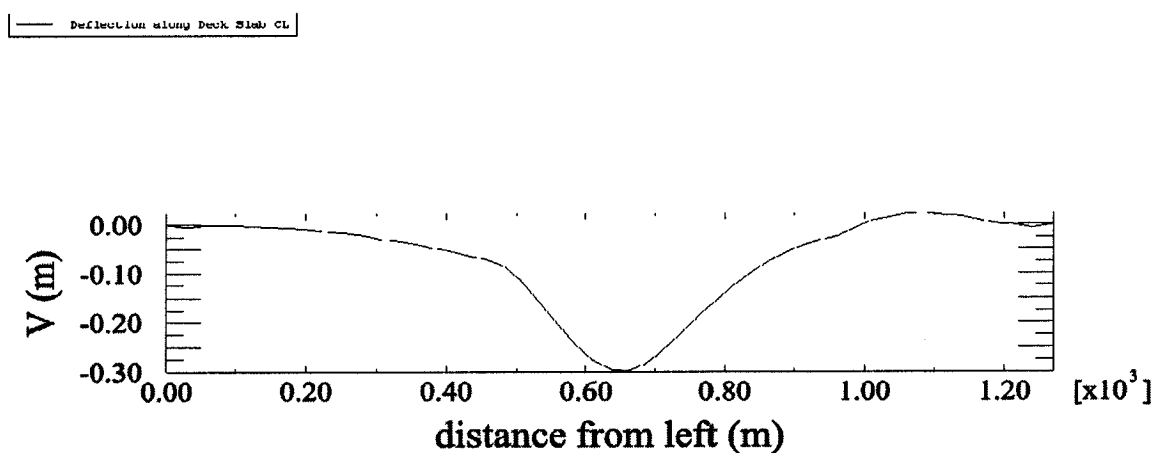


Figure 3-17-a Deck-Slab centerline deflection along Model I-2<sup>nd</sup> Chang River Bridge under the self and dead loads, (the distance in the horizontal axis is measured from the bridge left side).

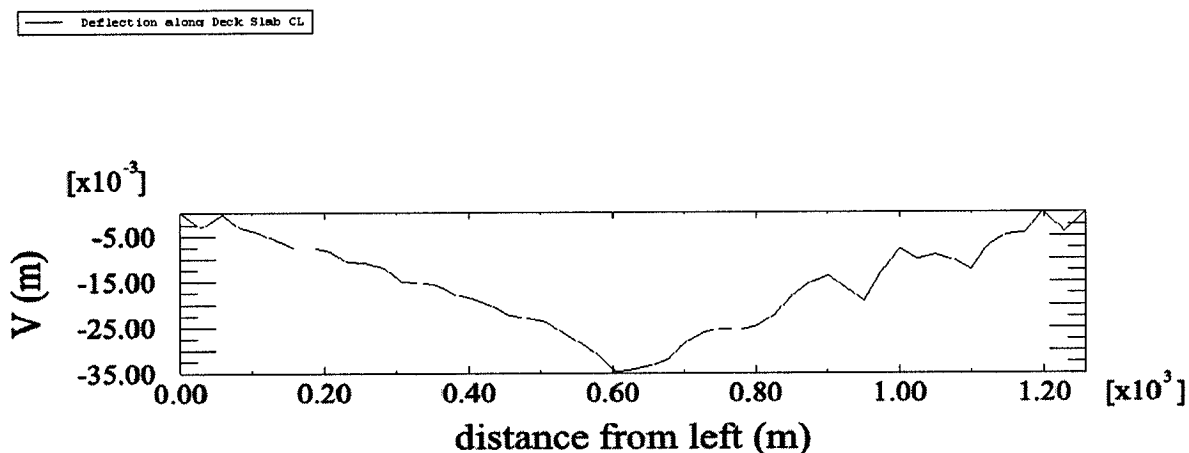


Figure 3-17-b Deck-Slab centerline deflection along Model I-2<sup>nd</sup> Chang River Bridge under the self and dead loads and pretension of cable stays, (the distance in the horizontal axis is measured from the bridge left side).

### 3.7.6 Approaching the Basic Static and Dynamic Properties of Reference Bridges:

The hybrid system is proposed in the form of three bridge arrangements, each dimensioned to match the general geometrical shape, dimensions and configuration of one of the reference bridges mentioned earlier. Although this “geometrical match” can be performed directly by the three-dimensional modeling step, approaching the structural static and dynamic characteristics of the reference bridges is very difficult and worthwhile task. The different materials and sectional properties would diverge the new system behavior from that of the reference bridges, particularly the dynamic behavior, but with proper design it could leads to a better performance.

The “geometrical match” includes matching (i) the bridge central and side span lengths, (ii) the pylon shape (with some minor changes mentioned earlier) and height, (iii) the cable stays configurations for the central span and the side spans, (iv) the deck section

boundary dimensions, and (v) the deck and pylons boundary conditions. On the other hand, approaching the structural basic characteristics includes (i) approaching the static deformations under extreme loading conditions provided that the stress failure criteria are satisfied, (ii) approaching the first fundamental vertical bending and torsional natural frequencies of the reference bridges.

The CFRP cables have very high apparent stiffness to weight, and strength to weight ratios. This plays a major role in differentiating the cable diameter and the overall static and dynamic behavior of the new system and the reference bridges. Adding to that, the four deck section models stiffness and weight, and their anisotropic behavior under the applied stresses, results in different deck behavior and different design variables from those of the traditional bridges.

However, approaching the structural characteristics of the reference bridges would lead to (i) distinguish and carefully define the material, geometrical and structural variables that affect the structural behavior to be investigated in the performance and optimization study, (ii) comprehend the proposed system capabilities and limits in comparison to the reference bridges, (iii) decide the types of advanced composites to be used in each model, together with optimizing their micro/macro design, and (iv) find applicable ranges of the deck section components internal dimensions and proportions.

### **3.7.7 Ultimate and Serviceability Limit States and Material-Sectional Optimization:**

This stage includes performing the ultimate and serviceability limit states of CHBDC-01, which leads to finalizing of the micro-macro-level-design of the advanced chopped composites, laminas and laminate together with deck section design in the structural level. At the end, the assembled composite box girder is optimized minimizing the materials needed, simplifying the micro/macro structure to a practically acceptable level and deciding the final detailed properties.

This includes study of the system behavior under the ultimate traffic loads of single or multiple design truck loads, or single or multiple design lane loads. As the proposed



bridges are assumed to have very long spans, it is apparent that the critical loading case is the full design lane loading of all lanes.

In order to avoid a very conservative prediction of the material resistance, stress ultimate limit states have to be identified in this stage. Those limit states are mainly related to the material failure modes such as the delamination of the advanced composite laminate box girder or de-bonding of the deck section components. Another limit state could be related to the material failure criteria, such as the material stress failure conditions (e.g. Tsai-Hill failure criterion).

On the other hand, the serviceability limit states would involve a thorough investigation of the deformations, particularly the deflections, under the service design loads, keeping those deformations within acceptable limits. In addition, limiting the vibration of the deck due to traffic loads, and vortex shedding within the acceptable ranges are parts of the serviceability checks.

Minimizing the amount of material used and maximizing the efficiency and the structural safety of the proposed system are the core factors of an optimization process in this step of the analysis design procedure. This optimization process can be divided into two interactive cycles; the structure level optimization cycle and the micro/macro level optimization cycle. The structure level cycle is similar to that followed in the design of traditional long span cable-stayed bridges, which involved minimizing the required sections and work needed to construct the bridge, and maximizing the structural performance of the system. The outputs of the structure level optimization process characterize the boundary of the second optimization cycle, hence bounding the required general properties of the advanced composite materials. Consequently, the micro structure design of the lamina and macro-structural design of the laminate during the second cycle of the optimization process lead to minimizing the fiber content, or choosing a suitable fiber type and alignment, optimum laminate lay-up ...etc. In this step, it is apparent that the final hybrid deck section details are approached and minor changes can take place during the next step.

### **3.7.8 Aerodynamic Stability of the Bridge:**

Long span bridges are very flexible structures and their aerodynamic stability is main concern in the analysis-design procedure. This step includes flutter and buffeting analysis as well as vortex shedding and cable vibrations. In this research, the investigation is limited to flutter analysis because it is deemed to be the most critical while the rest are assigned to future research. The data used in this research are the results of the section tests for the reference bridge; the Second Chang River, which have been collected from the original wind tunnel test results for this bridge. Referring to Figure 3.5, it is obvious that the deck section of Model IV does not perfectly match the reference bridges sections; hence the reference bridges aerodynamic test results can only be used for approximate evaluation of this particular model; however, the flutter stability is not a concern for this model. The torsional frequencies are very high in comparison with the vertical natural frequencies and the two types of frequencies, the vertical bending and the torsional, are never coupled.

Because of the anisotropic nature of the finite element model, and the highly nonlinear behavior of the proposed very-long span hybrid cable-stayed bridge, non-linear time domain analysis for the flutter response evaluation has to be employed rather than a frequency domain analysis. During this stage, the final section's internal dimensions together with advanced composite laminate and laminas material micro/macro final design is further approached. The damping properties of the system are also to be evaluated and the needs for adding supplementary damping systems for better dynamic and aerodynamic performance are to be investigated.

### **3.7.9 Summary and Flowchart of the proposed Analysis and Design Procedure:**

The analysis-design procedure, presented in the preceding sections to model the proposed hybrid cable-stayed bridge system, is summarized here. This procedure includes the micro/macro simulation of advanced composite material properties, the macro/structural level nonlinear anisotropic finite element modeling, checking the advanced composite material failure criteria, approaching the basic geometric, static and dynamic properties

of the reference bridges, application of pretension forces to the cable stays according to zero deflection under self and dead loads, investigation of ultimate and serviceability limit states with material and sectional optimization, and aerodynamic stability investigation.

Figure 3-18; represents a flow chart of the analysis design procedure presented in this research.

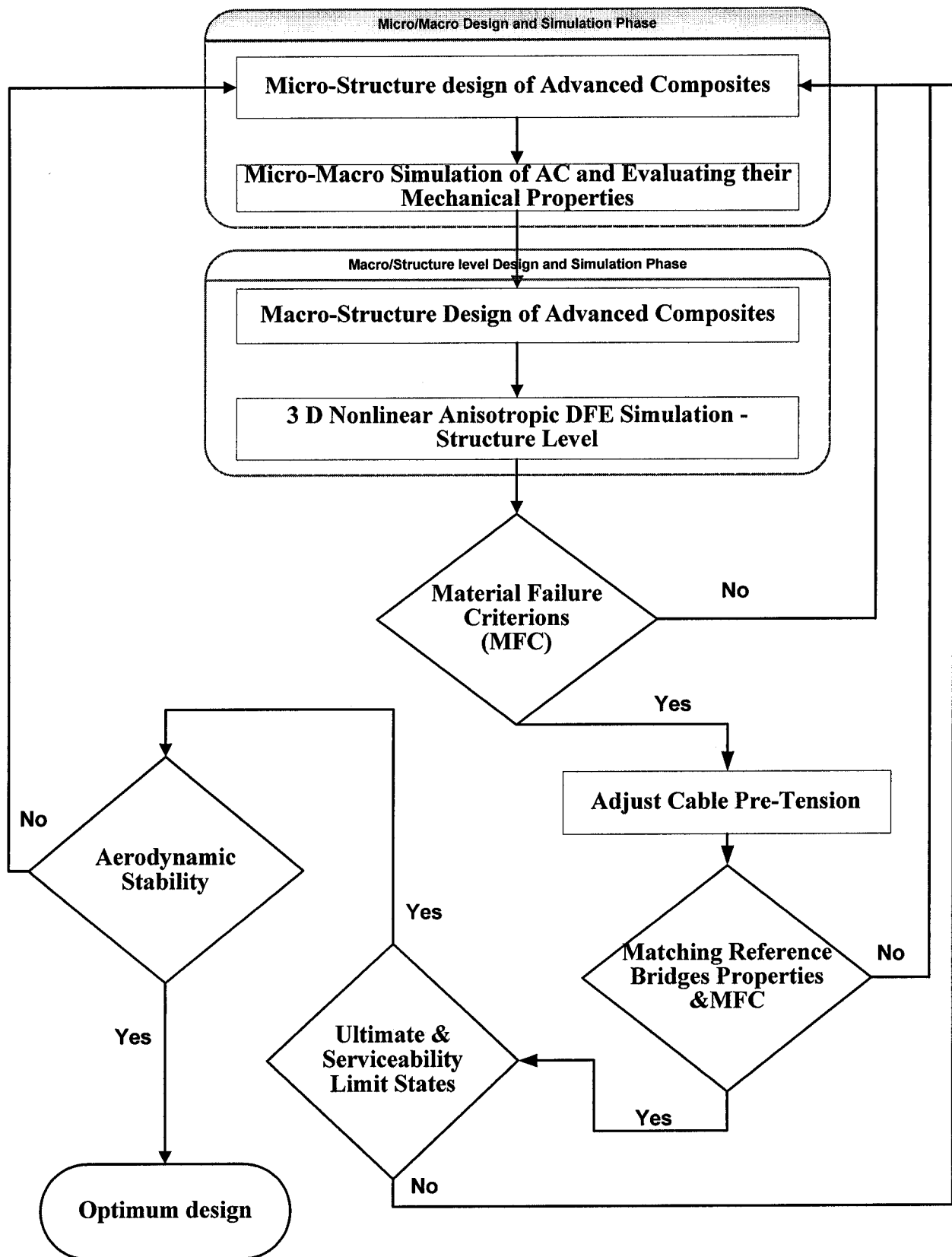


Figure 3-18 Analysis Design Protocol for Long-Span Hybrid Cable-Stayed Bridge

## **Chapter 4**

# **Evaluation of Advanced Composite Material Properties**

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# **Evaluation of Advanced Composite Material Properties**

### **4.1 Introduction:**

In this chapter, two approaches have been proposed for the simulation and evaluation of advanced materials at the micro-scale level, depending on the random or periodic nature of the material. Both methods represent the material properties at the micro-level and transfer them to the macro level. Consequently, the individual properties of the advanced composites components and the random or the periodic nature of their existence are the input and the material properties of the composite at the macro-level are the output of this important step in the analysis and design procedure. Then, with these resulting properties, the basic assumptions of continuum mechanics are valid. Hence the structure being built using advanced composite materials can be analyzed using continuum mechanics.

The first approach has been developed in this research using the stochastic finite element method to simulate a single and multi fiber random oriented composite material, introduced for use in civil-advanced-composite structures, after evaluating their mechanical properties. The second approach is adopted for the evaluation of the material properties regarding the periodic existence of their inclusions (fibers).

## **4.2 Stochastic Finite Element Modeling of Chopped Fiber Reinforced Composites:**

### **4.2.1 Introduction :**

The Chopped Fiber Reinforced Polymers are usually formed from a polymer matrix with short fiber or fine inclusions randomly distributed throughout the material. The stochastic finite element method is employed to evaluate their mechanical Properties. The mathematical representation of the stochastic nature of such composites was originally developed in the “Condensed matter physics” and adopted in the “paper industry” field and in the aerospace industry [eg. Freguson et al 1998, Ostaja-Starzewski et al 1993 &1999, Pellegrino et al 1999 and Miehe et al 2002]. In previous research, the method was derived to model composites with random inclusions of very fine micro structure (e.g. paper, duplex steel, particulate composites with very fine insertions...etc). In the present research, the method is extended to be applicable on less fine microstructures with fiber dimensions in the range of the regular glass, steel, or carbon fibers used in regular mechanical- or civil- advance composite structural applications. A simplified random system relating the random fiber fraction in the mesoscale to the size of the element, involving a variable “probability band width”, is also introduced. The use of multi-type-fibers in the chopped FRP, by introducing multi-layers (or transparent layers) of separated random distribution generators, is proposed. In addition, a practical field problem, the stress/strain analysis, is involved in the SFE program development. Such an application can be experimentally performed.

#### **Assumptions:**

- Both materials, the matrix and the inclusion (fibers in this case) are linear elastic materials with known properties.
- The dimensions of the fiber (diameter and length) are very small in comparison with the continuum dimensions.
- The distribution of the fibers throughout the FRP is random (with no possibility of local fiber alignment orientation due to the manufacturing process and/or placement of the material ...etc).

- The volumetric fiber fraction can be represented by a normal distribution in relation to a mesoscale window dimensions.

**Terminology:**

Jones, R, [1999] gives the following definitions for Micromechanics and Macromechanics:

“**Micromechanics:** the study of composite material behavior wherein the interaction of the constituent materials is examined in detail as part of the definition of the behavior of the heterogeneous composite material.

**Macromechanics:** the study of composite material behavior wherein the material is assumed homogeneous and the effects of the constituent materials are detected only as averaged apparent properties of the composite material.”

Consistently, the following definitions for different scales are to be used:

**Microscale:** a scale where each of the constituent materials of the composite material can represent a single phase and the boundary between any two elements is a phase-phase boundary.

**Macroscale:** is the scale where the properties of the composite material are the averaged apparent properties of the composite material.

**Mesoscale:** a transition scale larger than the microscale and less than the macroscale and represent a bridge between the two scales.

#### 4.2.2 Mathematical Description of the Random Micro-Set and the Meso-Window :

The following is a mathematical description of the problem summarized from the related literature, [e.g. Ostaja-Starzewski et al, 1993 and 1999, and Miehe et al, 2002]:

A random microstructure **B**, which is taken as a set of all realizations, **B**( $\omega$ ) is parameterized by the sample events  $\omega$  of the sample space  $\Omega$

$$B = \{B(\omega); \omega \in \Omega\} \quad (4-1)$$



It follows that it is a specific heterogeneous material sample and its probability is introduced to deal with the set (4-1).

In the present research, planer, two-phase microstructures governed by linear constitutive laws is considered, such that  $\mathbf{B}$  is given by the two state random field  $\mathbf{C}$

$$\mathbf{C} = \{\mathbf{C}(\tilde{\mathbf{x}}, \omega); \tilde{\mathbf{x}} \in D, \omega \in \Omega\} \quad \mathbf{C}(\tilde{\mathbf{x}}, \omega) = \{\mathbf{C}^{(1)}, \mathbf{C}^{(2)}\} \quad (4-2)$$

Where  $\mathbf{C}^{(1)}$  and  $\mathbf{C}^{(2)}$  are second rank tensor representing the material properties.  $\mathbf{C}(\mathbf{x}, \omega)$ 's are realizations of constitutive laws, defined on a certain domain  $D$  of  $\mathbb{R}^2$  in the two dimensional problem, or  $\mathbb{R}^3$  space in the three-dimensional problem.

To account in the analysis, for all the details of a typical realization  $\mathbf{C}(\omega)$  of the random field  $\mathbf{C}$ , one may employ a very fine FE mesh, so that each element would be within the same phase and the phase-phase boundaries would be traced along the element sides. Such an approach would be very expensive for mesh generation and could be an extensive task highly dependent on the micro structural realizations and the computer capacity could limit the computational efficiency. Therefore meso-scale moduli are introduced to bridge the gap between micro-scale material heterogeneity and macro scale mechanical behaviors.

Define a square shaped ( $L$  by  $L$ ) window, which leads to a non-dimensional scale parameter:

$$\delta = \frac{L}{d} \quad (4-3)$$

Where ( $d$ ) denotes a typical micro scale and  $L$  or  $\delta$  are each called a “mesoscale”. The window may be placed anywhere in the domain of the random microstructure  $\mathbf{B}$ , and so, at any fixed  $L$  it defines a new random medium:

$$\mathbf{B}_L = \{\mathbf{B}_L(\omega); \omega \in \Omega\} \quad (4-4)$$

It is clear that the limit  $\delta$  approaching infinity should result in a conventional continuum typically sought in the bounding and effective medium theories of micro mechanics. In

other words, the response of any nearly infinitely large specimen becomes almost the same as that of any other one subjected to the same loading conditions.

The effective moduli for a finite window domain  $\mathbf{B}_L$  with its boundary sampled in random FRP material can be defined in several different ways. One approach, based on an interpretation of Hooke's law, is one in which either a uniform strain or a uniform stress is prescribed. In the first case, the essential (or displacement, Dirichlet) boundary condition is employed, while in the second case, the natural (or traction Neumann) boundary condition is used. In the first step, the stress-strain relation can be described as:

$$\bar{\sigma}_i = C_{ij}^e \varepsilon_j^0 \quad \text{under} \quad u(\tilde{x}) = \varepsilon_j^0 x_j \quad \forall \tilde{x} \in \partial B_L \quad (4-5)$$

where  $\bar{\sigma}_i$  is the resulting mean (volume average) stress. In the second step, the strain-stress relation can be described as:

$$\bar{\varepsilon}_i = S_{ij}^n \sigma_j^0 \quad \text{under} \quad t(\tilde{x}) = \sigma_j^0 n_j(\tilde{x}) \quad \forall \tilde{x} \in \partial B_L \quad (4-6)$$

where  $\bar{\varepsilon}_i$  is the resulting mean strain.

The relation (4-5) is employed in the finite element formulation (section 4.2.3). Figure 4.1 shows the mesoscale bridging the properties in the microscale to the macroscale levels.

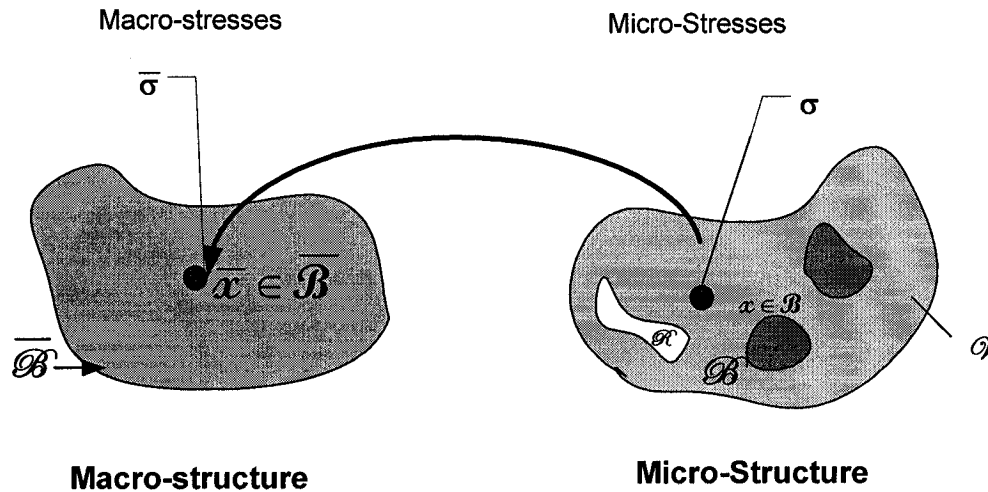


Figure 4.1 The Mesoscale is bridging the properties of micro to the macro sets.

### 4.2.3 The Finite Element Formulation:

The Stochastic Finite element formulation contains two parts; a random process to generate the material properties (Section 4.2.4) and a standard finite element formulation of plane-stress analysis of a thin slice. In the first process, the domain is divided into a finite number of meso-windows. The heterogeneous composite material properties are homogenized in this process resulting in averaged properties for every meso-window, which is defined by the random process and has different material properties from any other window. Since the material of any meso-window is homogenized, continuum mechanics can be applied, and the finite element formulation can be employed.

A meso-window can be represented by one or more finite elements. In the direct formulation, a finite element (e) is defined by its nodes (m, n, o ...etc) and boundary lines. Let the displacement (u) at any point within the element be approximated by a vector:

$$u = \hat{u} = \sum N_i \alpha_i^e = \begin{bmatrix} N_m & N_n & N_o & \dots \end{bmatrix} \begin{Bmatrix} \alpha_m \\ \alpha_n \\ \alpha_o \\ \vdots \end{Bmatrix} = N \alpha^e \quad (4-7)$$

Where;

N: prescribed functions of position, or shape functions,

$\alpha^e$ : the nodal displacements for element (e).

For the case of plane stress;

$$u = \begin{Bmatrix} u(x, y) \\ v(x, y) \end{Bmatrix} \quad (4-8)$$

is the movement of any point within the element, which has the two shown components, and the corresponding displacements of node (m) are:

$$a_m = \begin{Bmatrix} u_m \\ v_m \end{Bmatrix} \quad (4-9)$$

The shape functions  $N_m$   $N_n$   $N_o$  ...etc have to be chosen to give appropriate displacements when the coordinates of the appropriate nodes are substituted. As the displacements are known at all points, then the strains are given by:

$$\varepsilon = Lu \quad (4-10)$$

Where L is a suitable linear operator, and is defined for the plane stress case as:

$$\varepsilon = \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} = \begin{Bmatrix} \frac{\partial u}{\partial x} \\ \frac{\partial v}{\partial y} \\ \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \end{Bmatrix} = \begin{bmatrix} \frac{\partial}{\partial x} & 0 \\ 0 & \frac{\partial}{\partial y} \\ \frac{\partial}{\partial y} & \frac{\partial}{\partial x} \end{bmatrix} \begin{Bmatrix} u \\ v \end{Bmatrix} \quad (4-11)$$

and the strains can be approximated as:

$$\begin{aligned} \hat{\varepsilon} &= L\hat{u} \\ \hat{\varepsilon} &= LNa \\ \hat{\varepsilon} &= Ba \end{aligned} \quad (4-12)$$

where;

$$B = LN$$

Then Matrix B can be easily evaluated using the selected shape functions.

Ignoring any possible initial strains and any initial residual or pre-stresses, and assuming linear elastic behavior for the homogenized material in the element, the stress can be defined as;

$$\sigma_i^e = C_i \varepsilon_i^e \quad (4-13)$$

where,

$\sigma_i^e$  : the stress of element (e) at the meso-window (i),

$C_i$  : the elasticity matrix and defined for every meso-window, and hence for every element in that window,

$\varepsilon_i^e$  : the strain of element (e) at the meso-window (i).

The stress is also has three components corresponding to the strain in equation (4-11);

$$\sigma = \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} \quad (4-14)$$

For the homogenized material of the meso-scale (i);

$$C_i = \begin{bmatrix} 1 & \nu_i & 0 \\ \nu_i & 1 & 0 \\ 0 & 0 & \frac{(1-\nu_i)}{2} \end{bmatrix} \quad (4-15)$$

Let;

$$q^e = \begin{Bmatrix} q_m^e \\ q_n^e \\ q_o^e \\ \vdots \end{Bmatrix} \quad (4-16)$$

define the nodal forces, which are equivalent to the integration of the boundary stresses and the distributed loads on the element. The nodal forces for the plane stress are:

$$q_m^e = \begin{Bmatrix} q_{um} \\ q_{vm} \end{Bmatrix} \quad (4-17)$$

The components  $q_{um}$ , and  $q_{vm}$  correspond to the displacements  $u$  and  $v$ .

Let  $\partial a^e$  be virtual displacements applied at the element's nodes. This results in the following displacements and strains within the element:

$$\begin{aligned} \partial u &= N \partial a^e \\ \partial \varepsilon &= B \partial a^e \end{aligned} \quad (4-18)$$

The work done by the nodal forces is equal to the sum of the products of the force components and corresponding displacements:

$$\partial a^{eT} q^e$$

and the internal work per unit volume done by the stresses are, (ignoring the work done by the body forces);

$$\partial \varepsilon_i^{eT} \sigma_i^e$$

or  $\partial a^T B^T \sigma_i^e$

Equating the external work with the total internal work obtained by integrating over the volume of the element  $V^e$ :

$$\partial a^{eT} q^e = \partial a^{eT} \int_{V^e} B^T \sigma_i^e .dV^e \quad (4-19)$$

Equation (4-19) can be rewritten as:

$$q^e = K_i^e a^e \quad (4-20)$$

where

$$K_i^e = \int_{V^e} B^T C_i^e B .dV^e \quad (4-21)$$

#### 4.2.4 Random Process to Generate the Composite Material Properties:

Recall the mesoscale,  $\delta = \frac{L}{d}$  (Equation 4.3), which is the ratio between the meso-window dimension and the microscale dimension “d”. The dimension (d) can be assumed equal to the fiber diameter in this case since, with this dimension, the two phases of the heterogeneous material are theoretically separated.

A bandwidth ( $\gamma$ ) is introduced, which represents the band or “bar” on the central part of a normal distribution, where the probability of achieving the average fiber fraction is very high. This “probability band width” (Figure 4.2) is related to the dimension of the mesoscale as follows:

As  $\delta$  increases to a very high value (say the order of  $10^6$ ) then  $\gamma$  increases and approaches a maximum value close to 1.0 in a normalized normal distribution, and the band would cover the overall distribution in an ideal process. Conversely, as  $\delta$  is decreased to a very low value (say the order of 10),  $\gamma$  also decreases and approaches a minimum value close to zero. In other words, as the mesoscale “zooms in” or the meso-window dimensions are decreased, a sudden and nonlinear reduction in  $\gamma$  should be

expected and the independent realizations (the fiber fraction in this simplified case) diverge from the mean values with very high probabilities.

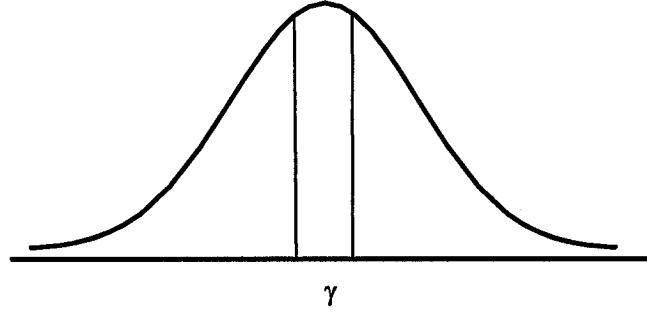


Figure 4.2 Normal distribution (Gauss distribution) with probability band width ( $\gamma$ )

A good and simple representation of such process can be quantified by introducing an exponential relation between the “probability band width” and the “mesoscale” dimension (assuming a constant “d”) in the form of:

$$\gamma = a_1 e^{a_2 \delta} \quad (4.22)$$

and the limits for this relation are:

$$\begin{array}{lll} \gamma = \gamma_1 & \text{at} & \delta = \delta_1, \text{ and,} \\ \gamma = \gamma_2 & \text{at} & \delta = \delta_2 \end{array} \quad (4-23)$$

sub (4-23) into (4-22) will end with:

$$\begin{aligned} \gamma &= \gamma_1 \cdot e^{\eta(1-\frac{\delta}{\delta_1})} \\ \eta &= -\Delta \cdot \ln\left(\frac{\gamma_2}{\gamma_1}\right) \\ \Delta &= \frac{\delta_1}{\delta_2 - \delta_1} \end{aligned} \quad (4-24)$$

An example of a probability band width relation with the mesoscale is shown in Figure 4.3, for  $\gamma_1 = 0.05, \gamma_2 = 0.95, \delta_1 = 10^2$ , and  $\delta_2 = 10^5$ .

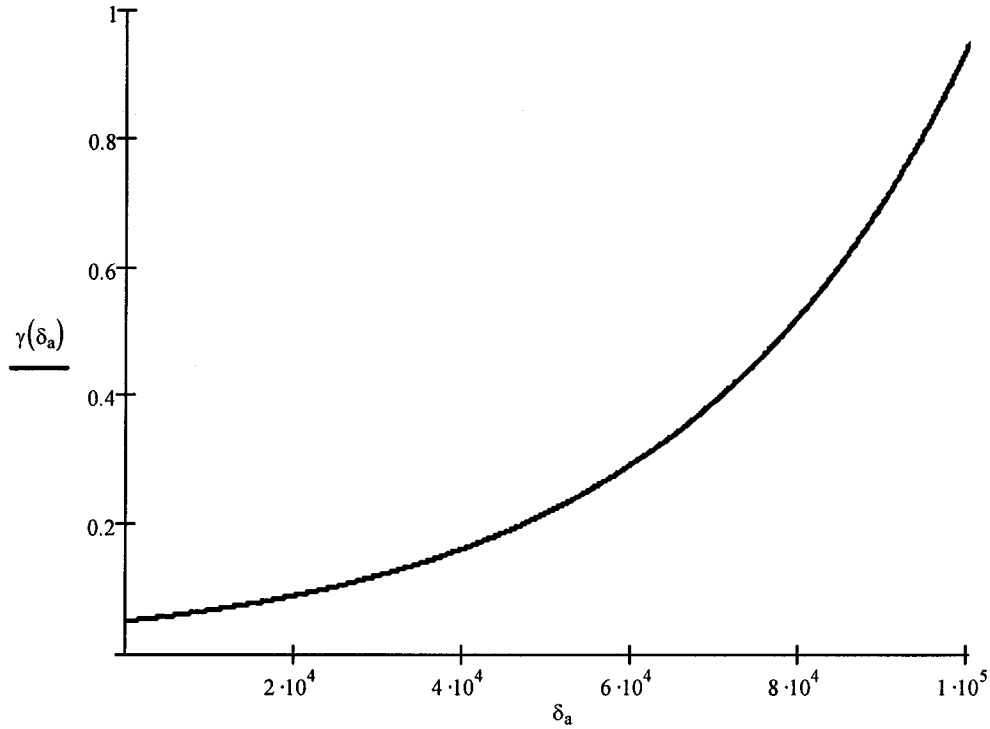


Figure 4.3 The Probability band width for  $\gamma_1 = 0.05, \gamma_2 = 0.95, \delta_1 = 10^2$ , and  $\delta_2 = 10^5$

The Random Process of generating the random realization (the fiber fraction) can be summarized as follows:

- Decide suitable values for  $\delta_1, \delta_2, \gamma_1$  and  $\gamma_2$  according to the minimum size of the inclusions (fiber diameter) and the available experimental data.
- Chose a mesoscale according to fiber size and the desired FE mesh (this can be done automatically in the SFE program).
- Evaluate  $\gamma$  from the exponential relation (Equation 4.24).
- Generate a random variable through a normalized normal distribution;

$$0 \leq Ran \leq 1.0 \quad (4-25)$$

- Evaluate the divergence of the random variable (Ran) from the distribution centre;

$$Div = Ran - 0.5$$

$$\begin{aligned} \text{if } |Div| \leq \gamma & \quad \text{then } f_i = f \\ \text{Otherwise,} & \quad f = f(1 + Div) \end{aligned} \quad (4-26)$$



#### 4.2.5 Chopped FRP with Two or More Fiber Types:

If more than a single-fiber type is used in the chopped FRP material (to improve the composite mechanical properties, for example), one can also use the “two-phase” stochastic process that was introduced earlier employing as many independent random variables as the number of fiber types. However, the distribution of each fiber in the FRP is assumed to be independent from the distribution of any other fiber in the material, provided that all the distributions are normal distributions, with the same general characteristics (i.e. the general “ $\gamma \Leftrightarrow \delta$ ” relation is applicable), but with independent random occurrences. The system can now be represented by transparent slides; on each of those slides, there is an independent random generation process of one fiber fraction. Thereafter, the projection of the meso-window-mesh on those transparent slides gives the random values of different realizations (fiber fractions of all independent fibers) in any specified meso-window. Then the properties at each window, and according to the fiber fractions of all the fibers at that window, are averaged and the resulting homogenized material represents the properties of a composite material in a meso-window. The many different materials of the meso-windows represent the random properties of the random distribution of different fibers in the original heterogeneous material. In other words, each window in the meso-mesh will represent a different material, according to the fiber fractions of different fibers that are generated randomly.

#### 4.2.6 Averaged Properties in the Meso-Window:

Applying the principle of minimum complementary energy leads to a lower bound for the modulus of elasticity of a composite material [Jones, 1999] as:

$$E_i \geq \frac{E_m \cdot E_{f_i}}{V_m E_{f_i} + f_i E_m} \quad (4-27)$$

where,

$E_i$  : averaged modulus of elasticity in the meso-window (i)

$E_m$  : modulus of elasticity of the matrix

$E_{f_i}$  : modulus of elasticity of the fiber

$V_m$ : volumetric fraction of the matrix ( = 1.0 –  $f_i$  )

$f_i$  : fiber fraction at the meso-window (i), (random)

$$f_i = \sum f_j$$

where,

$f_j$ : fiber fraction at the meso-window (i) for fiber type j, (random).

The “lower bound” Poisson’s ratio of the FRP is usually calculated as the volume average of Poisson’s ratios of the matrix and the fiber (assuming that both, the matrix and the inclusions, are linear elastic).

On the other hand, applying the principle of minimum potential energy leads to an upper bound of the composite material properties [Jones, 1999 and Vinson, 2002] as follows:

$$\begin{aligned} E_i &\leq \frac{(1 - \nu_{f_i} - 4\nu_{f_i}\nu_i + 2\nu_i^2)}{(1 - \nu_{f_i} - 2\nu_{f_i}^2)} * E_{f_i} * f_i + \frac{(1 - \nu_m - 4\nu_m\nu_i + 2\nu_i^2)}{(1 - \nu_m - 2\nu_m^2)} * E_m * V_m \\ \nu_i &= \frac{(1 - \nu_m - 2\nu_m^2) * \nu_{f_i} * E_{f_i} * f_i + (1 - \nu_{f_i} - 2\nu_{f_i}^2) * \nu_m * E_m * V_m}{(1 - \nu_m - 2\nu_m^2) * E_{f_i} * f_i + (1 - \nu_{f_i} - 2\nu_{f_i}^2) * E_m * V_m} \end{aligned} \quad (4-28)$$

Where;

$\nu_m$ : Poisson’s ratio of the matrix,

$\nu_{f_i}$ : Poisson’s ratio of the fiber at the meso-window (i).

In equations (4-28), it is important here to note that the composite material modulus of elasticity is a function of its Poisson’s ratio.

The final properties in the meso-window are assumed as an average of the Upper and Lower bound properties. However, the difference between the two bounds is very high, and also the industrial-experimental results indicate that the modulus of elasticity of the chopped FRP is less or equal to the lower bound when the volumetric fiber fraction (f) is less than 17%, and increases over the lower bound as the fiber fraction is increased. Consequently, the mid-average of the upper and lower bound, or  $(E_{i-lowB} + E_{i-upB})/2$ , gives an over estimation for the modulus of elasticity.

The material properties of the overall chopped FRP material (the sum of all the meso-windows in the present model) are the average properties of all the windows. The averaged properties calculated using the theoretical bounds are higher than the available experimental results. However, it has been noted that matching the available experimental results can be achieved by narrowing the wide band between the upper and

lower bounds according to  $f_i$ . It has been found that the use of the following tri-linear upper bound in the meso-window, results in final average properties that matches the experimental results for the overall material. The relation is linear with one variable parameter,  $\xi_{f_i}$ , which is a function of the fiber fraction  $f_i$ , and the values of the constants can be found by trial-and-error. Following is the tri-linear upper bound of the modulus of elasticity in the meso-window:

$$\begin{aligned}
 E_i &= E_{i-lowB} + \xi_{f_i} (E_{i-upperB} - E_{i-lowB}) \\
 \xi_{f_i} &= 0 \quad \text{when } f_i \leq 0.17 \\
 \xi_{f_i} &= 3.4f - 0.57 \quad \text{when } 0.17 < f_i \leq 0.22 \\
 \xi_{f_i} &= 1.8f - 0.3 \quad \text{when } f_i > 0.22
 \end{aligned} \tag{4-29}$$

#### 4.2.7 Finite Element procedure and SFE Program:

The FE procedure includes two major parts; a stochastic process to generate the material data, and a standard FE part to generate the stress matrices, and elements' stiffness involving appropriate shape functions, ...etc. With a homogeneous material, the material properties are the same everywhere in the specified domain, and hence at all the finite elements. However, for a heterogeneous composite, which is homogenized in different meso-windows, the material properties are randomly different at every window, and accordingly at every finite element. Once the material properties are generated through the random process, any field problem can be represented over the examined domain. Here, it is convenient to perform a plane-stress analysis like a rectangular thin plate (Section 4.2.3) fixed on one side and under constant tensile stress on the other side.

The following is a brief summary of the SFE program and verification tests:

- i) The program has five major subroutines; INPUTA, ELEMA, ASTIF, SOLVE and STRES.
- ii) INPUTA: this subroutine does the following:
  - Inputs the degree of the element to be used, the number of Gauss points, and the type of the problem (Plane Stress/ Plane strain).
  - Inputs the number of elements in the X and Y directions, the X-length and Y-length, the applied stress and the plate thickness.

- Inputs the number of fiber types, fiber fractions (up to three types),  $\delta_1, \delta_2, \gamma_1, \gamma_2$ , modulus and Poisson's ratio for the matrix and the fiber(s).
- Performs a Random process to generate the material properties for every window, after relating the window size to the minimum fiber diameter, number of elements, and the minimum dimensions of the plate (or the domain).
- Inputs or generate the nodal data and calculate the nodal forces.
- Inputs or generate the boundary elements (where the boundary conditions are applied).
- Inputs or generate the element data.

iii) ELEMA: this subroutine does the following:

- Forms the element constitutive matrix.
- Accumulates the stiffness by looping over the Gauss points and calls the SHAPE subroutine (shape functions).
- Forms the Jacobian for Gauss points.
- Inverts the Jacobian.
- Forms the shape function derivatives with respect to global coordinates
- Forms the stress matrix for gauss points and writes on a sequential file (FILE 1).
- Forms and accumulates contributions to the lower half of the element stiffness matrix and writes on a second sequential file (FILE 2).

iv) ASTIF: This subroutine forms the Load Vector and assembles the Global Stiffness Matrix by reading the element stiffness matrices from (FILE 2). Boundary conditions are imposed (by adding boundary element stiffnesses to the assembled global stiffness matrix).

v) SOLVE: solves the resulting simultaneous equation system, taking into account the symmetric banded nature of the stiffness matrix, prints the solution vector and searches for the absolute maximum displacements in both directions.

vi) STRES: This subroutine reads the elements' stress data from the sequential (FILE 1), computes stresses at the GAUSS points of all elements, and searches for the maximum stresses.

vii) Verification tests:

- To check the Finite Element formulation of the plane stress/plane strain part of the program and to check the accuracy of the calculations for this part, the program is tested using homogeneous material (steel and aluminum) and the results are compared with solved examples from different texts books.
- To check the effect of refining the mesh on the numerical stability of the solution, a detailed examination is carried out using different meshes, up to very fine meshes of 100 element per direction.
- The Stochastic generation of the heterogeneous material is tested separately.
- Hundreds of runs are performed for different material combinations, different fiber fractions, and different mesh sizes ...etc.

#### **4.2.8 Discussion and Results:**

The Stochastic Finite element procedure proposed in the previous sections is a capable tool to evaluate the mechanical properties of the composite material with random oriented fibers. The available methods in the mechanics of composite materials can evaluate only the upper or lower bounds of the modulus of elasticity and Poisson's ratio, while the material strength is only evaluated by experimental tests. The numerical effort in the proposed method and the resulting accuracy are acceptable with the use of a reasonably fine meso-mesh. Consequently, the method can be efficient in deciding the proper fiber types, their volumetric fractions and the matrix type in the chopped composite material from the specified mechanical properties. Such mechanical properties are usually defined by the required structural characteristics of a specific structural component where the material will be involved. Subsequently, the experimental tests, that are usually very expensive, can be done to verify and/or calibrate the numerical model. The results obtained from the proposed method show a good match with the available industrial-experimental results. Figure 4.4 show the average of the SFE results in comparison with analytical upper and lower bounds of the modulus of elasticity, for a chopped FRP formed from Glass Fibers and a Vinylester matrix, with fiber fractions between 17.3% and 35.6%. It has been found that the SFE method average moduli results are close in

general to the experimental moduli results (collected from APGF China Ltd and Bodycore Material Testing Ltd).

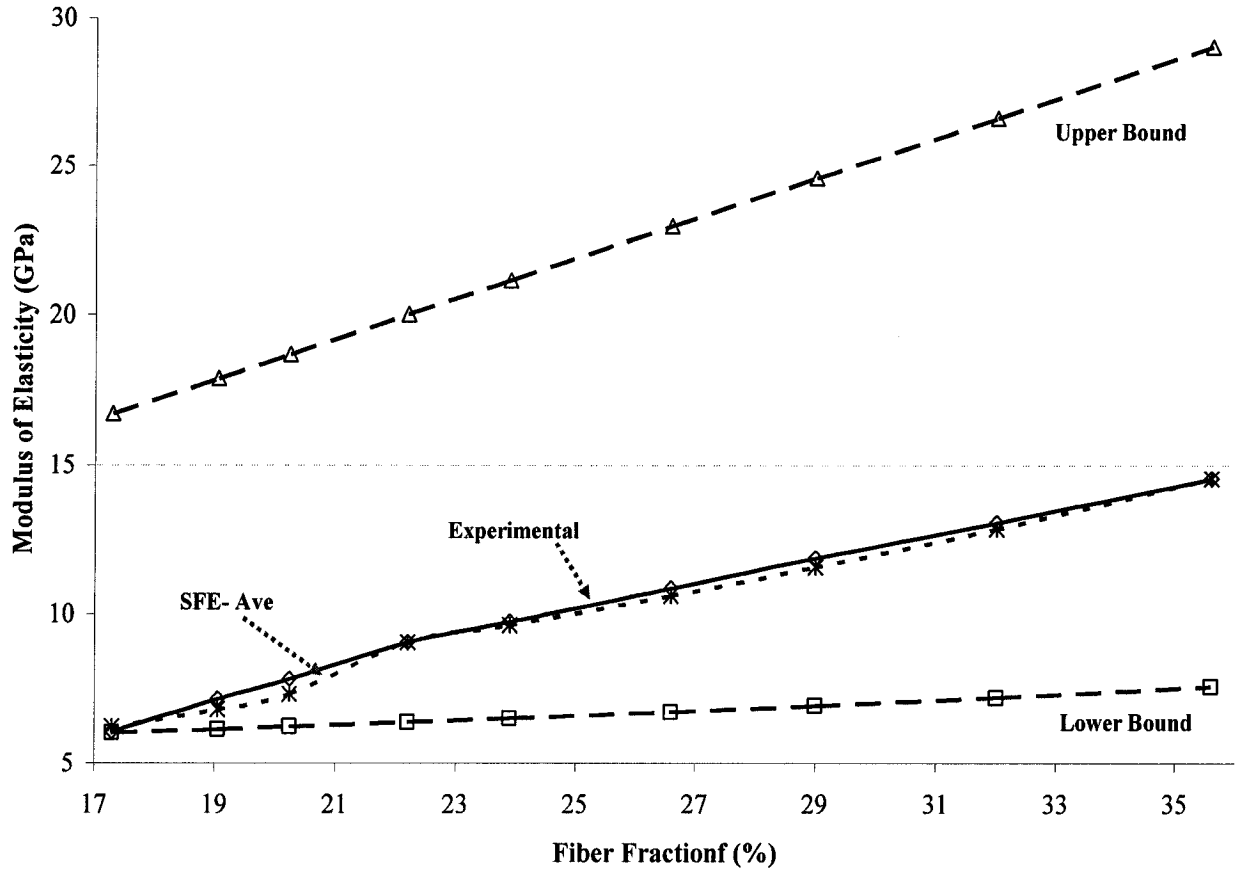


Figure 4.4 The experimental and the calculated averaged elastic moduli for E-Glass fiber and Vinylester matrix for ( $17.3\% \leq f \leq 35.6\%$ ), bordered by the upper and lower bounds.

| Material         | Modulus of Elasticity E (GPa) | Shear Modulus (GPa) | Poisson Ratio $\nu$ |
|------------------|-------------------------------|---------------------|---------------------|
| E-Glass Fibers   | 72.34                         | 28.82               | 0.255               |
| Carbon Fibers    | 190                           | 70                  | 0.29                |
| Steel Fibers     | 207                           | 77                  | 0.29                |
| Vinylester resin | 5.055                         | 1.634               | 0.3                 |

Table 4.1 the properties of different fibers and matrix used in the Chopped FRP Material.

As shown earlier, the method can evaluate any chopped composite formed from single or multi-fiber-types; however, there are no available industrial or experimental results for comparison purpose. The Chopped Fiber Reinforced Polymers formed from E-Glass, or E-glass with the addition of Carbon and/or Steel fibers, of different small fiber fractions (up to one third of the E-Glass fiber fraction) together with a Vinylester matrix are examined. Table 4.1 gives the material properties of the different fibers and the Vinylester matrix involved. Table 4.2 shows the elastic moduli, Poisson's ratios and the strength of different chopped composites obtained from applying the proposed method. The average volumetric fiber fraction and standard deviation for every fiber type resulting from the multi-random process are also shown.

| $f_1$  | $f_{1ave}$ | f1-Standard<br>Deviation<br>( $\sigma_{f_1}$ ) | $f_2$ | $f_{2ave}$ | f2-Standard<br>Deviation<br>( $\sigma_{f_2}$ ) | $f_3$ | $f_{3ave}$ | f3-Standard<br>Deviation<br>( $\sigma_{f_3}$ ) | $E_{ave}$<br>GPa | $\nu_{ave}$ | $F_u$<br>MPa |
|--------|------------|------------------------------------------------|-------|------------|------------------------------------------------|-------|------------|------------------------------------------------|------------------|-------------|--------------|
| 17.3*  | 17.38      | 3.518                                          | -     | -          | -                                              | -     | -          | -                                              | 6.21             | 0.265       | 92           |
| 19.05* | 19.13      | 3.890                                          | -     | -          | -                                              | -     | -          | -                                              | 6.789            | 0.261       | 99           |
| 20.24* | 20.33      | 4.143                                          | -     | -          | -                                              | -     | -          | -                                              | 7.313            | 0.258       | 106          |
| 19.0** | 19.05      | 3.89                                           | 2.0   | 1.99       | 0.408                                          | -     | -          | -                                              | 9.318            | 0.276       | 139          |
| 19.0   | 19.057     | 3.893                                          | 5.0   | 4.97       | 1.019                                          | -     | -          | -                                              | 11.623           | 0.284       | 164          |
| 19.0   | 19.048     | 3.891                                          | -     | -          | -                                              | 2.0   | 1.99       | 0.408                                          | 9.817            | 0.276       | 127          |
| 19.0   | 19.046     | 3.893                                          | -     | -          | -                                              | 5.0   | 4.97       | 1.021                                          | 12.219           | 0.283       | 148          |
| 19.0   | 19.07      | 3.92                                           | 3.0   | 2.974      | 0.614                                          | 3.0   | 3.02       | 0.612                                          | 12.512           | 0.285       | 167          |

\* Less than 2% different from the experimental properties given by Bodycore Material Testing Ltd.

\*\* This combination is used throughout this research for the chopped FRP “deck slab” of Models I, II, and III, and major parts of Model IV.

Table 4.2 The mechanical properties of the Chopped FRP resulting from the Micro/Micro level homogenization process for combinations of E-Glass, Carbon and steel fibers and Vinylester Matrix for different fiber fractions.



### 4.3 Analytical Method to Evaluate Periodic Fiber Reinforced Composites:

#### 4.3.1 Mathematical Presentation of the Method:

On the other hand, in most of the cases, and for a more automated construction process, together with the huge amount of material to be produced for the proposed hybrid long span cable-stayed bridges, it is more practical to use an advanced composite of periodic nature in the micro-scale structure. Hence a second approach, which was developed with dependence on an analytical simulation of the periodic composite material properties, has been used rather than the stochastic method shown earlier. This approach relates the stiffness and strength properties of the advanced composite material to the basic general properties of the components, the fiber and the matrix, and the volumetric percentage of the fiber in the composite. The procedure was originally developed by Luciano et al 1994, and adopted and applied by Davalos et al 1996, Salim et al 1997, Qiao et al 2000 and He et al 2003. This approach is re-derived and presented here completely consistent with the proposed deck laminated assembly.

Consider an infinitely extended linearly elastic solid represented by an assembly of unit cells. For simplicity, let each cell  $D$  be a parallelepiped with dimensions  $a_j$  (Figure 4.5) in the direction of the coordinate axes  $X_j$ , where  $j = 1, 2, 3$ , and let  $V$  be its volume. Then denote by  $\Omega$  the part of  $D$  occupied by the inclusions, let  $(D-\Omega)$  denote the matrix and let  $f$  be the volume fraction of  $\Omega$ .

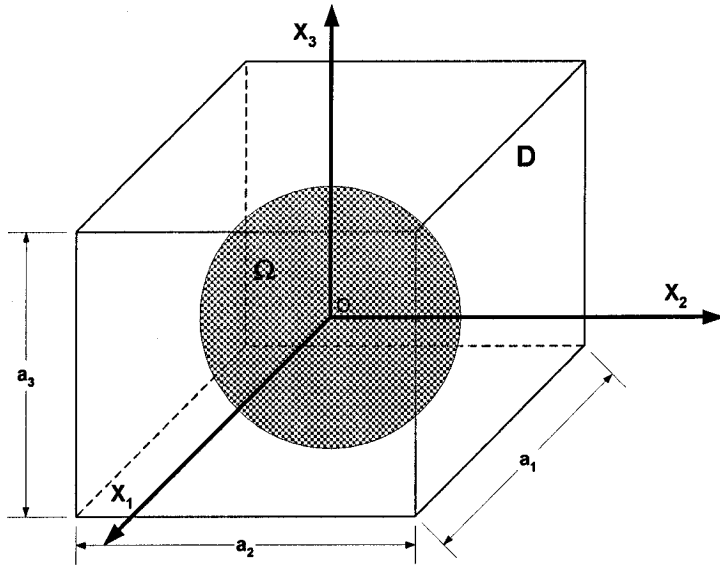


Figure 4.5 Geometry of the unit cell D.

Next, the relation between the eigenstrain (such as residual strains from the manufacturing process, shrinkage and thermal expansion...etc) and the strain inside the inclusion is introduced. Experts of continuum mechanics and elasticity, including Timoshenko and Goodier [1970] and Mura [1987], acknowledge that residual stress is the result of some inelastic strain field, which does not satisfy the compatibility. This strain field is present due to mechanical and thermal processes, which the body has to undergo. Here, Mura's [1987] terminology is adopted, by calling all the mentioned strains as "eigenstrain".

In order to simulate the inclusions inside the body, consider the homogenized eigenstrain  $\varepsilon^*$  defined in all  $D$ , which must be periodic for the particular geometry of the problem and non- zero only in  $\Omega$ . Since the material is linear elastic, the actual stress tensor  $\sigma$  inside the unit cell can be expressed in terms of  $\varepsilon^*$  and the actual strain tensor  $\varepsilon$  in the following way:

$$\sigma = C(\varepsilon - \varepsilon^*) \quad \text{in} \quad D \quad (4.30)$$

where  $C$  is the elasticity tensor of the matrix. Then, assuming the body forces are equal to zero, the tensor  $\sigma$  must satisfy the following equilibrium conditions:

$$\text{div} \sigma = 0 \quad \text{in} \quad D \quad (4.31)$$

where  $\text{div}$  denotes the divergence of a tensor field. Furthermore, since in a solid with a periodic structure and suitable boundary conditions the displacements  $\mathbf{u}$  are periodic, the following Fourier series representation of  $\mathbf{u}$ ,  $\varepsilon$  and  $\varepsilon^*$  can be considered [Luciano et al 1994]:

$$u(x) = \sum_{\xi} \bar{u}(\xi) \exp(i\xi x) \quad (4.32)$$

$$\varepsilon(x) = \text{sym}(\nabla u(x)) = \sum_{\xi} \bar{\varepsilon}(\xi) \exp(i\xi x) \quad (4.33)$$

$$\varepsilon^*(x) = \sum_{\xi} \varepsilon^*(\xi) \exp(i\xi x) \quad (4.34)$$

where  $\xi = (\xi_1, \xi_2, \xi_3)$  with  $\xi_j = 2\pi n_j / a_j$  ( $n_j = 0, \pm 1, \pm 2, \dots, j$ ) not summed,  $j = 1, 2, 3$ )

and:

$$\bar{u}(\xi) = \int_D u(x) \exp(-i\xi x) dx \quad (4.35)$$

$$\bar{\varepsilon}(\xi) = \frac{i}{2}[\xi \otimes \bar{u}(\xi) + \bar{u}(\xi) \otimes \xi] \quad (4.36)$$

$$\varepsilon^*(\xi) = \int_D \varepsilon^*(x) \exp(-i\xi x) dx \quad (4.37)$$

Combining equations (4.30) and (4.31) gives:

$$\text{div}(C(\varepsilon - \varepsilon^*)) = 0 \quad \text{in} \quad D \quad (4.38)$$

Then by using equations (4.33), (4.36) and (4.34) in (4.38), the following expressions are obtained:

$$-\xi \cdot C(\xi \otimes \bar{u}(\xi)) = i\xi \cdot C\varepsilon^*(\xi) \text{ for every } \xi \neq 0 \quad (4.39)$$

where the symbols  $\otimes$  and  $\cdot$  represent the outer and the inner products, respectively [Spiegel 1959]. Thus, since  $C$  represents the elastic tensor of the matrix, the coefficients  $\bar{u}(\xi)$  are obtained uniquely in terms of the  $\varepsilon^*(\xi)$  in the following manner:

$$\bar{u}(\xi) = -i(\xi \cdot C \cdot \xi)^{-1} \cdot \xi \cdot C\varepsilon^*(\xi) \text{ for every } \xi \neq 0 \quad (4.40)$$

and from equation (4-36), the Fourier coefficients of the corresponding strain are:

$$\bar{\varepsilon}(\xi) = \text{sym}(\xi \otimes (\xi \cdot C \cdot \xi)^{-1} - 1 \otimes \xi) : C\varepsilon^*(\xi) \text{ for every } \xi \neq 0 \quad (4.41)$$

Denoting:

$$P'(\xi) = \text{sym}(\xi \otimes (\xi \cdot C \cdot \xi)^{-1} \otimes \xi) \quad (4.42)$$

the actual strain inside the inclusion from equation (4-41) using equations (4-33) and (4-37) is:

$$\varepsilon(x) = \frac{1}{V} \sum_{\xi}^{\pm\infty} P'(\xi) : C \int_D \varepsilon^*(x) \exp(-i\xi(x' - x)) dx' \quad (4.43)$$

where a prime on the sum indicates that  $\xi = 0$  is excluded in the summation.

Now, note that the exact expression of the strain tensor  $\varepsilon(x)$  is not necessary to obtain the overall elastic tensor  $C^*$  but only its volume average on  $\Omega$ , denoted by  $(\bar{\varepsilon} = \int_{\Omega} \varepsilon(x) dx / V_{\Omega})$ :

$$\bar{\varepsilon} = \frac{1}{V} \sum_{\xi}^{\pm\infty} P'(\xi) : C \left( \frac{g_0(\xi)}{V_{\Omega}} \right) \int_D \varepsilon^*(x) \exp(-i\xi x') dx' \quad (4-44)$$

where  $V_{\Omega}$  is the volume of the inclusion and:

$$g_0(\xi) = \int_{\Omega} \exp(i\xi x) dx \quad (4-45)$$

A good approximation of equation (4-44) is obtained when a constant  $\varepsilon^*$  is considered in  $\Omega$ . Then, replacing  $\varepsilon^*$  with its volume average  $\bar{\varepsilon}^*$ , equation (4-44) becomes:

$$\bar{\varepsilon} = \frac{1}{V} \sum_{\xi}^{\pm\infty} P'(\xi) : C \left( \frac{g_0(\xi)g_0(-\xi)}{V_{\Omega}} \right) \bar{\varepsilon}^* \quad (4-46)$$

or,

$$\bar{\varepsilon} = f \sum_{\xi}^{\pm\infty} \left( \frac{g_0(\xi)}{V_{\Omega}} \right) \left( \frac{g_0(-\xi)}{V_{\Omega}} \right) P'(\xi) : C : \bar{\varepsilon}^* \quad (4-47)$$

where  $f$  is the volume fraction of the inclusion, and by denoting:

$$t(\xi) = f \left( \frac{g_0(\xi)}{V_{\Omega}} \right) \left( \frac{g_0(-\xi)}{V_{\Omega}} \right) \quad (4-48)$$

and,

$$P = \sum_{\xi}^{\pm\infty} t(\xi) P'(\xi) \quad (4-49)$$

By substitute equations (4-34), (4-35) into (4-33) gives:

$$\bar{\varepsilon} = P : C : \bar{\varepsilon}^* \text{ in } \Omega \quad (4-50)$$

Equation (4-50) represents the relation between the volume average of the strain inside the inclusion  $\bar{\varepsilon}$  and the volume average of the eigenstrain  $\bar{\varepsilon}^*$ .

In order to obtain the homogenization eigenstrain, which simulates the presence of the periodic inclusions inside the body, consider an average strain tensor  $\bar{\varepsilon}_0$  in the unit cell, which is arbitrarily prescribed. In this hypothesis, the following average consistency condition [equivalent eigenstrain method of Nemat-Nasser and Hori 1993] can be used:

$$C' : (\bar{\varepsilon}_0 + P : C : \bar{\varepsilon}^*) = C : (\bar{\varepsilon}_0 + (P : C - I^{(4)}) : \bar{\varepsilon}^*) \quad (4-51)$$

where  $C'$  is the elastic tensor of the inclusion and  $(I^{(4)})$  is the identity fourth-order tensor. Observe that the tensor  $P$  takes into account the geometry of the inclusion and can be evaluated once and for all. Then from equation (4-51), the equivalent average volume eigenstrain  $\bar{\varepsilon}^*$  can be solved in terms of the tensors  $C'$ ,  $C$ ,  $P$  and  $\bar{\varepsilon}_0$  as :

$$\bar{\varepsilon}^* = [((C - C')^{-1} - P)C]^{-1} \bar{\varepsilon}_0 \quad (4-52)$$

since in this case the uniform overall stress  $\sigma_0$  in the unit cell is :

$$C^* : \bar{\varepsilon}_0 = C : (\bar{\varepsilon}_0 - f\bar{\varepsilon}^*) \quad (4-53)$$

Using equation (4-52) and noting that  $\bar{\varepsilon}_0$  is arbitrary, the following expression for the overall stiffness tensor of the composite material is obtained:

$$C^* = C - f((C - C')^{-1} - P)^{-1} \quad (4-54)$$

It is worth noting that the evaluation of  $C^*$  [equation (4-54)] involves the inversion of a symmetric tensor, since  $P$ ,  $C$  and  $C'$  are all symmetric tensors. In particular, if the matrix is isotropic, denoting by  $\bar{\xi} = \xi/|\xi|$ , the tensor  $P$  is:

$$P = \frac{1}{\mu_0} \sum_{\xi} t(\xi) \left( \text{sym}(\bar{\xi} \otimes I^{(2)} \otimes \bar{\xi}) - \frac{1}{2(1-\nu_0)} (\bar{\xi} \otimes \bar{\xi} \otimes \bar{\xi} \otimes \bar{\xi}) \right) \quad (4-55)$$

where  $\mu_0$  and  $\nu_0$  are the shear modulus and Poisson's ratio of the matrix, respectively and  $I^{(2)}$  is the identity second-order tensor. Hence, when the matrix and the inclusion are both isotropic, equation (4-54) can be written:

$$C^* = \lambda_0 I^{(2)} \otimes I^{(2)} + 2\mu_0 I^{(4)} - f[(\lambda_0 - \lambda_1)I^{(2)} \otimes I^{(2)} + 2(\mu_0 - \mu_1)I^{(4)}]^{-1} \\ - \frac{1}{\mu_0} \sum_{\xi}^{\pm\infty} t(\xi) \left[ \text{sym}(\bar{\xi} \otimes I^{(2)} \otimes \bar{\xi}) - \frac{1}{2(1-\nu_0)} (\bar{\xi} \otimes \bar{\xi} \otimes \bar{\xi} \otimes \bar{\xi}) \right]^{-1} \quad (4-56)$$

Here  $\mu_0, \lambda_0, \mu_1$  and  $\lambda_1$  are Lamé constants of the matrix and the inclusion respectively. Then, defining the following series  $S_l$  (with  $l = 1-9$ ) as:

$$S_1 = \sum_{\xi}^{\pm\infty} t(\xi) \bar{\xi}_1^2, \quad S_2 = \sum_{\xi}^{\pm\infty} t(\xi) \bar{\xi}_2^2, \\ S_3 = \sum_{\xi}^{\pm\infty} t(\xi) \bar{\xi}_3^2, \quad S_4 = \sum_{\xi}^{\pm\infty} t(\xi) \bar{\xi}_1^4, \\ S_5 = \sum_{\xi}^{\pm\infty} t(\xi) \bar{\xi}_2^4, \quad S_6 = \sum_{\xi}^{\pm\infty} t(\xi) \bar{\xi}_3^4, \\ S_7 = \sum_{\xi}^{\pm\infty} t(\xi) \bar{\xi}_2^2 \bar{\xi}_3^2, \quad S_8 = \sum_{\xi}^{\pm\infty} t(\xi) \bar{\xi}_1^2 \bar{\xi}_3^2, \\ S_9 = \sum_{\xi}^{\pm\infty} t(\xi) \bar{\xi}_1^2 \bar{\xi}_2^2 \quad (4-57)$$

the final expressions of the non-zero components of the tensor  $C^*$  can be written as follows:

$$C_{11}^* = \lambda_0 + 2\mu_0 - f \left( \frac{S_3 S_2}{\mu_0^2} - \frac{S_5 S_3 + S_6 S_2}{\mu_0^2 g} - \frac{a(S_2 + S_3)}{2\mu_0 c} + \frac{S_6 S_5 - S_7^2}{\mu_0^2 g^2} \right. \\ \left. + \frac{a(S_5 + S_6) + 2bS_7}{2\mu_0 g c} + \frac{a^2 - b^2}{4c^2} \right) / D$$

$$C_{12}^* = \lambda_0 + f \left( \left( -\frac{S_9}{\mu_0^2 g} + \frac{b}{2c\mu_0} \right) S_3 + \frac{S_9 S_6 - S_8 S_7}{\mu_0^2 g^2} - \frac{b(S_6 + S_7) - bS_8 - aS_9}{2c\mu_0 g} \right. \\ \left. - \frac{ba + b^2}{4c^2} \right) / D$$

$$\begin{aligned}
C_{13}^* &= \lambda_0 - f\left(\left(\frac{S_8}{\mu_0^2 g} + \frac{b}{2c\mu_0}\right)S_2 + \frac{S_8 S_5 - S_9 S_7}{\mu_0^2 g^2} + \frac{b(S_5 + S_7) - aS_8 - bS_9}{2c\mu_0 g} \right. \\
&\quad \left. + \frac{ab + b^2}{4c^2}\right) / D \\
C_{22}^* &= \lambda_0 + 2\mu_0 - f\left(\frac{S_3 S_1}{\mu_0^2} - \frac{S_4 S_3 + S_6 S_1}{\mu_0^2 g} - \frac{a(S_1 + S_3)}{2\mu_0 c} + \frac{S_6 S_4 - S_8^2}{\mu_0^2 g^2} \right. \\
&\quad \left. + \frac{a(S_4 + S_6) + 2bS_8}{2\mu_0 g c} + \frac{a^2 - b^2}{4c^2}\right) / D \\
C_{33}^* &= \lambda_0 + 2\mu_0 - f\left(\frac{S_2 S_1}{\mu_0^2} - \frac{S_4 S_2 + S_5 S_1}{\mu_0^2 g} - \frac{a(S_1 + S_2)}{2\mu_0 c} + \frac{S_5 S_4 - S_9^2}{\mu_0^2 g^2} \right. \\
&\quad \left. + \frac{a(S_5 + S_4) + 2bS_9}{2\mu_0 g c} + \frac{a^2 - b^2}{4c^2}\right) / D \\
C_{23}^* &= \lambda_0 + f\left(\left(-\frac{S_7}{\mu_0^2 g} + \frac{b}{2c\mu_0}\right)S_1 + \frac{S_7 S_4 - S_9 S_8}{\mu_0^2 g^2} - \frac{b(S_4 - S_8 - S_9) - aS_7}{2c\mu_0 g} \right. \\
&\quad \left. - \frac{ab + b^2}{4c^2}\right) / D \\
C_{44}^* &= \mu_0 - f\left(-\frac{S_2}{\mu_0} - \frac{S_3}{\mu_0} + (\mu_0 - \mu_1)^{-1} + \frac{4S_7}{\mu_0(2 - 2\nu_0)}\right)^{-1} \\
C_{55}^* &= \mu_0 - f\left(-\frac{S_1}{\mu_0} - \frac{S_3}{\mu_0} + (\mu_0 - \mu_1)^{-1} + \frac{4S_8}{\mu_0(2 - 2\nu_0)}\right)^{-1} \\
C_{66}^* &= \mu_0 - f\left(-\frac{S_1}{\mu_0} - \frac{S_2}{\mu_0} + (\mu_0 - \mu_1)^{-1} + \frac{4S_9}{\mu_0(2 - 2\nu_0)}\right)^{-1} \tag{4-58}
\end{aligned}$$

where:

$$\begin{aligned}
D = & -\frac{S_3 S_2 S_1}{\mu_0^3} + \frac{(S_6 S_2 + S_6 S_2 + S_6 S_2) S_1}{\mu_0^3 g} + \frac{a(S_1 S_2 + (S_1 + S_2) S_3)}{\mu_0^2 c} \\
& + \frac{(S_5 S_4 + S_7^2) S_1 + (S_6 S_4 + S_8^2) S_2 + (S_5 S_4 + S_9^2) S_3}{\mu_0^3 g^2} \\
& - \frac{(aS_5 + aS_6 + 2bS_7^2) S_1 + (aS_4 + aS_6 + 2bS_8^2) S_2 + (aS_4 + aS_5 + 2bS_9^2) S_3}{2\mu_0^2 gc} \\
& + \frac{(b^2 - a^2)}{4\mu_0 c^2} (S_1 + S_2 + S_3) + \frac{(S_5 S_6 - S_7^2) S_4 - S_8^2 S_5 - S_9^2 S_6 - 2S_8 S_9 S_7}{\mu_0^3 g^3} \\
& + \frac{(aS_5 + aS_6 + 2bS_7) S_4 + (aS_7 + 2bS_8 + 2bS_9) S_7 + (2bS_5 + aS_8 + 2bS_9) S_8 - aS_9^2 + (2bS_9 + aS_5) S_6}{2\mu_0^2 g^2 c} \\
& + \frac{a(aS_4 + aS_5 + aS_6 + 2(bS_7 + bS_8 + bS_9))}{4\mu_0 g c^2} + \frac{d(2(S_7 + S_8 + S_9) - (S_4 + S_5 + S_6))}{4} \\
& + \frac{a^3 - 3ab^2 - 2b^3}{8c^3}
\end{aligned} \tag{4-59}$$

and,

$$\begin{aligned}
a &= \mu_1 - \mu_0 - 2\mu_1 \nu_0 + 2\mu_0 \nu_1 \\
b &= -\mu_0 \nu_0 + \mu_1 \nu_1 + 2\mu_0 \nu_0 \nu_1 - 2\mu_1 \nu_0 \nu_1 \\
c &= (\mu_0 - \mu_1)(-\mu_0 + \mu_1 - \mu_0 \nu_0 - 2\mu_1 \nu_0 + 2\mu_0 \nu_1 + \mu_1 \nu_1 + 2\mu_0 \nu_0 \nu_1 - 2\mu_1 \nu_0 \nu_1) \\
d &= b^2 / (\mu_0 g c^2) \\
g &= (2 - 2\nu_0)
\end{aligned} \tag{4-60}$$

In the case of a composite material reinforced by long cylindrical fibers, five series differ from zero and only three are independent. For unidirectional fibers aligned with the  $x_1$  axis, the tensor  $\varepsilon^*(x)$  is constant in the  $x_1$ -direction, therefore the Fourier series of  $\varepsilon^*(x)$  in the  $x_1$ -direction reduces to a constant. Then, for the case of fibers aligned with the  $x_1$ -axis:

$$\begin{aligned}
S_1 &= S_4 = S_8 = S_9 = 0 \\
S_2 &= S_3, \quad S_5 = S_6
\end{aligned} \tag{4-61}$$

Therefore the earlier expressions are eliminated to give the following formulas to evaluate the stiffness tensor of a uni-directional composite with periodic microstructure:



$$C_{11}^* = \lambda_0 + 2\mu_0 - f \left[ \frac{S_3^2}{\mu_0^2} - \frac{2S_6S_3}{\mu_0^2g} - \frac{aS_3}{\mu_0c} + \frac{S_6^2 - S_7^2}{\mu_0^2g^2} + \frac{aS_6 + bS_7}{\mu_0gc} + \frac{a^2 - b^2}{4c^2} \right] / D$$

$$C_{12}^* = \lambda_0 + fb \left[ \frac{S_3}{2c\mu_0} - \frac{S_6 - S_7}{2c\mu_0g} - \frac{a+b}{4c^2} \right] / D$$

$$C_{23}^* = \lambda_0 + f \left[ \frac{aS_7}{2\mu_0gc} - \frac{ba + b^2}{4c^2} \right] / D$$

$$C_{22}^* = \lambda_0 + 2\mu_0 - f \left[ -\frac{aS_3}{2\mu_0c} + \frac{aS_6}{2\mu_0gc} + \frac{a^2 - b^2}{4c^2} \right] / D$$

$$C_{44}^* = \mu_0 - f \left[ -\frac{2S_3}{\mu_0} + (\mu_0 - \mu_1)^{-1} + \frac{4S_7}{\mu_0(2 - 2\nu_0)} \right]^{-1}$$

$$C_{66}^* = \mu_0 - f \left[ -\frac{S_3}{\mu_0} + (\mu_0 - \mu_1)^{-1} \right]^{-1} \quad (4-62-a)$$

Where:

$$D = \frac{aS_3^2}{2\mu_0^2c} - \frac{aS_6S_3}{\mu_0^2gc} + \frac{a(S_6^2 - S_7^2)}{2\mu_0^2g^2c} + \frac{S_3(b^2 - a^2)}{2\mu_0c^2} + \frac{S_6(a^2 - b^2) + S_7(ab + b^2)}{2\mu_0gc^2} + \frac{(a^3 - 2b^3 - 3ab^2)}{8c^3} \quad (4-62-b)$$

and,

$$\begin{aligned} a &= \mu_1 - \mu_0 - 2\mu_1\nu_0 + 2\mu_0\nu_1 \\ b &= -\mu_0\nu_0 + \mu_1\nu_1 + 2\mu_0\nu_0\nu_1 - 2\mu_1\nu_0\nu_1 \\ c &= (\mu_0 - \mu_1)(-\mu_0 + \mu_1 - \mu_0\nu_0 - 2\mu_1\nu_0 + 2\mu_0\nu_1 + \mu_1\nu_1 + 2\mu_0\nu_0\nu_1 - 2\mu_1\nu_0\nu_1) \\ g &= (2 - 2\nu_0) \end{aligned} \quad (4-62-c)$$

where the series  $S_3, S_6, S_7$  are evaluated numerically for each value of the fiber fraction, however; it is convenient to use the simplified suggestion of Luciano et al [1994] for the three series, in relation with the fiber fraction, as:

$$\begin{aligned}
S_3 &= 0.49247 - 0.47603f - 0.02748f^2 \\
S_6 &= 0.36844 - 0.14944f - 0.27152f^2 \\
S_7 &= 0.12346 - 0.32035f - 0.23517f^2
\end{aligned} \tag{4-62-d}$$

For a transversely isotropic material the stiffness tensor  $C^*$  of a uni-directional composite with periodic microstructure characterizes an orthotropic material with square symmetry. Hence the directions  $X_2$  and  $X_3$  are equivalent and the stiffness tensor is unchanged by a rotation about  $X_1$  of  $n\pi/2$  ( $n = 0, \pm 1, \pm 2 \dots$ ). These results, with only six components, are required to describe the tensor completely.

The following averaging procedure [Aboudi, 1991] is used to achieve a transversely isotropic stiffness tensor, equivalent in the average sense to the stiffness tensor with square symmetry

$$B(\theta) = Q(\theta)C^*Q^T(\theta) \tag{4-63}$$

Where  $Q(\theta)$  is the fourth-order orthogonal rotation tensor. Then the equivalent transversely isotropic tensor is given as:

$$\bar{B} = \frac{1}{\pi} \int_0^\pi B(\theta) d\theta \tag{4-64}$$

The following expressions are found explicitly in terms of the coefficients of tensor  $C^*$  employing the relations between the engineering constants and the components of the  $\bar{B}$  tensor:

$$\begin{aligned}
E_1 &= C_{11}^* - \frac{2C_{12}^{*2}}{C_{22}^* + C_{23}^*} \\
E_2 &= \frac{(2C_{11}^*C_{22}^* + 2C_{11}^*C_{23}^* - 4C_{12}^{*2})(C_{22}^* - C_{23}^* + 2C_{44}^*)}{3C_{11}^*C_{22}^* + C_{11}^*C_{23}^* + 2C_{11}^*C_{44}^* - 4C_{12}^{*2}} \\
G_{12} &= C_{66}^* \\
\nu_{12} &= \frac{C_{12}^*}{C_{22}^* + C_{23}^*} \\
\nu_{21} &= \frac{C_{11}^*C_{22}^* + 3C_{11}^*C_{23}^* - 2C_{11}^*C_{44}^* - 4C_{12}^{*2}}{3C_{11}^*C_{22}^* + C_{11}^*C_{23}^* + 2C_{11}^*C_{44}^* - 4C_{12}^{*2}}
\end{aligned} \tag{4-65}$$

### 4.3.2 Discussion and Results:

The Luciano method explained earlier gives the modulus of elasticity in the fiber direction ( $E_1$ ), transverse direction ( $E_2$ ) and in-plane shear modulus ( $G_{12}$ ) of the uni-directional fiber alignment lamina that agree very well with other methods and the experimental results, [Luciano et al 1994, Davalos et al 1996, and others]. It also calculates both in-plane Poisson's ratios ( $\nu_{12}$  and  $\nu_{21}$ ), while traditional methods such as (Halpin-Tsai) method gives only ( $\nu_{12}$ ). However, the result for  $\nu_{12}$  calculated using the Luciano method is higher than that calculated by the Halpin-Tsai method and is sometimes higher than the Poisson's ratios of the matrix and the fiber. Depending on the ratio between the modulus of elasticity of the fiber and that of the matrix, i.e.  $(E_{fib} / E_{mat})$ , the results of the two methods become close to each other.

For  $(E_{fib} / E_{mat}) \geq 21$  the results for ( $\nu_{12}$ ) range between  $\nu_{12mat}$  and  $\nu_{12fib}$ , while with a lower  $(E_{fib} / E_{mat})$  ratio,  $\nu_{12}$  becomes higher than that of the matrix or the fiber in general. Figures 4.6 and 4.7 show the results of  $\nu_{12}$  and  $\nu_{21}$  calculated by the Luciano method and ( $\nu_{12}$ ) calculated by the Halpin-Tsai method for fiber fraction  $(0.1 \leq f \leq 0.8)$  and for  $(E_{fib} / E_{mat}) = 19$  and 14.3 respectively. Figure 4.8 shows the results of  $\nu_{12}$  and  $\nu_{21}$  for  $(E_{fib} / E_{mat}) = 21.2$  reported by Luciano et.al. [1994].

In this research, the ratio of  $(E_{fib} / E_{mat})$  for the E-glass fiber and Vinylester Matrix is relatively low, and the calculated lamina composite material moduli using the Luciano method match the experimental results given by Davalos et al [1996] for fiber fraction  $(f = 0.353)$ , and by Aref et al [2000] for fiber fraction  $(f = 0.47)$ . The calculated material moduli also agree with those calculated using the Halpin-Tsai method. However, the calculated values for Poisson's ratios for different fiber fractions are high in general and for conservative evaluation, it is decided to use the Halpin-Tsai method to calculate Poisson's ratios only. Table 4.3 gives the results for elastic moduli (calculated by Luciano method) and in-plane Poisson's ratio calculated by the Halpin-Tsai method of E-glass fiber and Vinylester Matrix laminas, and for  $(0.25 \leq f \leq 0.5)$ . It is also important

here to mention that the calculated properties in Table 4.3 are only used for investigating the effect of fiber fraction of the lamina-material on the bridge performance. For the rest of the performance study and for all the investigated static and dynamic cases, the material of the lamina is taken as that calculated for a fiber fraction of ( $f = 0.47$ ). The calculated and the experimental properties for this material are almost similar, and also the experimental results for the strength of this specific material are available as many researchers had tested and used it [ Zweben et al, 1990, and Aref et al, 2000]. Table 4.4 shows the mechanical properties and strength of the lamina material properties, which are used in the performance study (Chapter 6), except of course, for the case of studying the effect of the fiber fraction on the bridge system performance.

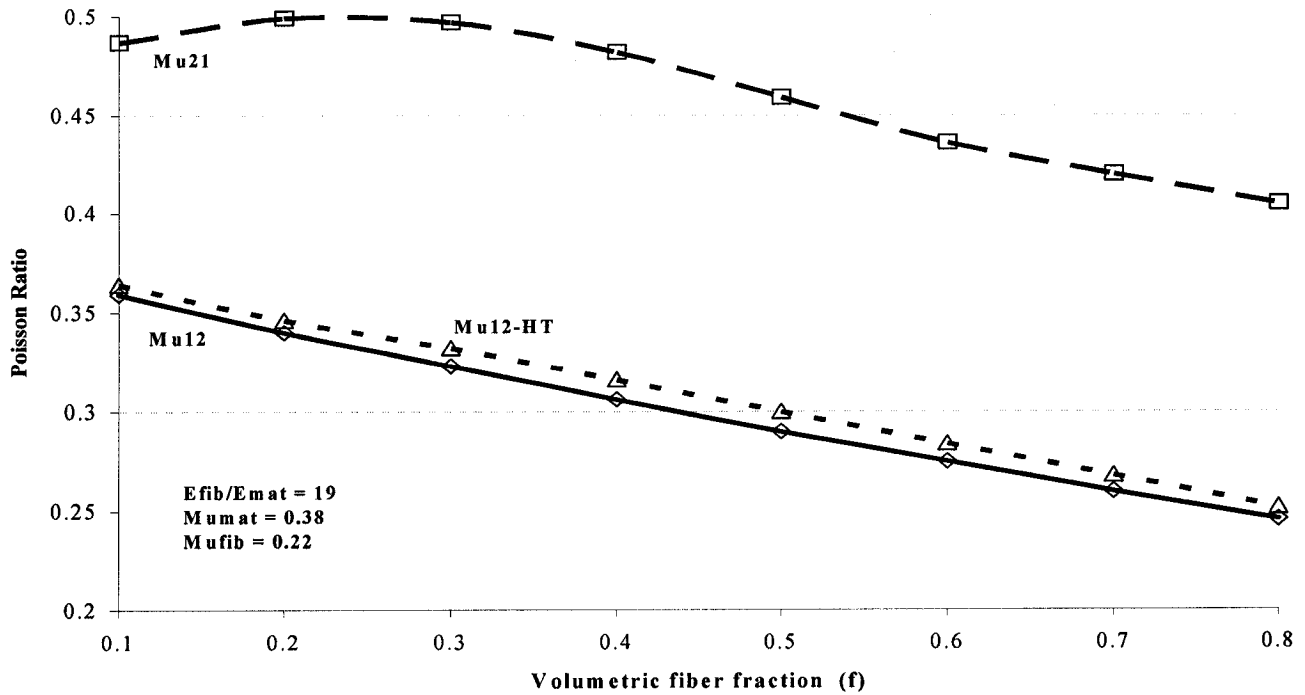


Figure 4.6 Poisson's ratios  $\nu_{12}$  &  $\nu_{21}$  of Luciano method compared with  $\nu_{12}$  of Halpin-Tsai method for  $E_{fiber} / E_{mat} = 19.0$

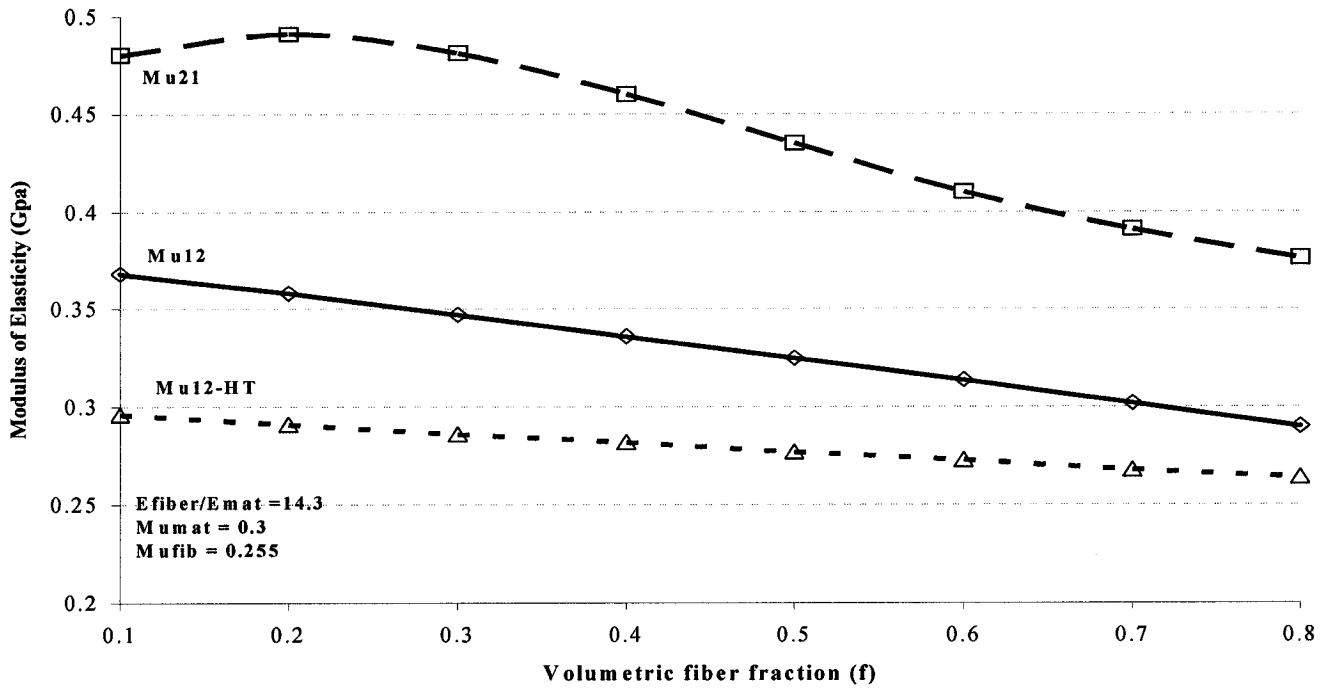


Figure 4.7 Poisson's ratios  $\nu_{12}$  &  $\nu_{21}$  of Luciano method compared with  $\nu_{12}$  of Halpin-Tsai method for  $E_{fiber}/E_{mat} = 14.3$

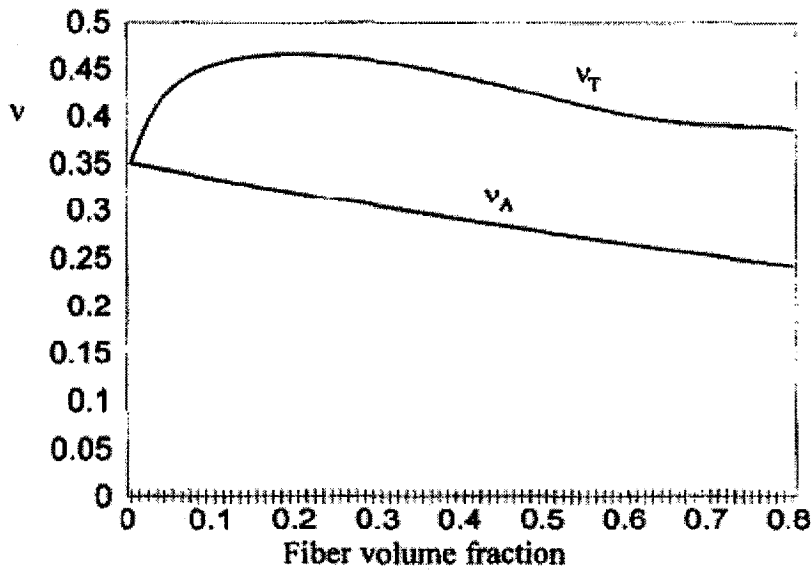


Figure 4.8 Poisson's ratios  $\nu_A = \nu_{12}$  &  $\nu_T = \nu_{21}$  of Luciano method, for  $E_{fiber}/E_{mat} = 21.2$ , [after Luciano et al 1994].

| <b>Fiber fraction (f)</b> | <b>E<sub>1</sub> (GPa)</b> | <b>E<sub>2</sub> (GPa)</b> | <b><math>\nu_{12}</math> **</b> | <b>G<sub>12</sub> (GPa)</b> |
|---------------------------|----------------------------|----------------------------|---------------------------------|-----------------------------|
| 0.25                      | 21.59                      | 7.188                      | 0.289                           | 2.58                        |
| 0.353*                    | 28.63                      | 8.794                      | 0.284                           | 3.147                       |
| 0.4                       | 31.83                      | 9.916                      | 0.282                           | 3.461                       |
| 0.45                      | 35.24                      | 11.75                      | 0.28                            | 3.843                       |
| 0.47                      | 36.61***                   | 12.83***                   | 0.279***                        | 4.013***                    |
| 0.5                       | 38.65                      | 13.52                      | 0.277                           | 4.289                       |

\* Same properties as given by Davalos et al [1996].

\*\* The in-plane Poisson's ratio  $\nu_{12}$  is calculated here using Halpin-Tsai method.

\*\*\* Very close to the experimental results given by Zweben et al [1990] and Aref et al [2000].

Table 4.3 The mechanical properties of the lamina formed from E-Glass fiber and Vinylester Matrix (GFVM) for different fiber fractions.

|                                                          |                  |
|----------------------------------------------------------|------------------|
| <b>Longitudinal modulus of elasticity, E<sub>1</sub></b> | <b>36.61 GPa</b> |
| <b>Transverse modulus of elasticity, E<sub>2</sub></b>   | <b>12.83 GPa</b> |
| <b>In-Plane Poisson Ratio, <math>\nu_{12}</math></b>     | <b>0.279</b>     |
| <b>In-Plane shear modulus</b>                            | <b>4.013 GPa</b> |
| <b>Longitudinal Tensile strength</b>                     | <b>1020 MPa</b>  |
| <b>Longitudinal Compressive strength</b>                 | <b>620 MPa</b>   |
| <b>Transverse Tensile strength</b>                       | <b>40 MPa</b>    |
| <b>Transverse compressive strength</b>                   | <b>140 MPa</b>   |
| <b>In-Plane shear strength</b>                           | <b>70 MPa</b>    |

Table 4.4 The material properties of the lamina that are used in the performance study, which also match the experimental results given by [ Zweben et al, 1990, and Aref et al, 2000].

## **Chapter 5**

# **Finite Element Modeling in the Structure Level, and Aerodynamic Stability of the HLSCSB System**

# Chapter 5

## Finite Element Modeling in the Structure Level, and Aerodynamic Stability of the HLSCSB System

### 5.1 Introduction:

In this chapter, two separated steps in the analysis and design procedure are presented; (i) the finite element modeling of the Hybrid Long Span Cable Stayed Bridge (HLSCSB) system, and (ii) the aerodynamic stability.

Three-dimensional non-linear finite element modeling is involved in this research using “detailed geometrical copy” of the bridge system components. As shown in the previous chapters, the bridge system includes laminated composite materials in the deck, which have anisotropic properties, and chopped FRP. Consequently, the four bridge deck models are simulated using shell finite elements and solid elements according to each deck section detail. The cable stays are modeled using isoparametric beam elements while the pylons are modeled by a solid element.

The aerodynamic stability investigation in this research is limited to the numerical modeling of aerodynamic forces to evaluate the flutter instability response of the proposed long span hybrid cable-stayed bridge. In order to involve nonlinearities in the structural origins, a time domain scheme is adopted and shown here.



## **5.2 Structural Level Finite Element Modeling of the Hybrid Long Span Cable Stayed Bridge:**

It is required, in this step, to simulate the proposed bridge system numerically at the structural level using the Deterministic Finite Element Method (DFEM). Many complications are overcome in this modeling process such as: (i) the system has huge dimensions (recall the three reference bridge dimension shown in Chapter 3), which results in a huge number of elements, nodes, integration points, ...etc, (ii) the need for three dimensional simulation of the deck section added more complications, and further increased the size of the numerical model. It also requires special treatments, especially with the complicated sections of Models III and IV, (iii) the apparent anisotropy of the laminated composite complicates the calculations of the shell elements stiffness matrices and further increases the running time, (iv) the orientation of the different laminas and their relation with the laminate orientation in the structure, in addition, including the interlaminar shear in the laminate further extends the calculations in the element level, (v) All the bridge components are affected by in-plane or direct stresses (tension or compression), and (vi) the nonlinearity of the system has to be included both in the static and dynamic analyses.

ABAQUS advanced finite element program is used to model the bridge system at the structural level, taking into account all the material and geometrical characteristics of the system and the resulting complications mentioned earlier. Shell elements are used to model the deck of Models I, II, III. Solid elements are used to represent the pylons and Model IV deck (except the tubular and the stiffened laminates, which are modeled using shell elements). The stay cables are modeled using beam elements. The following is a brief description of the structural-modeling techniques involved in this research.

### **5.2.1 Shell Finite Element Versus Continuum Shell Finite Element Modeling of Laminated Composite Shells:**

Two finite element modeling procedures are available to simulate the shell structures; the conventional shell finite element or simply “shell element” and the continuum shell finite element or “continuum shell element”. Both procedures have been well developed for the

modeling of laminated composites and are available in the ABAQUS program elements library. Shell structures have one of their dimensions, the thickness, significantly smaller than the other dimensions, and shell elements use this condition to discretize a body by defining the geometry at the reference surface. The thickness is defined through the section property definition, and the elements have displacements and rotational degrees of freedom. On the other hand, Continuum shell elements discretize an entire three dimensional body. The thickness is calculated from the element nodal geometry. Similar to three-dimensional continuum solid elements, from the modeling point of view, continuum shell elements have only displacement degrees of freedom; however, their kinematics and constitutive behavior are analogous to the conventional shell elements. Figure 5.1 shows both finite element modeling procedures to simulate a shell structure.

The interlaminar shear in the laminated shells is of some concern and usually is accounted for in any formulation of the shell element by involving first order (or higher order, i.e. second or third order) transverse shear flexible theory. In the first order transverse shear flexible theory, the transverse shear strain is assumed to be constant through the laminated shell element thickness. It has been found that using first order transverse shear flexible theory is accurate and numerically efficient in the modeling of the laminated shell [Reddy et al, 1999, Reddy, 2004].

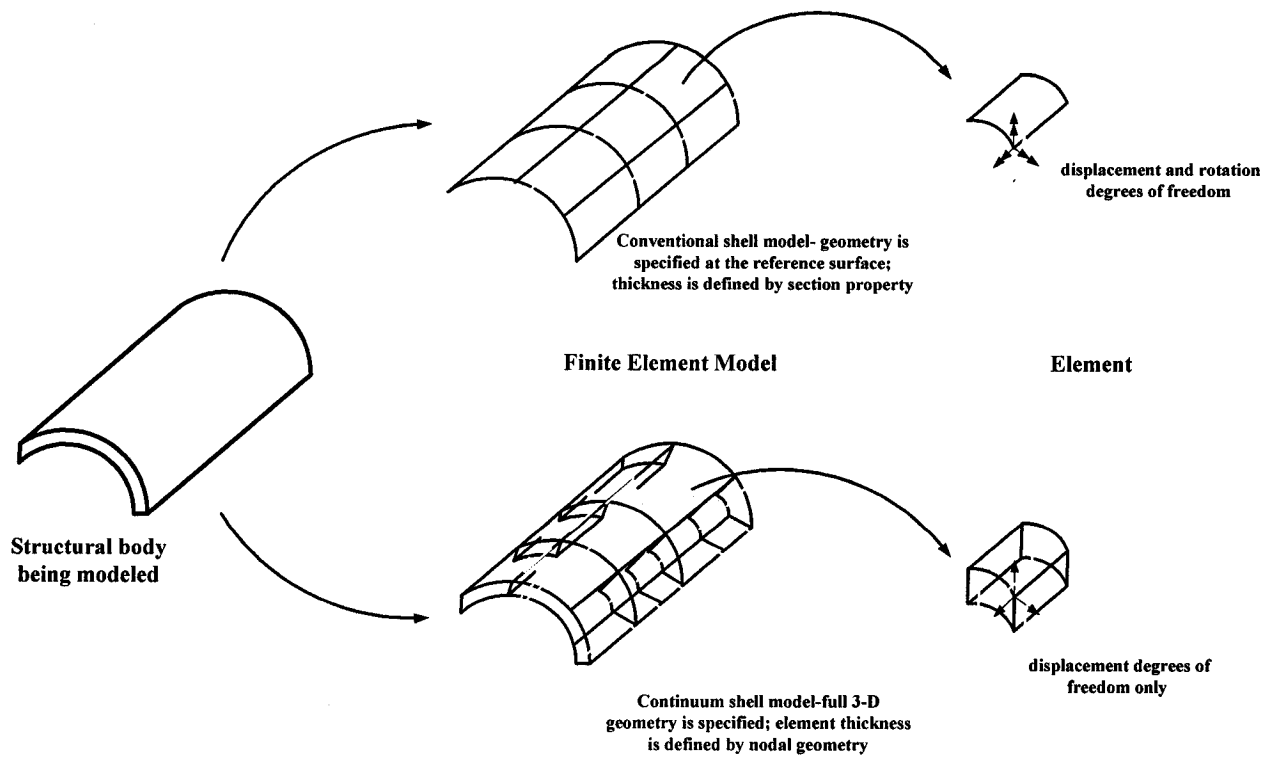


Figure 5.1 Finite Element Modeling of shell structure.

In this research, both modeling techniques have been investigated but the “shell element” is used for the bridge system performance study. The proposed hybrid long span cable-stayed bridge systems have enormous dimensions and the need for accurate geometrical simulation of the deck section in three-dimensions results in a huge number of finite elements, and hence a huge number of degrees of freedom. In addition, the system is non linear and its dynamic performance, which needs extensive computational time for acceptably accurate modeling, is of major concern. Consequently, the numerical efforts and the modeling-running time required for continuum shell finite element modeling are impractically huge, hence are excluded from use in the performance study of this research, although it is theoretically more accurate than conventional shell element modeling.

### **5.2.2 Laminated Composite Shell Section:**

A section of a laminated shell, made of one or more composite materials, can be defined by specifying the properties of its laminas and their sequence in relation to the shell geometry. Consequently, it is required to specify the thickness, the number of integration points (Figure 5.2), the material moduli, and the orientation of each lamina (either as a reference to an orientation definition of the laminate or as an angle measured relative to the overall reference orientation). The order of the laminas in the laminate is usually defined with respect to the positive direction of the laminated shell normal.

For continuum shell elements, the thickness is determined from the element geometry and may vary through the model for a given section definition. Hence, the specified thicknesses are only relative thicknesses for each layer. The actual thickness of a layer is the element thickness times the fraction of the total thickness that is accounted for by each layer. An example of a section with four layers and three integration points per layer is shown in Figure 5.2.

To account for the laminated cross section behavior for the shell element or the continuum shell element, the section is numerically integrated through the thickness during the analysis. This provides complete generality in material modeling, and any number of material points can be defined through the thickness so that the material response can vary from point to point. Indeed, this allows layers (or laminas) of different materials, in different orientations through the cross section, to be simulated.

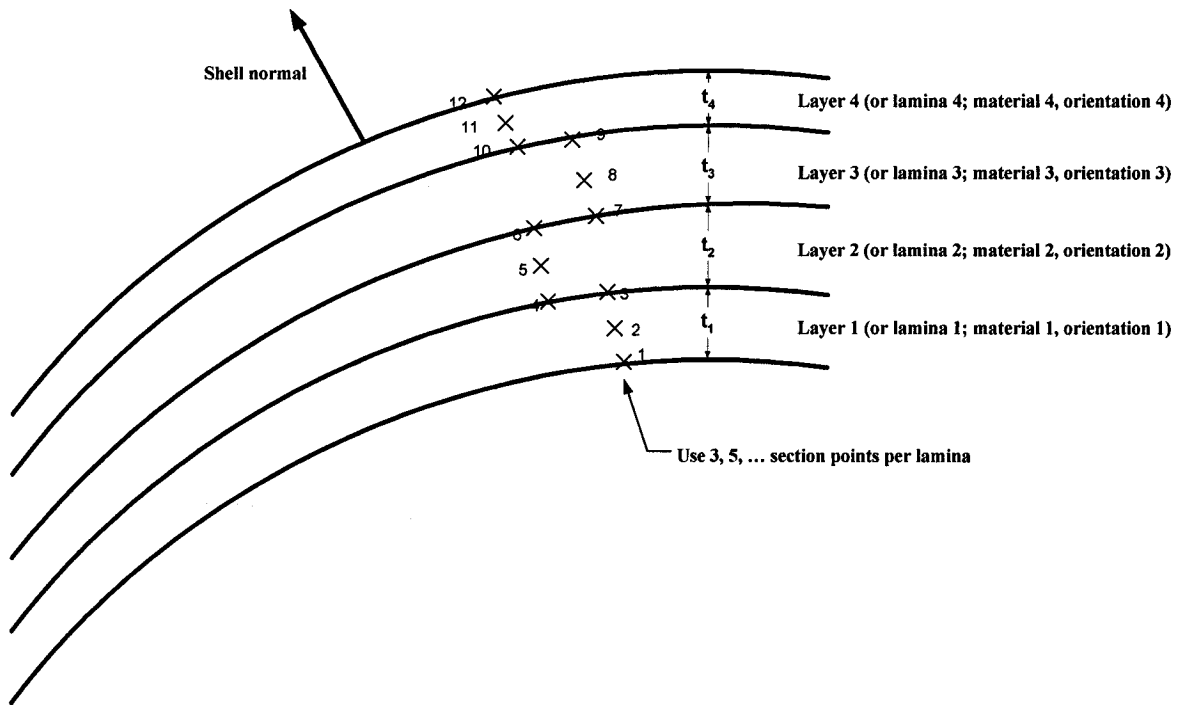


Figure 5.2 Example of composite shell section definition.

### 5.2.3 Laminated Shell Finite Element:

Two types of elements were used from the ABAQUS element library, namely S4R and S8R. Both shell elements use reduced (lower-order) integration to form the element stiffness. The mass matrix and distributed loadings are still integrated exactly. Reduced integration usually provides more accurate results and significantly reduces the running time [Cook, 1982], especially in the present three dimension modeling. The first element (S4R) is a general-purpose shell element, and provides a robust and accurate solution in all loading conditions for thin and thick shells [ABAQUS Theory Manual, 2005]. The element is a quadrilateral double-curved finite-membrane-strain element with four nodes, hourglass control and with the thickness change as a function of in-plane deformation being allowed in its formulation. The element offers significant advantages in computational speed and has been used in the performance study in Chapter 6 in all the cases, except where the laminate thickness is very high and the S8R element is involved. The second element is a quadrilateral shear flexible thick shell element with eight nodes. The formulations of both elements are given in the ABAQUS Theory Manual, hence it

will not be shown here; however, some important characteristics of the elements are discussed in the following sections:

### 5.2.3.1 Shell Element Cross-Section Thickness:

In geometrically nonlinear analyses in ABAQUS, the cross-section thickness of the shell elements changes as a function of the membrane strain based on the “effective section Poisson's ratio”, which is calculated as follows:

In plane stress  $\sigma_{33} = 0$ ; and linear elasticity gives

$$\varepsilon_{33} = -\frac{\nu}{1-\nu}(\varepsilon_{11} + \varepsilon_{22})$$

Treating these as logarithmic strains,

$$\ln\left(\frac{t}{t_0}\right) = -\frac{\nu}{1-\nu}\left(\ln\left(\frac{l_1}{l_1^0}\right) + \ln\left(\frac{l_2}{l_2^0}\right)\right) = -\frac{\nu}{1-\nu}\ln\left(\frac{A}{A^0}\right)$$

where  $A$  is the area on the shell's reference surface. This nonlinear analogy with linear elasticity leads to the thickness change relationship:

$$\frac{t}{t_0} = \left(\frac{A}{A_0}\right)^{-\frac{\nu}{1-\nu}}$$

### 5.2.3.2 Transverse Shear Stiffness in Composite Shell:

The ABAQUS elements, which are used in the performance investigation of the bridge system for simulating the laminated composite shells of the deck, are based on first-order transverse shear flexible theory, in which the transverse shear strain is assumed to be constant through the thickness of the shell. This assumption necessitates the use of shear correction factors. The development of these factors also provides a basis for estimating interlaminar shear stresses in a composite section. The development of the transverse shear stiffness is described here.

It is necessary to establish the transverse shear stiffness correction factors for laminated and sandwich constructions. For this purpose, one calculates the distribution of transverse shear stress through the thickness of the shell, for the case of unidirectional bending and

assuming linear elastic response. Then the shear strain energy, expressed in terms of section forces and strains, is equated to the strain energy of this distribution of transverse shear stresses.

The method, outlined below, provides an approximate analogy for calculating interlaminar shear stresses and gives reasonable estimates of transverse shear stiffness. In this calculation, it is assumed that the shell section directions are the principal bending directions (bending about one principal direction does not require a restraining moment about the other direction). For composite shells, with orthotropic layers that are not symmetric about the shell mid-surface, the shell section directions may not be the principal bending directions. In such cases, the transverse shear stiffness and interlaminar shear stresses are less accurate approximations and will change if different shell section directions are used.

Consider a shell that is plane in the  $x-y$ . Assuming only bending and shear in the  $x$ -direction, without gradients in the  $y$ -direction, then the membrane forces in the shell are zero, or  $N_{xx} = N_{yy} = N_{zz} = 0$ , and  $\partial/\partial y = 0$  for all response-variables. In this case, equilibrium within the section in the  $x$ -direction is;

$$\frac{\partial \sigma_{xx}}{\partial x} + \frac{\partial \tau_{xy}}{\partial z} = 0 \quad (5-1)$$

Moment equilibrium about the  $y$ -axis gives;

$$V_x + \frac{\partial M_{xx}}{\partial x} = 0 \quad (5-2)$$

where  $V_x$  is the transverse shear force per unit width in the shell and  $M_{xx}$  is the bending moment per unit width for bending about the  $y$ -axis.

For the bending behavior, assume the strain varies linearly across the section:

$$\varepsilon_{\alpha\beta} = \bar{\varepsilon}_{\alpha\beta} - (z - z_0) \bar{\kappa}_{\alpha\beta} \quad (5-3)$$

where  $\varepsilon_{\alpha\beta}$  is the membrane strain of the reference surface  $z = z_0$  and  $\bar{\kappa}_{\alpha\beta}$  is the curvature of that surface.

If the response of the shell is linear elastic, any “in-plane” component of stress at a point through the shell section is given by:

$$\sigma_{\alpha\beta} = D_{\alpha\beta\gamma\delta} \left( \bar{\varepsilon}_{\gamma\delta} - (z - z_0) \bar{\kappa}_{\gamma\delta} \right) \quad (5-4)$$

where the plane stress elastic stiffness,  $D_{\alpha\beta\gamma\delta}$ , is defined from the elasticity and orientation of the material at the particular layer of the shell. Greek subscripts here take the range (1, 2).

Integrating through the thickness and inverting the resultant section stiffness gives a  $6 \times 6$  section flexibility matrix,  $[H]$ :

$$\begin{Bmatrix} \bar{\varepsilon}_{\alpha\beta} \\ \bar{\kappa}_{\alpha\beta} \end{Bmatrix} = [H] \begin{Bmatrix} N_{\alpha\beta} \\ M_{\alpha\beta} \end{Bmatrix} \quad (5-5)$$

It is already assumed that;  $N_{xx} = N_{yy} = N_{xy}$ , and also assume that  $M_{yy} = M_{xy} = 0$ ; that is, it is possible to have no bending in the  $y$ -direction without any restraining moments associated with the  $y$ -direction. This is clearly not the case for an unbalanced composite section, but it is still used as a simplifying assumption to obtain the shear correction factors. Thus,

$$\begin{Bmatrix} \bar{\varepsilon}_{\alpha\beta} \\ \bar{\kappa}_{\alpha\beta} \end{Bmatrix} = \{H_{i4}\} M_{xx} \quad (5-6)$$

where  $\{H_{i4}\}$  is the fourth column of  $[H]$ . Combining this result with the elastic stiffness at a point through the shell thickness provides the in-plane stress components in terms of  $M_{xx}$  as

$$\sigma_{xx} = (B_{x1} - (z - z_0)B_{x2})M_{xx} \quad (5-7)$$

where

$$B_{x1} = D_{xxxx}H_{14} + D_{xxyy}H_{24} + D_{xxxy}H_{34} \quad (5-8)$$

and

$$B_{x2} = D_{xxxx}H_{44} + D_{xxyy}H_{54} + D_{xxxy}H_{64} \quad (5-9)$$



Combining the gradient of this equation in the  $x$ -direction with the equilibrium equations (5-1) and (5-2) yields a description of the variation of the transverse shear stress through the thickness of the plane shell:

$$\frac{\partial \tau_{xz}}{\partial z} = (B_{x1} - (z - z_0)B_{x2})V_x \quad (5-10)$$

In calculating  $\partial \sigma_{xx} / \partial x$ , it has been assumed that the elasticity and thickness of the composite section do not vary (or vary slowly) with position along the shell.

A laminated composite shell section consists of  $N$  layers (1,2,3,...) with different values of  $B_{x1}^1, B_{x2}^1$  at layer 1,  $B_{x1}^2, B_{x2}^2$  at layer 2, ...  $(B_{x1}^N, B_{x2}^N)$  at layer  $N$ . Layer ( $i$ ) extends from  $z_i$  to  $z_{i+1}$  and its thickness is  $t_i = z_{i+1} - z_i$ . Integrating Equation (5.10) through the shell, using the boundary conditions  $\tau_{xz} = 0$  at  $z = 0$ ,  $\tau_{xz}^i = \tau_{xz}^{i+1}$  at  $z = z_{i+1}$  and  $\tau_{xz} = 0$  at  $z = z_{N+1}$ , gives the transverse shear stress in layer ( $i$ ) as

$$\tau_{xz}^i = \left[ B_{x1}^i(z - z_i) - \left( \frac{1}{2}(z^2 - z_i^2) - z_{x0}(z - z_i) \right) B_{x2}^i + B_{x0}^i \right] V_x \quad (5-11)$$

where

$$B_{x0}^i = \sum_{j=1}^{i-1} t_j \left[ B_{x1}^j - \left( \frac{1}{2}(z_j + z_{j+1}) - z_{x0} \right) B_{x2}^j \right] \quad (5-12)$$

and

$$z_{x0} = \frac{\sum_{i=1}^N t_i \left( \frac{1}{2}(z_i + z_{i+1}) B_{x2}^i - B_{x1}^i \right)}{\sum_{i=1}^N t_i B_{x2}^i} \quad (5-13)$$

The subscript  $z_{x0}$  is used instead of  $z_0$  in this case because the result is associated with pure bending in the  $x$ -direction. The variation of  $\tau_{yz}$  through the shell thickness is obtained using a similar procedure, based on pure bending in the  $y$ -direction. These results provide the estimates of the interlaminar shear stresses.

The shear flexibility of the section is defined by matching the shear strain energy obtained and by integrating the elastic strain energy density associated with the transverse shear stress distribution obtained above:

$$\frac{1}{2} \begin{bmatrix} V_x & V_y \end{bmatrix} [F^s] \begin{Bmatrix} V_x \\ V_y \end{Bmatrix} = \frac{1}{2} \sum_{i=1}^N \int_{z_i}^{z_{i+1}} \begin{bmatrix} \tau_{xz} & \tau_{yz} \end{bmatrix} [F^i] \begin{Bmatrix} \tau_{xz} \\ \tau_{yz} \end{Bmatrix} dz \quad (5-14)$$

where  $[F^s]$  is the shear flexibility of the section and  $[F^i]$  is the continuum transverse shear flexibility within layer  $i$ . Here, the assumption that the transverse shear flexibility within a layer is not coupled to the in-plane flexibility is introduced. This is usually the case for shell constructions.

Substituting the relations for  $\tau_{xz}^i$  and  $\tau_{yz}^i$  into the above equation and integrating leads to a definition of the shear flexibilities of the section as:

$$F_{xx}^s = \sum_{i=1}^N F_{xx}^i t_i \left[ \begin{aligned} & \left( B_{x0}^i \right)^2 + t_i B_{x0}^i \left( B_{x1}^i - (z_i - z_{x0}) B_{x2}^i \right) + \frac{1}{3} t_i^2 \left( B_{x1}^i - (z_i - z_{x0}) B_{x2}^i \right)^2 \\ & - \frac{1}{4} t_i^3 B_{x2}^i \left( B_{x1}^i - (z_i - z_{x0}) B_{x2}^i \right)^2 + \frac{1}{20} t_i^4 (B_{x2}^i)^2 \end{aligned} \right] \quad (5-15)$$

$$F_{yy}^s = \sum_{i=1}^N F_{yy}^i t_i \left[ \begin{aligned} & \left( B_{y0}^i \right)^2 + t_i B_{y0}^i \left( B_{y1}^i - (z_i - z_{y0}) B_{y2}^i \right) + \frac{1}{3} t_i^2 \left( B_{y1}^i - (z_i - z_{y0}) B_{y2}^i \right)^2 \\ & - \frac{1}{4} t_i^3 B_{y2}^i \left( B_{y1}^i - (z_i - z_{y0}) B_{y2}^i \right)^2 + \frac{1}{20} t_i^4 (B_{y2}^i)^2 \end{aligned} \right] \quad (5-16)$$

$$F_{xy}^s = \sum_{i=1}^N F_{xy}^i t_i \left[ \begin{aligned} & B_{x0}^i B_{y0}^i + \frac{1}{2} t_i \left[ B_{x0}^i (B_{y1}^i - (z_i - z_{y0}) B_{y2}^i) + B_{y0}^i (B_{x1}^i - (z_i - z_{y0}) B_{x2}^i) \right] + \\ & \frac{1}{3} t_i^2 (B_{x1}^i - (z_i - z_{x0}) B_{x2}^i) (B_{y1}^i - (z_i - z_{y0}) B_{y2}^i) - \\ & - \frac{1}{8} t_i^3 \left[ B_{x2}^i (B_{y1}^i - (z_0 - z_{y0}) B_{y2}^i) + B_{y2}^i (B_{x1}^i - (z_0 - z_{x0}) B_{x2}^i) \right] + \\ & \frac{1}{20} t_i^4 B_{x2}^i B_{y2}^i \end{aligned} \right] \quad (5-17)$$

The transverse shear stiffness of the section is then available as  $[F_{\alpha\beta}^s]^{-1}$ . Note that  $F_{12}^s$  will be nonzero if any layer is anisotropic or orthotropic in a local system (since then  $F_{xy}^s$  will be nonzero).

### 5.2.3.3 Reference Surface and Midsurface:

The geometry of the shell is defined by kinematic variables that exist at the nodes on the shell reference surface. The kinematics of the shell theory consist of measuring membrane strain on the reference surface and bending strain from the derivatives of the unit normal vector on the reference surface. The usual reference surface is the shell midsurface. However, many situations arise in which it is more convenient to define the reference surface as offset from the midsurface. In this case, it is assumed that the in-plane strain at any material point varies linearly across the section:

$$\varepsilon_{\alpha\beta} = \bar{\varepsilon}_{\alpha\beta} - (z - z_0) \bar{\kappa}_{\alpha\beta} \quad (5-3, \text{ repeated})$$

where  $\alpha$  and  $\beta$  represent the two orthogonal axes on the reference surface,  $\bar{\varepsilon}_{\alpha\beta}$  is the membrane strain of the reference surface,  $z_0$  is the distance to the reference surface measured from the midsurface, and  $\bar{\kappa}_{\alpha\beta}$  is the curvature of that surface. The positive values of  $z_0$  are given in the positive normal direction. When  $z_0 = t/2$ , the top surface of the shell is the reference surface, where  $t$  is the shell thickness. The bottom surface of the

shell becomes the reference surface when  $z_0 = -t/2$ . When  $z_0 = 0$ , the midsurface represents the reference surface.

If the response of the shell is linear elastic, any in-plane component of stress at a point through the shell section,  $\sigma_{\alpha\beta}$ , is given by

$$D_{\alpha\beta\gamma\delta} \left( \bar{\epsilon}_{\gamma\delta} - (z - z_0) \bar{\kappa}_{\gamma\delta} \right) \quad (\text{R.H.S. of 5-4, repeated})$$

where the plane stress elastic stiffness,  $D_{\alpha\beta\gamma\delta}$ , is defined from the elasticity and orientation of the material at the particular layer of the shell. As before, Greek subscripts take the range (1,2).

The section force and moment resultants per unit length can then be defined as

$$\begin{aligned} N_{\alpha\beta} &= \int_{-z_0-t/2}^{z_0+t/2} \sigma_{\alpha\beta} dz, \\ M_{\alpha\beta} &= \int_{-z_0-t/2}^{z_0+t/2} \sigma_{\alpha\beta} z dz, \end{aligned} \quad (5-18)$$

Integrating the above equations through the thickness leads to the resultant section stiffness,  $[A]$ :

$$\begin{Bmatrix} N_{\alpha\beta} \\ M_{\alpha\beta} \end{Bmatrix} = [A] \begin{Bmatrix} \bar{\epsilon}_{\alpha\beta} \\ \bar{\kappa}_{\alpha\beta} \end{Bmatrix} \quad (5-19)$$

#### 5.2.4 Boundary Conditions:

For comparison purpose, the boundary conditions imposed for the four hybrid cable stayed bridge models presented are the same as those of the reference bridges (Chapter 3, section 3.5). The pylons' bases are assumed to be fully fixed and the original soil conditions of the reference bridges are not included due to a lack of necessary data and to reduce further complications of the modeling process. The deck is simply supported at four planes, and each couple plane is symmetrical on the bridge system centre in the longitudinal direction (or traffic direction): (i) two planes at both deck ends, (ii) two planes at a specified distance from both deck ends; these distances are the same as those

of the reference bridge under consideration (as shown in Chapter 3). The additional elastic supports connect the deck and each of the two pylons at two planes passing through the centre of the two pylons. At each of the six planes of the deck boundary conditions, it is assumed that rigid diaphragms are present in the deck section, and hence all the relative vertical movement in the deck section is restricted. Figure 5-3 shows Model I dimensioned to match the Second Chang River Bridge with the application of the boundary conditions.

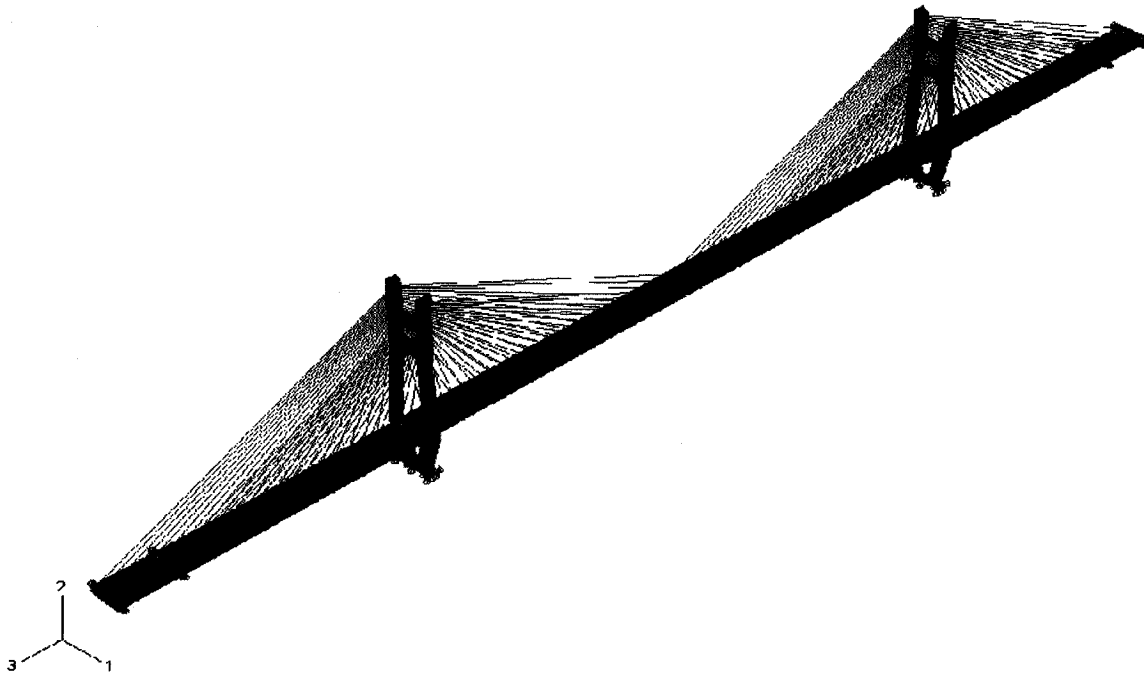


Figure 5-3 Model I-Second Chang River Bridge with boundary conditions.

### 5.3 Characteristics of the Non-linear Three-Dimensional Model:

In all the previous research related to finite element modeling of cable-stayed bridges, the deck is usually simulated using an equivalent grid involving beam elements with torsional degrees of freedom, the cables represented by simple truss elements and the pylons are simulated using simple beam-column elements. Although the general configurations of such models show a three dimensional shape, none of them can capture all the actual three dimensional deformations and stresses of the bridge components. Most

of those models are linear isotropic, and a few of them added geometric nonlinearity, assuming small ranges of deformations in comparison to the bridge dimension. Accordingly, the capability of such models are limited in simulating the general behavior of cable-bridge-systems and mostly need further refinement in order to understand the system parts behavior for further detailed design. For example, the cables actual static deformation, their vibration, and their interaction with the deck and pylons, and their behavior in three dimensions are not captured in any of those models. Figure (5-4) shows a traditional “three dimensional” finite element model and its representative results.

The three dimensional nonlinear-anisotropic finite element models involved in this research are more consistent with the actual material and geometrical properties and the complicated nature of the proposed hybrid cable-stayed bridge. In addition to the major purpose of capturing all the properties of the new bridge system, the “full-three-dimensional” character of the model can also be used, with the proper material properties, to improve the current simplified models and to get a more accurate understanding of the traditionally systems (steel, concrete, or composite; steel and concrete). It has been noted throughout this research that many properties, which have never been noted before or studied individually, are well covered in the present modeling technique, such as the cables’ actual deformation shape during deck vibration, the internal stress distribution and the distribution of the stress failure function (Tsai-Hill Failure Function) of the advanced composites in each component of the deck and the stress concentrations...etc. Figure 5.5 shows some details of the geometry of the presented 3D anisotropic non-linear finite element model and Figure 5.6 presents some the mode shapes in connection with the natural frequencies of the bridge.

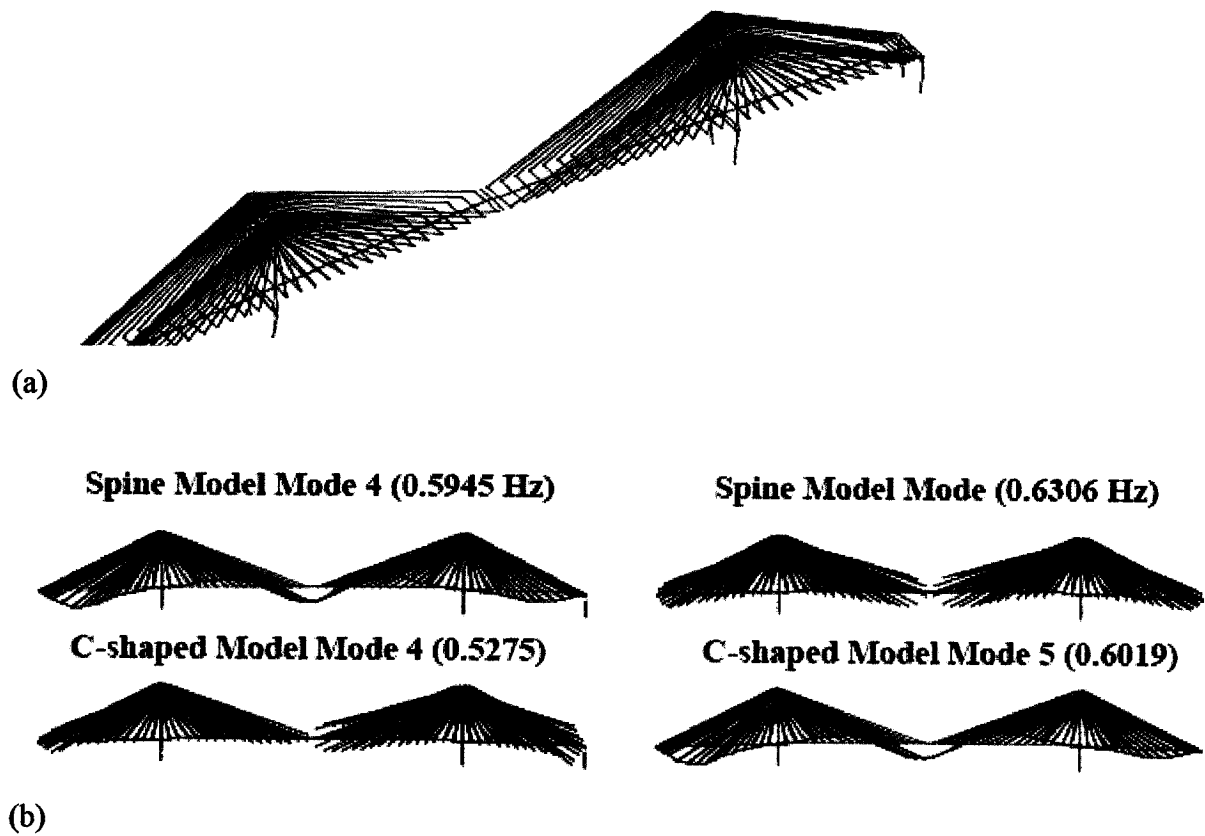


Figure 5.4 (a) Typical “Three dimensional” finite element model of long-span cable stayed bridge. (b) some of the mode shapes using these models.

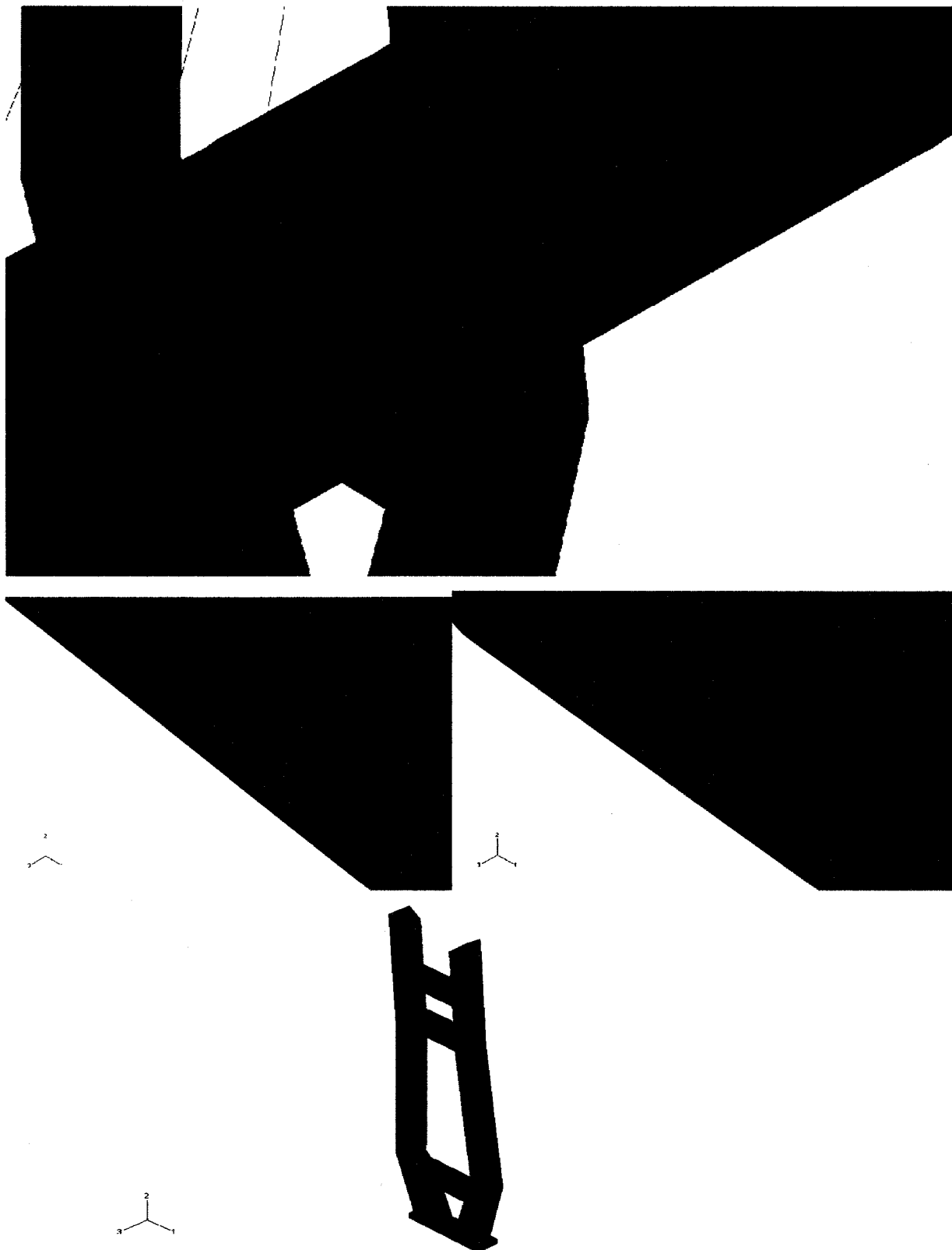


Figure 5.5 Geometrical details of the non-linear three dimensional- anisotropic finite element model: Second Change River Bridge, Model III-Stiff pylon.



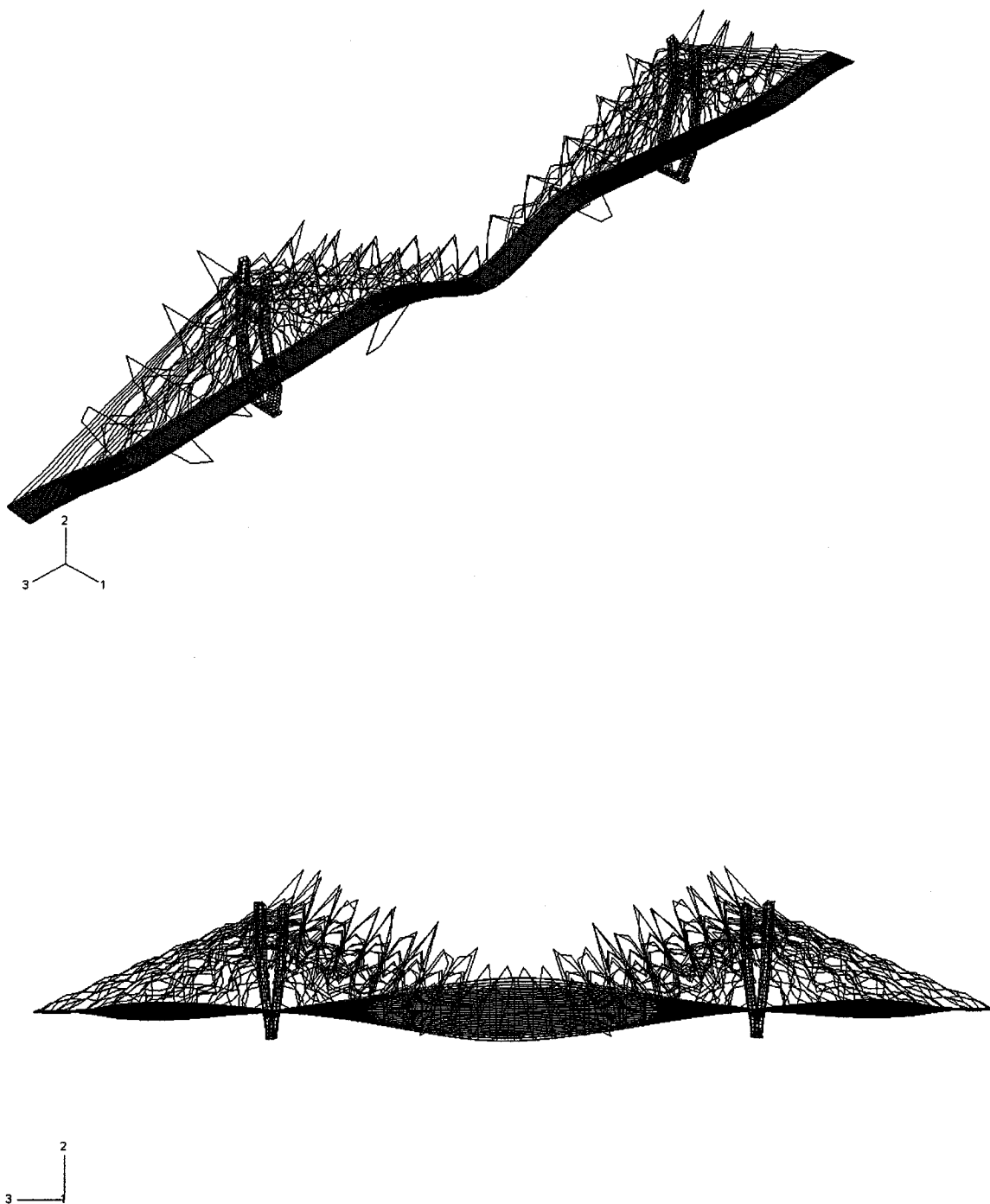


Figure 5.6 (a) Sample mode shapes showing the full-three dimensional deformation of the system for the third Vertical Bending Natural Frequency and side view of the first Torsional Natural Frequency; Second Change River Bridge, Model III.

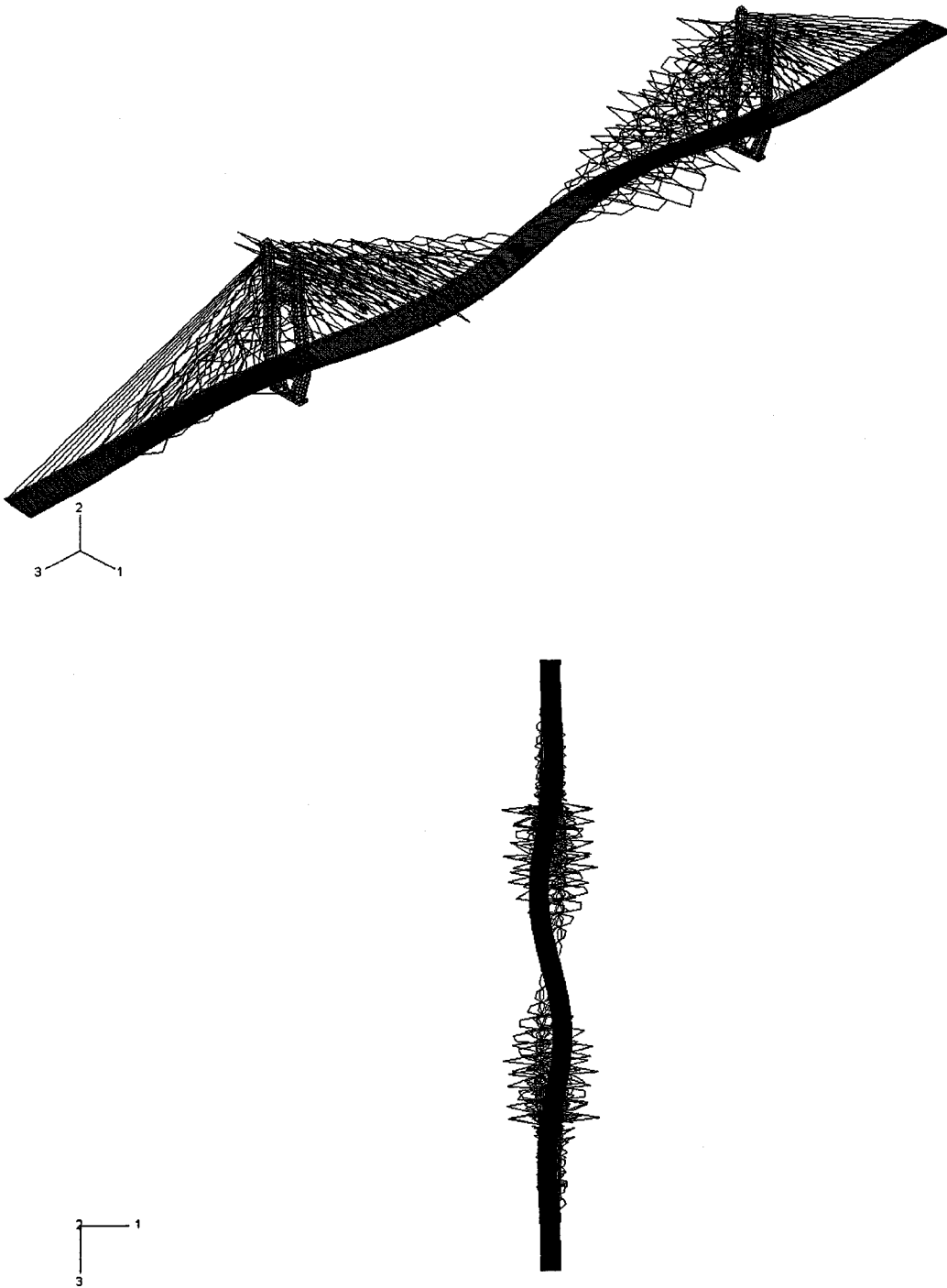


Figure 5.6 (b) Sample mode shapes showing the full-three dimensional deformation of the system for a second Horizontal (or Lateral) Bending Natural Frequency; Second Change River Bridge, Model III.

## 5.4 Convergence Study:

For the finite element model in the structure level, it is very difficult to optimize the meshes for the components of a very long span cable-stayed bridge of such hybrid nature. The convergence of the discretized model is a very complicated process because of the geometrical complication of such a huge three-dimensional system and also the nonlinearity of the system. Indeed, there is no specified optimized discretization approach for the proposed structure to follow, hence leading to the best approximation with the minimum number of elements for each component of the system. For example, if one has to solve a problem of a linear elastic simply supported rectangular plate under uniformly distributed static load, it is possible to test different mesh arrangements such as 2 X 2, 4 X 4, 8 X 6, 8 X 8 elements and so on. It is possible then to evaluate the convergence of such a simple finite element model through the major variables, the deformations of the plate, which can be examined on any line of symmetry of the plate. Once a situation is reached where the numerical change in values of these examined variables in any two successive mesh arrangements is negligible, and the distribution of these variables over the problem domain, or for simplicity the tested line of symmetry has not changed, then the penultimate mesh arrangement is an optimum for this model. For more complicated problems, this number, of course, is affected predominantly by the order of the element used; linear, parabolic or higher order, the boundary conditions, the geometry, the problem nature (linear or nonlinear), the problem time dependency etc. If there is a change in the geometry of the rectangular plate example, like having an elliptical hole, one needs then to refine the mesh around the hole ...etc.

In order to decide the optimum discretization of the proposed hybrid bridge system, it has been found that: (i) one of the static loading cases would be enough for deciding the optimum mesh of different components of the bridge system, (ii) choosing a specified mesh arrangement of any component of the bridge system, the deck, the cable stays and the pylon would effect the mesh refinement of any other component, (iii) the mesh size of each of the bridge components is dependent on the element type used for that component. Accordingly, the principle followed for approaching the optimum mesh for each of the system components, is to examine the distribution of the deformations over a critical plane in every component of the system and then to compare them with those resulting

from further consecutive mesh refinement for that component. Of course, the refinement of the mesh of each component affects the convergence of the discretization of the other components and the overall structure. It is also important to point out that the discretization optimization process is highly dependent on the element used in the finite element and definitely on the user experience. A convergence criterion has been chosen such that the difference in the maximum deformation of any of the bridge components should not be changed by more than 0.001% between the two final steps of the mesh refinement. Figure 5-7 shows the three dimensional deformed shapes of Model I dimensioned to match the Second Chang River bridge for different mesh arrangements of the deck element in the direction of the traffic. The test mesh sizes are 200 m, 100m, 50m, 25m and 15m in the bridge's deck longitudinal direction. This sample convergence study for the deck mesh in the longitudinal bridge direction longitudinal, shown here, followed three other mesh optimization steps: (i) optimizing the mesh of the pylons, (ii) optimizing the mesh of the eight cable-stay sets, and (iii) optimizing the mesh of the deck section. Figure 5-8 shows the deflection on the centerline of the top surface of the deck slab for the different five mesh arrangements. It is apparent that the 25 m mesh would be enough for the discussed case, since the 15 m mesh gives almost the same displacement field in the deck top surface.

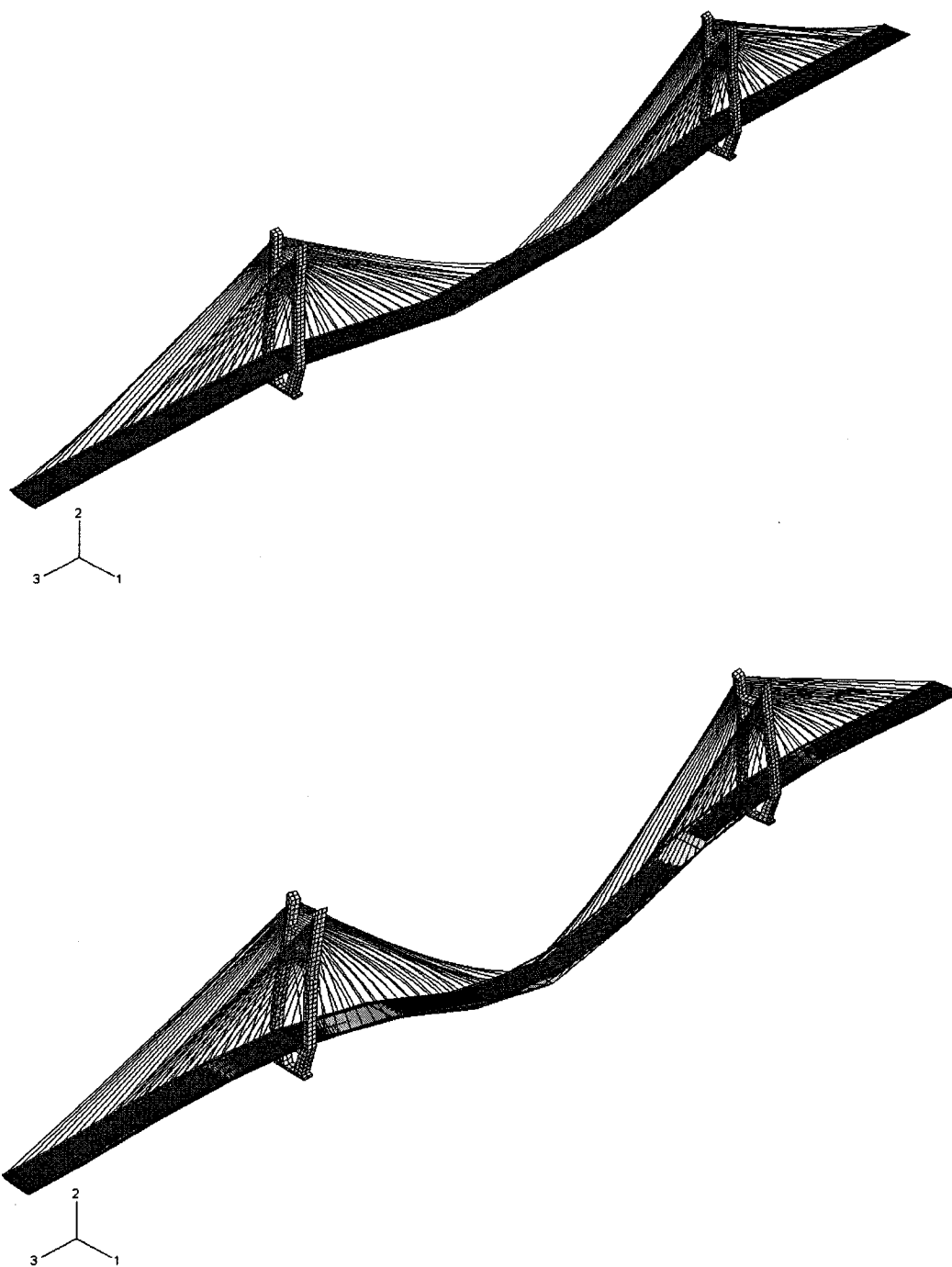


Figure 5-7 (a, b) Three dimensional deformation of Model I-Second Chang River Bridge, under the effect of self loads, dead loads and live load (on the mid span only or LLA) without the application of cable stays pretension, for mesh size of the deck element in the longitudinal direction of 200 and 100 m.

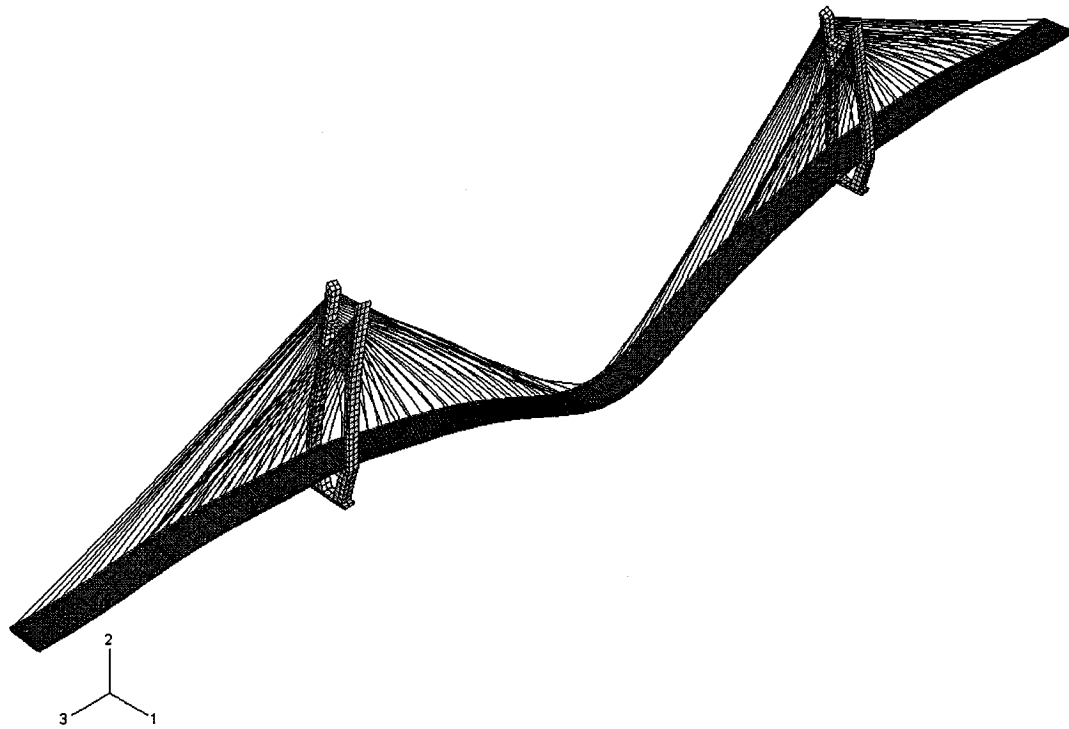
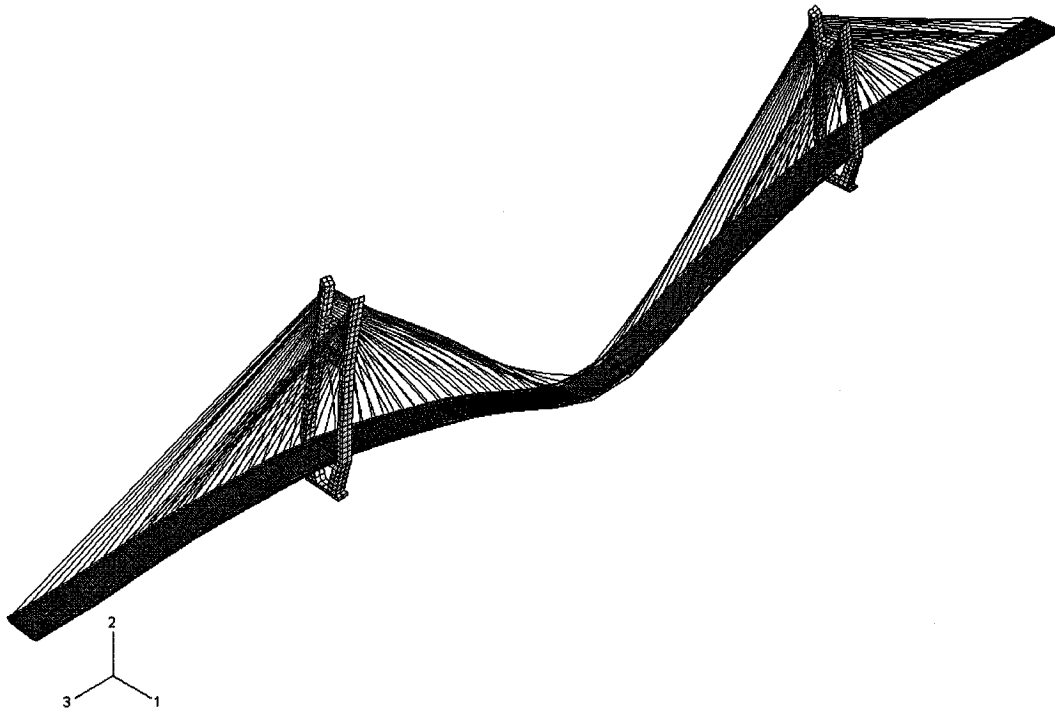


Figure 5-7 Continued... (c, d), for mesh size of the deck element in the longitudinal direction of 50 and 25 m.

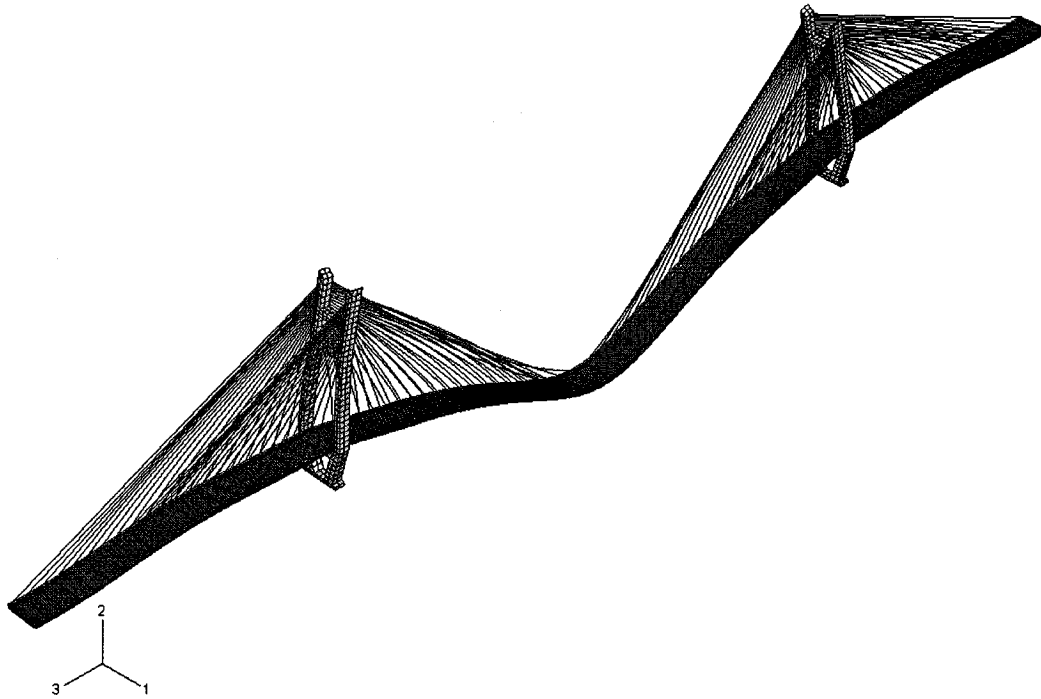


Figure 5-7 Continued... (e), for mesh size of the deck element in the longitudinal direction of 15 m.

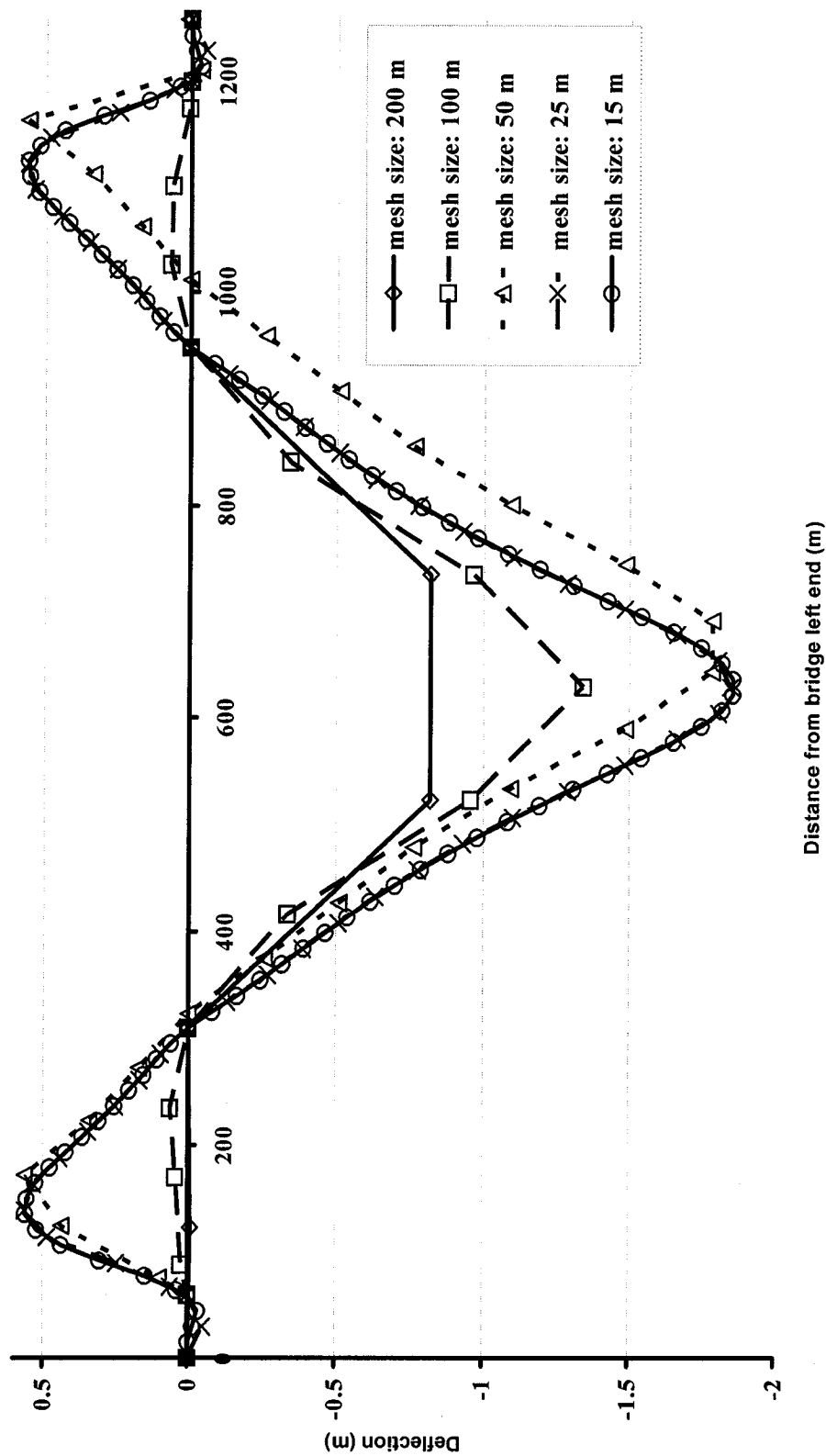


Figure 5-8 Deflection along top surface of deck-slab of Model I-Second Chang River Bridge, under the effect of self loads, dead loads and live load (on the mid span only or LLA) without the application of cable stays pretension, for different mesh size of the deck element in the longitudinal direction of 200, 100, 50, 25, 15 m.



## **5.5 Aerodynamic Stability:**

### **5.5.1 Introduction:**

The aerodynamic stability investigation in this research is primarily limited to the numerical modeling of aerodynamic forces to evaluate the flutter instability response of the proposed long span hybrid cable-stayed bridge. In order to involve the nonlinearities of the structural origins, a time domain scheme is adopted, and an investigation is done to evaluate the proposed bridge flutter velocity limits for different bridge design characteristics. The available data collected from wind-tunnel section tests of the reference bridges is used for this purpose.

The Model I deck cross section has the same boundary shape of the original aerodynamically smoothed sections of the Second Chang River and the Tatara reference bridges. Model II has variable depth, deep in the bridge centerline and shallow in both sides with sharp edges. The concrete alternative design of the Annacis Island bridge section has almost similar boundary shape of Model II. The Model III deck section has a similar boundary shape as Model I with rounded connections between the bottom face and the two inclined-bottom side faces. On the other hand, Model IV deck section has almost the same boundary size as the reference bridges deck sections, however, the lower edges of the tubular parts and the huge cut in the top mid part lead to divergence of this model from the reference bridges sections. Although, Model IV needs more refined wind tunnel tests to evaluate the flutter derivatives and to examine the vortex shedding characteristics ...etc, it is apparent from the dynamic structural analysis stage results that this model has extremely high Torsional/ Vertical natural frequencies (exceeding 10 in most of the cases) and the flutter stability is not a concern for this particular shape.

Vortex shedding analysis is considered to be out of the scope of this research. Buffeting analysis sometimes can be a major factor in the bridge design but in this research it is assumed also to be of minor influence on the aerodynamic stability of the proposed bridge, mainly because of the high stiffness to mass ratio. In addition, it could be further controlled by adding an artificial damping system.

### 5.5.2 Time Domain Flutter Analysis:

Principally, the analytical approach for flutter instability analysis has been carried out in the frequency domain [Davenport, 1962, and Scanlan, 1978]. The computational efficiency offered by the frequency domain is the main reason for adoption, predominantly when treating the unsteady aerodynamic forces that are functions of frequency. The flutter analysis is usually carried out by complex eigenvalue analysis. Recently, an efficient approach for coupled multi-mode flutter analysis has been proposed by introducing the unsteady self-excited aerodynamic forces in terms of rational function approximations [Matsumoto et al, 1994, Chen et al, 2000]. This has led to a convenient transformation of the equations into a frequency independent space-space format. A significant feature of this approach is that an iterative solution for determining flutter conditions is unnecessary because the equations of motion are independent of frequency. In this research, and in order to involve the nonlinearities of the structural origins, the time domain scheme proposed by Chen et al [2000] will be adopted and the following is a concise description of the procedure.

The governing equation of motion with respect to the static equilibrium position of a bridge excited by aerodynamic forces is given in matrix form as:

$$M\ddot{Z} + C\dot{Z} + KZ = F_{se} + F_b \quad (5-20)$$

Where

M : Mass matrix,

C : damping matrix,

K : stiffness matrix

Z: nodal displacement vector, and each dot denotes partial differentiation with respect to time t

F: the nodal force vector, and subscript (se) and (b) represent the self-excited and turbulence-induced buffeting force components, respectively.

According to Scanlan [1978], for harmonic motion, the self-excited forces per unit length (Figure 5-9), which are the lift [ $L_{se}(t)$ ], drag [ $D_{se}(t)$ ] and pitching moment [ $M_{se}(t)$ ], are commonly described utilizing flutter derivatives as follows:

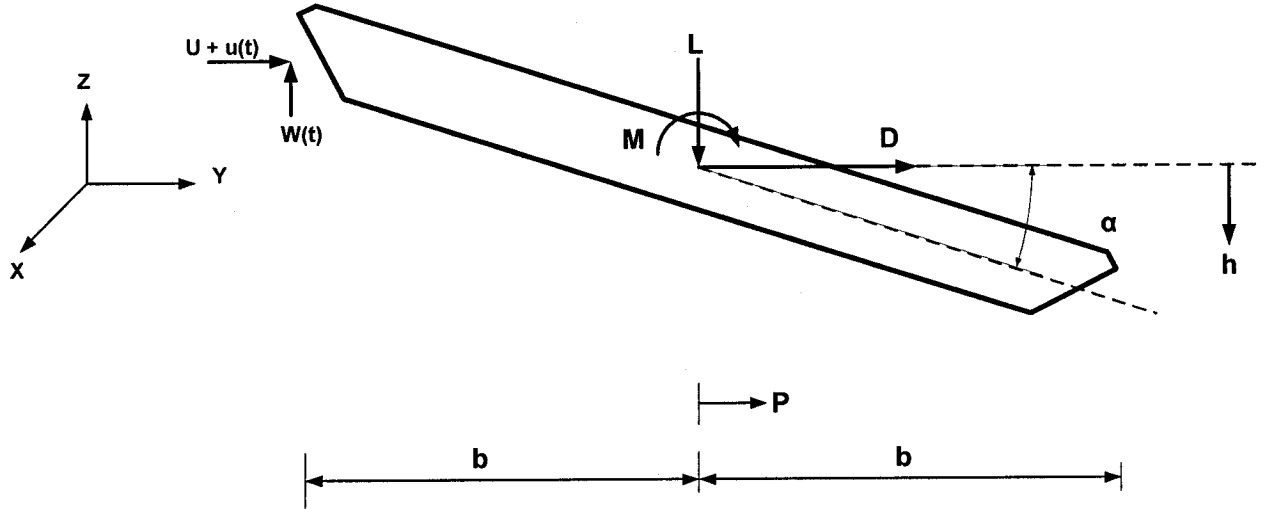


Figure 5-9 Aerodynamic forces on bridge section

$$L_{se}(t) = \frac{1}{2} \rho U^2 (2b) \left( kH_1^* \frac{\dot{h}}{U} + kH_2^* \frac{b\dot{\alpha}}{U} + k^2 H_3^* \alpha + k^2 H_4^* \frac{h}{b} + kH_5^* \frac{\dot{P}}{U} + k^2 H_6^* \frac{P}{b} \right) \quad (5-21-a)$$

$$D_{se}(t) = \frac{1}{2} \rho U^2 (2b) \left( kH_1^* \frac{\dot{h}}{U} + kH_2^* \frac{b\dot{\alpha}}{U} + k^2 H_3^* \alpha + k^2 H_4^* \frac{h}{b} + kH_5^* \frac{\dot{P}}{U} + k^2 H_6^* \frac{P}{b} \right) \quad (5-21-b)$$

$$M_{se}(t) = \frac{1}{2} \rho U^2 (2b^2) \left( kA_1^* \frac{\dot{h}}{U} + kA_2^* \frac{b\dot{\alpha}}{U} + k^2 A_3^* \alpha + k^2 A_4^* \frac{h}{b} + kA_5^* \frac{\dot{P}}{U} + k^2 A_6^* \frac{P}{b} \right) \quad (5-21-c)$$

where;

$\rho$  : the air density;

$U$ : mean wind velocity;

$k = \omega b / U$  is the reduced frequency with  $(\omega)$  being the circular frequency of vibration;

$H_i^*, P_i^*$  and  $A_i^* (i=1..6)$  are frequency dependent flutter derivatives,

$h, P, \alpha$  : are vertical, lateral and torsional displacements respectively.

The self-excited forces per unit span induced by arbitrary structural motion can be expressed in terms of convolution integrals as follows:

$$L_{se}(t) = \frac{1}{2} \rho U^2 \int_{-\infty}^t (I_{L_{seh}}(t-\tau)h(\tau) + I_{L_{sep}}(t-\tau)p(\tau) + I_{L_{se\alpha}}(t-\tau)\alpha(\tau))d\tau \quad (5-22-a)$$

$$D_{se}(t) = \frac{1}{2} \rho U^2 \int_{-\infty}^t (I_{D_{seh}}(t-\tau)h(\tau) + I_{D_{sep}}(t-\tau)p(\tau) + I_{D_{se\alpha}}(t-\tau)\alpha(\tau))d\tau \quad (5-22-b)$$

$$M_{se}(t) = \frac{1}{2} \rho U^2 \int_{-\infty}^t (I_{M_{seh}}(t-\tau)h(\tau) + I_{M_{sep}}(t-\tau)p(\tau) + I_{M_{se\alpha}}(t-\tau)\alpha(\tau))d\tau \quad (5-22-c)$$

where I represents the impulse function of the self-excited forces.

Taking Fourier transforms of (5-22) and comparing the corresponding terms with (5-21), a relation connecting the aerodynamic impulse functions and the flutter derivatives can be written as:

$$\begin{aligned} \bar{I}_{L_{seh}} &= 2k^2(H_4^* + iH_1^*); \\ \bar{I}_{L_{sep}} &= 2k^2(H_6^* + iH_5^*); \end{aligned} \quad (5-23-a)$$

$$\begin{aligned} \bar{I}_{L_{se\alpha}} &= 2k^2b(H_3^* + iH_2^*) \\ \bar{I}_{D_{seh}} &= 2k^2(P_6^* + iP_5^*); \\ \bar{I}_{D_{sep}} &= 2k^2(P_4^* + iP_1^*); \end{aligned} \quad (5-23-b)$$

$$\begin{aligned} \bar{I}_{D_{se\alpha}} &= 2k^2b(P_3^* + iP_2^*) \\ \bar{I}_{M_{seh}} &= 2k^2b(A_4^* + iA_1^*); \\ \bar{I}_{M_{sep}} &= 2k^2b(A_6^* + iA_5^*); \\ \bar{I}_{M_{se\alpha}} &= 2k^2b^2(A_3^* + iA_2^*) \end{aligned} \quad (5-23-c)$$

Referring to (5-22), each of the self-excited forces can be formulated in the frequency domain as a product of the bridge displacement and the corresponding transfer function, which has been expressed in terms of flutter derivatives. Since the flutter derivatives are normally known only at discrete values of the reduced frequency (k), approximate expressions are used to develop these as continuous function of the reduced frequency. The rational function approximation approach known as Roger's approximation can be employed for this intention. The aerodynamic transfer function in relation to the term consequent to the lift induced by the vertical motion  $L_{seh}(t)$  can be written as:

$$\bar{I}_{L_{seh}}(i\omega) = 2k^2(H_4^* + iH_1^*) = A_1 + A_2 \left( \frac{i\omega b}{U} \right) + A_3 \left( \frac{i\omega b}{U} \right)^2 + \sum_{l=1}^m \frac{A_{l+3} i\omega}{i\omega + \frac{d_l U}{b}} \quad (5-24)$$

were  $A_1, A_2, A_3, \dots, A_{l+3}$ , and  $d_l$  ( $d_l \geq 0$ ;  $l = 1$  to  $m$ ) are frequency independent coefficients; the first and second terms characterize non-circulatory and static-aerodynamics damping respectively; the third term denotes the additional aerodynamic mass that is normally negligible; and the rational terms represent the unsteady components that lag the velocity term and permit an approximation of the time delay through positive values of the parameter  $d_l$ . The quantity “ $m$ ” is related to the degree of accuracy of the estimate. The coefficient shown in (5-24) can be evaluated using linear and non linear least squares methods using the experimentally obtained flutter derivatives at different reduced frequencies.

The preceding rational function representation of the aerodynamic transfer function can be extended into the Laplace domain with  $s$  [where  $s = (-\xi + i)\omega$ , and  $\xi$  is the damping ratio of the motion] in (5-24) substituted for  $i\omega$ . The inverse Laplace transform yields the aerodynamic impulse function:

$$I_{L_{seh}}(t) = A_1\delta(t) + A_2\dot{\delta}(t) + A_3\frac{b}{U}\ddot{\delta}(t) + \sum_{l=1}^m \int_{-\infty}^t A_{l+3} \exp\left(-\frac{d_l U}{b}(t-\tau)\right) \dot{\delta}(\tau) d\tau \quad (5-25)$$

Where  $\delta(t)$  is Dirac delta function.

Then, the self-excited lift induced by arbitrary vertical motion can be expressed as:

$$L_{seh}(t) = \frac{1}{2} \rho U^2 \left( A_1 h(t) + A_2 \frac{b}{U} \dot{h}(t) + A_3 \frac{b^2}{U^2} \ddot{h}(t) + \sum_{l=1}^m \phi_l(t) \right) \quad (5-26)$$

Where  $\phi_l(t)$  ( $l=1\dots m$ ) are new variables that are introduced to express the aerodynamic phase lag and satisfy the following equations:

$$\dot{\phi}_l(t) = -\frac{d_l U}{b} \phi_l(t) + A_{l+3} \dot{h}(t), \quad (l=1\dots m) \quad (5-27)$$

Similar formulations for the other self excited force components can be derived with analogous definitions.

## **Chapter 6**

# **Performance Investigation**

# **Chapter 6**

## **Performance Investigation**

### **6.1 Introduction:**

The performance investigation of the proposed hybrid long span cable stayed bridge is shown in this chapter through a thorough verification of the analysis design procedure presented earlier. This includes the basic characteristic evaluation of the deformations, the failure function of the advanced laminated composite material of the deck, the principal stresses of the Chopped FRP parts of different models, tensile stresses of the stay cables and the basic dynamic parameters. The relation between deformations, failure function and natural frequencies with the laminated box girder micro/macro design is reported. In addition, the effect of CFRP cable Radius on the deformations of the deck, failure function and natural frequencies are shown. The effect of pylon stiffness on the static and dynamic behavior is shown in brief. The effect of major micro/macro and structural levels parameters on the critical flutter Speed is also investigated.

## **6.2 The Deck:**

### **6.2.1 Laminate Macro Design:**

Optimizing the laminate macro design, or deciding the optimum orientation and thickness of each lamina and the optimum number of laminas in the laminate, is a very complicated process and can only be performed through a trial and error procedure. The laminate design is a convoluted function of: (i) the deck section, i.e. the internal cells and the outer binding box dimensions and shapes, and hence the “laminate panels” dimensions, (ii) the mechanical properties of advanced composite materials to be used for the different laminas, (iii) the geometrical and structural characteristics of the bridge system, (iv) The applied stress fields (due to self, super imposed dead and traffic loads, wind loads...etc), the required dynamic characteristics of the system and the acceptable deformations and the vibration limits, and (v) The laminate apparent anisotropic behavior, which produces deformations in all directions due to any applied stress in any direction, has to be employed by choosing suitable laminate lay-up, in order to reduce the deformations and to optimize the bridge structural system performance.

The deck section boundary dimensions and shape are dependent on the bridge type, the traffic volume, the bridge central span length, the applied stress level, vibration and aerodynamic considerations. For a cable stayed bridge system, they are also dependent on the cable-stays arrangement (harp, semi-harp or fan), the cable material and radius, and the distances between cable-anchorage to the deck. Consequently, the required deck dimensions and stiffness are major factors of the laminate design. However, in the present research, the boundary shape and dimensions are approaching those of the reference bridges and defined by the proposed models (Model I, II, III and IV). Hence, the problem is cut down to only deciding the internal cell dimensions, or more simply the “laminate panel” dimensions, and the laminate macro-structure.

The mechanical properties of different laminas are dependent on their material components and micro design, which are evaluated in the Micro/Macro phase of the analysis and design procedure, and employed as input to the present step. As the proposed hybrid bridge systems have huge dimensions, it is necessary to simplify the filament winding manufacturing process and to reduce the expected cost by using the



same composite material for all laminas. This would also simplify the laminate design and reduce the number of variables involved, although using different composite materials of specific properties (including high strength, high stiffness, high ductility, high fatigue resistance ...etc) for different laminas would improve the laminate properties and hence the deck section characteristics. Indeed, using different materials for different laminas would highly complicate the laminate design step, hugely increases the number of laminate macro design trial cycles, and requires the use of higher order transverse shear flexible theory to accurately evaluate the high variation of the interlaminar shear [Reddy 2004] of such arrangements.

The cable-stayed system is transferring the static loads in the longitudinal direction of the bridge; hence the laminate macro structure could be used to employ the anisotropic nature of the advanced material in most effective manner related to the bridge structural behavior. Accordingly, the “Laminas Dominant Fiber Alignment”, LDFA, is introduced in this research, which represents the alignment of the majority of laminas in the laminates, for example  $\pm 10^\circ$  or simply  $10^\circ$ . It also referred to the laminate orientation with respect to the main bridge direction (the longitudinal direction of the bridge, or the traffic direction). The LDFA is investigated as a variable in the performance study to show its effect on the structural behavior of the bridge system.

Trying different lay-up arrangements, assuming a constant lamina thickness and approaching acceptable static and dynamic characteristics, starts the optimum laminate design. This step will end with (i) number of the laminas in the laminate, and (ii) the orientation of each lamina (the inclination of the uni-directional-fiber- lamina measured in degrees from a reference axis or direction, which is the longitudinal bridge axis or the traffic direction in this research). Finalizing the optimum laminate design would be through achieving an optimum performance of the bridge system by varying the thicknesses for different laminas, provided that the numbers of laminas and their orientations have been decided as a result of the first step.

The following is an illustration of the Laminate Macro Design process through an example of a simply supported bridge, which shows the effect of the laminate macro-design on different structural parameters of the advanced composite bridge. The laminate

arrangements that are investigated in the performance study of the Hybrid Long Span Cable Stayed Bridge System are also given.

#### **6.2.1.1 Optimizing the Laminate Macro Design:**

A simply supported advanced composite bridge (Figure 6.2.1.1) is given below to show the optimization process of the Laminate micro design. The bridge is designed according to CHBDC-01 code using CL-625-ONT truck and lane loading (Figure 6.2.1.2). The bridge has two lanes of 3.75 m each, two shoulders of 2.5m each (including the barrier), and the design traffic speed is 100 km/hr. The advanced composite bridge deck is composed from a laminated advanced composite box girder and a top surface chopped FRP plate or “deck slab”. The laminated box girder is formed, as shown in the figure, from “inner” boxes and “outer” binding box in the same way as explained earlier for the Models I, II and III in Chapter 3. The composite materials properties for the chopped FRP “deck slab” are the same as those used in the performance study (Table 4.2). For the lamina, again the composite material formed from E-Glass fiber and Vinylester Matrix (GFVM) is involved, with the same fiber fraction and properties as that used in the HLSCSB performance study (Table 4.4).

It has been found that the truck loading conditions, rather than the lane loading conditions are the critical design loading conditions. Consequently, two extreme truckload settings are considered: (i) when the center of the bridge in the longitudinal direction is at the mid-distance between the center of gravity of the truck and the closest axel, for both opposite traffic directions (Figure 6.2.1.3-a), and (ii) when the second axel of a truck just passes over one of the bridge support lines (axel 1 is out of the bridge, axels 2, 3, 4 and 5 are on the bridge) in the first traffic direction, while axel 4 of a second truck is just passing on the same bridge support line (axel 5 is out of the bridge, axels 1, 2, 3 and 4 are on the bridge) in the second traffic direction (Figure 6.2.1.3-b).

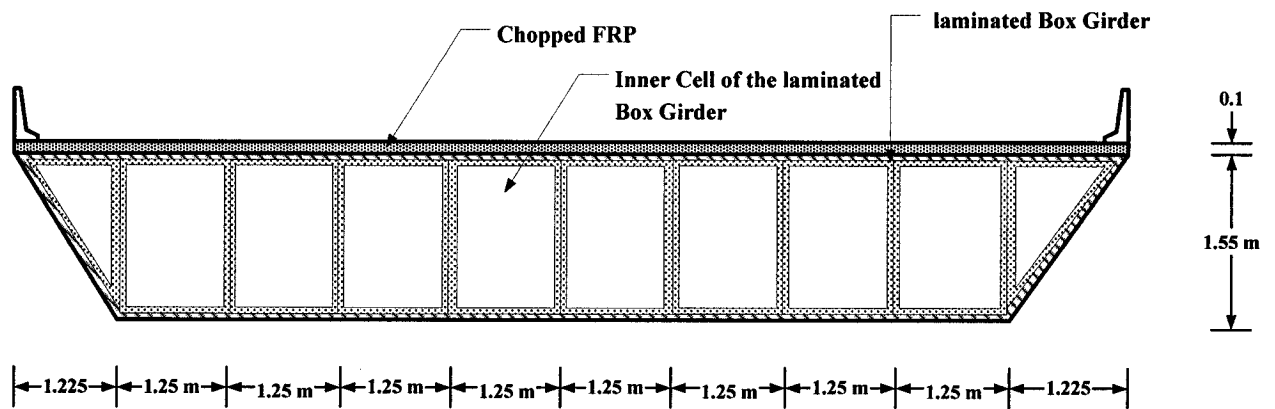


Figure 6.2.1.1 Deck Section of the simply supported advanced composite bridge superstructure sections.

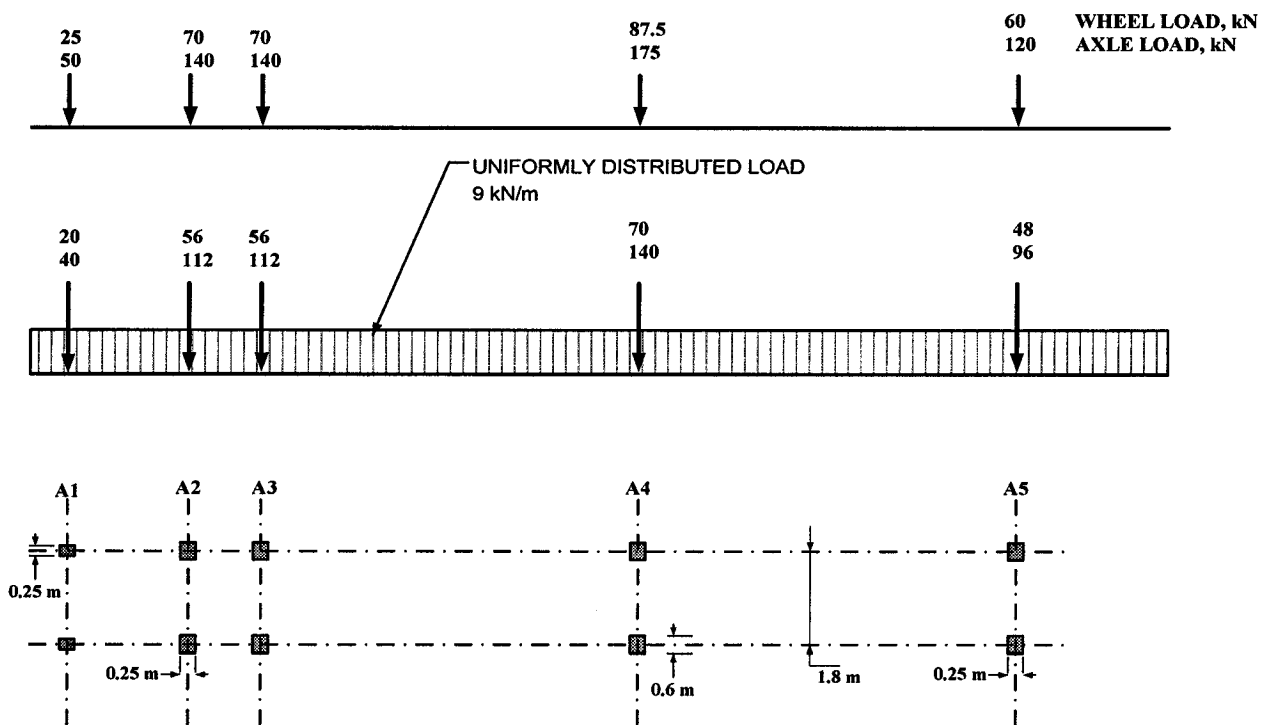


Figure 6.2.1.2 . CL-625-ONT Truck Loads, Lane loads and Design Truck Dimensions

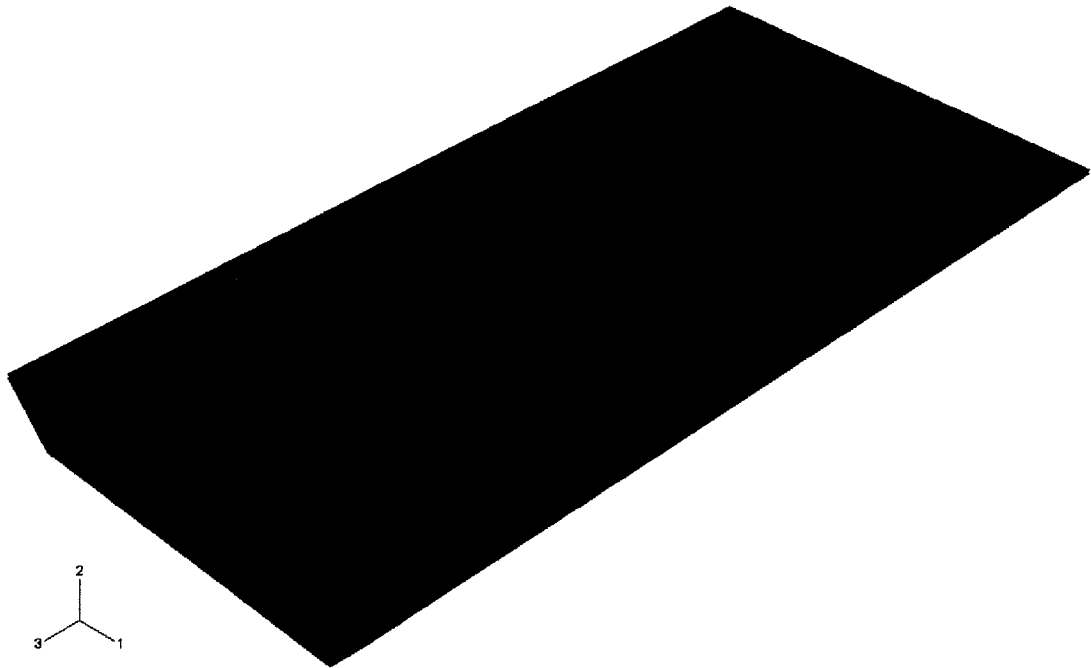


Figure 6.2.1.3-a The advanced composite bridge 3D shape with the first extreme CL-625-ONT truck loading (axes 1,2,3 are the global axes X,Y and Z). The traffic direction is in the  $\pm Z$ .

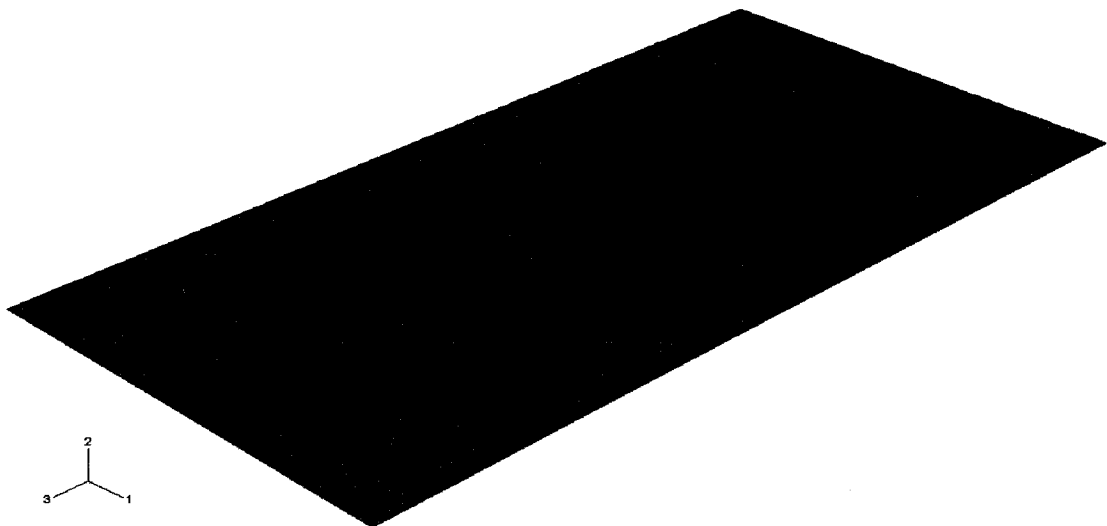


Figure 6.2.1.3-b The advanced composite bridge top surface with the second extreme CL-625-ONT truck loading.

The advanced composite bridge laminate macro design is investigated through seven different lay-up arrangements with 4, 8 and 12 laminas for either the inner box or the outer binding box laminates. Both laminate thicknesses are the same for all the studied arrangements. The lamina thickness is assumed to be constant for every case, and simply equal to the laminate thickness divided by number of laminas per laminate. Three different laminate thicknesses of 0.018 m, 0.027 m and 0.036 m are studied in this example. All the studied laminate arrangements are shown in Table 6.2.1. For all “Laminate Alignment” (LA) designs, except “Laminate Alignment design a” (or LAa in Table 6.2.1), the orientation has been arranged to be symmetrical on the laminate mid-surface, and every orientation has its skew-symmetric couple (e.g.  $+10^\circ$  and  $-10^\circ$  form an orientation couple). In laminate designs LAb, LAc and LAd, a single lamina-orientation of  $\pm 10^\circ$  or  $\pm 30^\circ$  or  $\pm 45^\circ$ , respectively, is used. In laminate designs LAe and LAf, two lamina-orientations of  $\pm 10^\circ$  and  $\pm 30^\circ$  or  $\pm 10^\circ$  and  $\pm 45^\circ$ , respectively, are used. In Laminate design LAg, three lamina-orientations of  $\pm 10^\circ$  and  $\pm 30^\circ$  and  $\pm 45^\circ$  are used. On the other hand, the Laminate Alignment “a”(or LAa), which is named LAa to be consistent with the laminate macro design of the hybrid long span cable stayed bridge (HLSCSB) shown in next section, results from modifying arrangement LAg, by trying many different plans, as will explained later.

The advanced composite bridge is modeled using ABAQUS Finite Element Program with laminated shell element S4R (Figure 6.2.1.4). The box girder deck geometry is exactly simulated with all its details, and the finite element mesh is refined in the regions where the applied wheel loads are high.

The first flexural natural frequency of the Advanced Composite Bridge (ACB) is ranged between 1.45 Hz to 1.7 Hz. This leads to an acceptable static deflection ranges between 0.08m to 0.11m [CHBDC-01, Figure 3.4.4]. The acceptable deflection range is higher than the maximum vertical displacements of all the examined cases and for all the laminate arrangements. The distribution of the deformations over the ACB is unsymmetrical; however, it is observed that the divergence from a symmetrical shape is minimized with LAg and LAa laminate arrangements. Both laminates have the highest number of laminas and higher number of orientations.

| Lamina                                                     | Lamina Alignment ( a )<br>or LAa (Degree) |                                     | LAb<br>(Degree) | LAc<br>(Degree) | LAd<br>(Degree) | LAe<br>(Degree) | LAf<br>(Degree) | LAg<br>(Degree) |
|------------------------------------------------------------|-------------------------------------------|-------------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
|                                                            | Inner Box<br>Laminate                     | Outer<br>Binding<br>Box<br>Laminate |                 |                 |                 |                 |                 |                 |
| 1                                                          | -10                                       | -10                                 | 10              | 30              | 45              | 10              | 10              | 10              |
| 2                                                          | 30                                        | 45                                  | -10             | -30             | -45             | 30              | 45              | 30              |
| 3                                                          | 10                                        | 10                                  | -10             | -30             | -45             | -10             | -10             | 45              |
| 4                                                          | -30                                       | -45                                 | 10              | 30              | 45              | -30             | -45             | -10             |
| 5                                                          | 10                                        | 10                                  | -               | -               | -               | -30             | -45             | -30             |
| 6                                                          | 45                                        | 90                                  | -               | -               | -               | -10             | -10             | -45             |
| 7                                                          | 45                                        | 90                                  | -               | -               | -               | 30              | 45              | -45             |
| 8                                                          | 10                                        | 10                                  | -               | -               | -               | 10              | 10              | -30             |
| 9                                                          | -30                                       | -45                                 | -               | -               | -               | -               | -               | -10             |
| 10                                                         | 10                                        | 10                                  | -               | -               | -               | -               | -               | 45              |
| 11                                                         | 30                                        | 45                                  | -               | -               | -               | -               | -               | 30              |
| 12                                                         | -10                                       | -10                                 | -               | -               | -               | -               | -               | 10              |
| Lamina Thickness in<br>Int. or Ext. Laminate<br>(LTh-a), m | 0.0015                                    |                                     | 0.0045          | 0.0045          | 0.0045          | 0.0025          | 0.0025          | 0.0015          |
| Int. or Ext. Laminate<br>Thickness (LTh-b), m              | 0.00225                                   |                                     | 0.00675         | 0.00675         | 0.00675         | 0.003375        | 0.003375        | 0.00225         |
| Int. or Ext. Laminate<br>Thickness (LTh-c), m              | 0.003                                     |                                     | 0.009           | 0.009           | 0.009           | 0.0045          | 0.0045          | 0.003           |

Table 6.2.1 Different designs of the internal and external laminates of the advanced composite bridge.

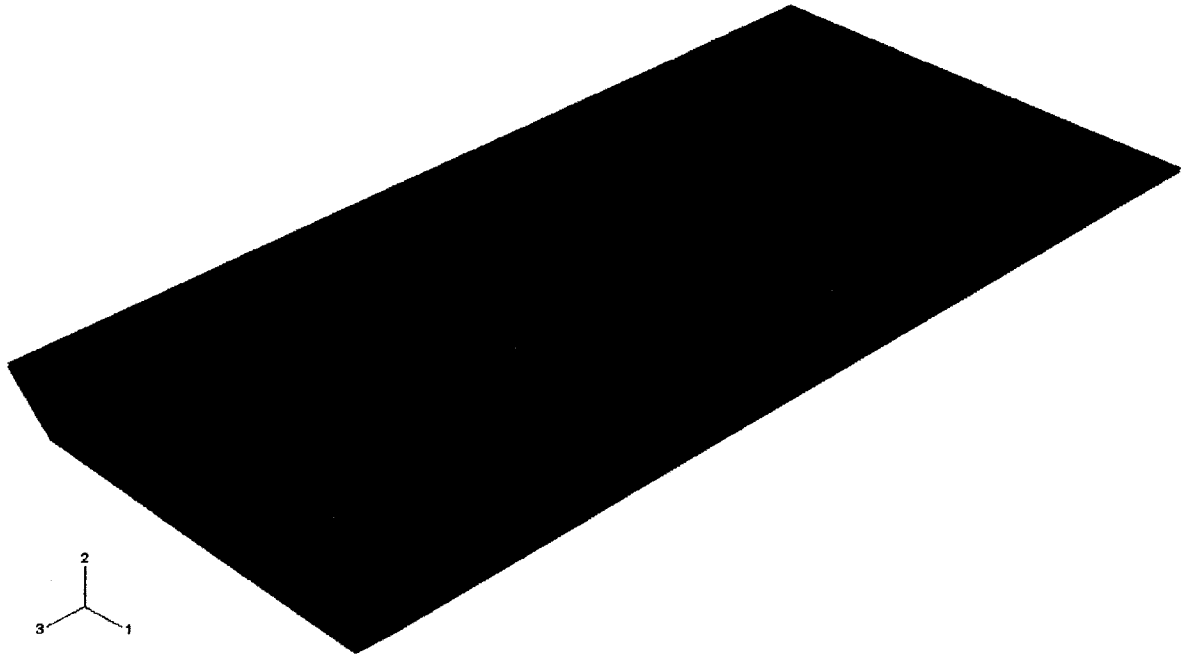


Figure 6.2.1.4. The finite element mesh for the advanced composite bridge.

Figures 6.2.1.5.a, b, c show the maximum displacements,  $U$  parallel to the  $X$  (or 1) coordinate,  $V$  parallel to the  $Y$  (or 2) coordinate, and  $W$  parallel to the  $Z$  (or 3) coordinate (which represents the traffic direction), for the studied laminate arrangements. Figure 6.2.1.6 shows the first vertical natural frequency and Figure 6.2.1.7 shows the maximum Tsai-Hill Failure Function (THFF) of the laminated box girder.

Comparing the results of single-orientation-angle laminate arrangements (i.e.  $LAb$ ,  $LAc$ , and  $LAd$ ), it is observed that  $LAb$  (or  $\pm 10^\circ$  arrangement) gives minimum results for the lateral displacement  $U$  and a reasonably low deflection  $V$  (except with very low laminate thickness),  $LAc$  (or  $\pm 30^\circ$ ) gives minimum results for the longitudinal displacement  $W$ , and  $LAd$  (or  $\pm 45^\circ$ ) gives minimum results for the deflection  $V$ , minimum Tsai Hill Failure Function and the highest flexural natural frequency.

For the double-angle-orientation laminate arrangements resulting from combining  $\pm 10^\circ$  with either  $\pm 30^\circ$  or  $\pm 45^\circ$  (i.e.  $LAe$ , and  $L Af$  respectively), it is found that  $LAe$  gives lower lateral displacement  $U$  for all laminate thicknesses, and lower longitudinal displacement  $W$  only with medium laminate thickness.  $L Af$  gives lower deflection  $V$ ,

lower Tsai-Hill Failure Function, lower longitudinal displacement  $W$  for low or high laminate thickness, and higher first vertical natural frequencies (1stVNF). On the other hand, the triple-angle-orientation laminate arrangement LAg, gives medium deformations  $U$ ,  $V$ , and  $W$ , medium flexural natural frequency and high Tsai-Hill Failure Function.

After testing many modifications to the LAg arrangement, the trials have been concluded with the arrangement LAa, which gives the second lowest lateral displacement  $U$ , the lowest deflection  $V$ , the lowest longitudinal displacement  $W$ , the highest vertical natural frequencies and the second lowest Tsai-Hill Failure Function. Hence this arrangement is the best among the others from the structural performance point of view. The modifications to the LAg arrangement that result in LAa are: (i) giving a priority to one orientation, which has the lowest inclination with respect to the bridge direction (i.e.  $\pm 10^\circ$  in the present case), for both inner and outer laminates. Indeed, 50% of the laminate thickness is oriented in this direction, two thirds of it (33.3% of the total laminate thickness) is aligned in  $+10^\circ$  and the rest (or 16.7%) is aligned in  $-10^\circ$ . As mentioned earlier, this “major orientation” is introduced in this research as the “Laminate Dominant Fiber Alignment” LDFA, (ii) separating the laminas orientations of the inner from the outer laminate by using low to medium orientation angles for the inner laminate ( $30^\circ$ , and  $45^\circ$ ), and medium to high orientation angles for the outer laminate ( $45^\circ$ ,  $90^\circ$ ), in addition to the LDFA ( $10^\circ$ ) for each laminate, of course, (iii) breaking the skew-symmetric rule followed in the other six arrangements but keeping the symmetry of the laminas on the mid-surface of the inner or outer laminates.

The number of laminas of the inner or the outer laminates for the laminate macro-design arrangements (LAb through LAg) in the preceding investigation is simply the number of angles of orientations of the laminate, arranged according to the required symmetry with respect to the laminate mid-surface. For example, if the number of orientation angles is two, e.g.  $\pm 10^\circ$  and  $\pm 45^\circ$ , it simply requires eight laminas to end with a symmetrical laminas lay-up in a laminate. In the LAa arrangement, some orientations are switched with each other, keeping the original number of orientations as for LAg, which is  $12 = 2$  (for “+” and “-”)  $\times 3$  (number of orientations, e.g.  $10^\circ$ ,  $30^\circ$  and  $45^\circ$ )  $\times 2$  (for symmetry on the laminate mid-surface).

On the other hand, the increase of the laminate thickness, keeping the lamina to the laminate thickness ratio as a constant, results in decreasing the resultant displacement



field, increasing the natural frequencies and decreasing the Tsai-Hill Failure Function. The deformations of the deck, specifically the displacements, are in three dimensions. However, the vertical displacement component  $V$  is the leading displacement and it has values far larger than the other two components, lateral displacement  $U$  and the longitudinal displacement  $W$ . The longitudinal displacement  $W$  shows an increase then decrease with “LAb” laminate design and a similar behavior with less variation for “LAF” is observed. Nevertheless, the resultant displacement field over the bridge is affected very slightly and it continues to have the same decrease pattern as  $LTh$  increases. For LAb the only fiber alignment is  $\pm 10^\circ$  and for LAF the fiber alignments of  $\pm 10^\circ$  and  $\pm 45^\circ$  are used. The use of higher alignment angles in the laminate design reduces the variation of the  $W$  with respect to the laminate thick. The rate of decrease of the deflection ( $V$ ) and Tsai-Hill Failure Function, and the rate of increase of the natural frequencies decay with further increase of the laminate thickness. Hence increasing the laminate thickness more than certain level will not economically improve the bridge structural characteristics. Furthermore, different lamina thicknesses can also be adjusted to give better structural performance of the bridge.

A simplified rule to obtain the optimum laminate macro design for the proposed bridge models can be summarized as follows:

- i) As an elementary step, use at least three different lamina orientations (with skew symmetric arrangement, i.e.  $\pm$ ).
- ii) Always keep the symmetry of lamina-orientations on the mid-surface of the laminate, for both the inner and the outer laminates. This would allow an accurate simulation of the interlaminar shear using the first-order transverse shear flexible theory.
- iii) In more advanced step, switch the lamina orientations and/or the sign, in a trial and error process which ends with an optimum orientation arrangement, breaking the skew-symmetry of the “orientation-couples” but keeping the overall lamina orientation symmetry on the mid surface of the laminate.
- iv) Separate the lamina orientations of the inner from the outer laminate by using low to medium orientation angles for the inner laminate, and medium to high orientation angles for the outer laminate.

- v) Give a priority to one orientation, which has the lowest inclination with respect to the bridge direction (i.e. the basic  $\pm 10^\circ$  in this research), for both the inner and the outer laminates. This orientation is defined as the “Laminate Dominant Fiber Alignment” LDFA.
- vi) Try different LDFAs to find the one that gives the best structural performance.
- vii) Optimize different laminas thicknesses in a trial-and-error process according to the required static and dynamic characteristics of the bridge.

The simply supported advanced composite bridge deck boundary dimensions and deflection are comparable with the boundary dimensions and long term deflection of the typical concrete bridges used for short spans. For example, the slab on prestressed girder bridge shown in Figure 6.2.8, which is designed for the same loading and according to the same code. Its slab thickness is 0.25 m and its concrete compression strength is ( $f'_c = 30$  MPa). The five prestressed concrete girders of this bridge are standard CPCI-1400 Girders with concrete compression strength of ( $f'_c = 40$  MPa). Each girder has a prestressing reinforcement of twenty-eight strands (low relaxation, size designation 13, grade 1860 and diameter 12.7 mm). Eighteen of them (nine on each side of the girder cross section, symmetrically on the vertical center line) are straight and lie at the bottom of the girder. The remaining ten strands (five on each side of the girder cross section, symmetrically on the vertical center line) are linearly varied from 0.85 m (average height from the girder bottom surface) at both girder ends to 0.15 m over one third of the girder length from each side; while on the middle third, the strands are straight. The maximum static deflection is calculated to be 0.0154 m and the first flexural natural frequency is 3.81 Hz. The maximum deflection limit for this highway bridge superstructure vibration is 20 mm, [CHBDC-01, Figure 3.4.4]. The final deflection (including creep, shrinkage...etc) is calculated to be 0.072 m.

The results of the Advanced Composite Bridge (ACB) show higher deflection than the short term deflection but close to the long term deflection of the slab on prestressed girder bridge. On the other hand, the ACB gives lower flexural natural frequencies than those of the Slab on Prestress Concrete Girders Bridge (SPCGB), however, the advanced composite materials have much higher fatigue resistance than the reinforced concrete.

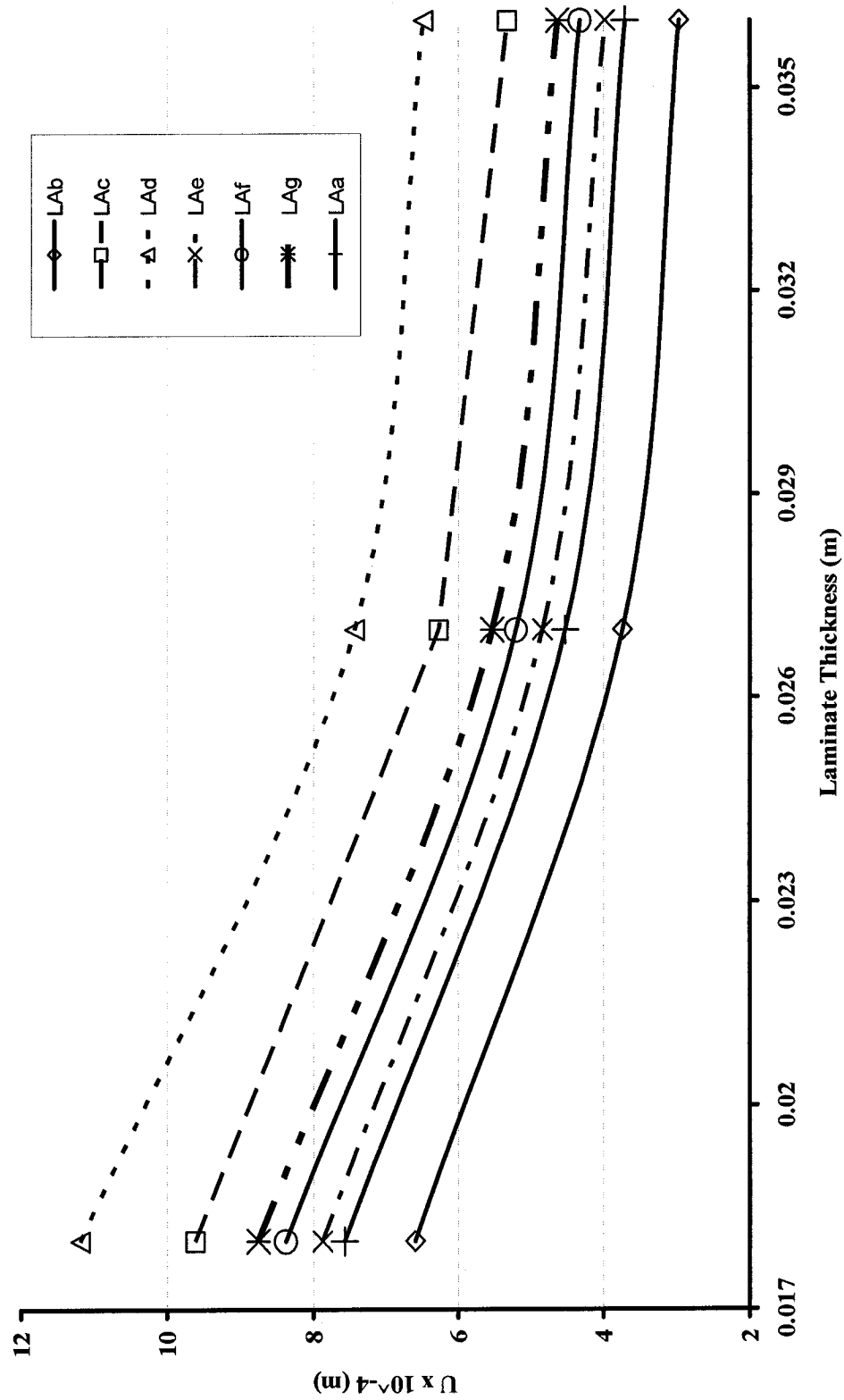


Figure 6.2.1.5-a Absolute maximum lateral displacement ( U ,in X or 1 Coordinate direction) of the deck top surface of the advanced composite box girder simply supported bridge, under dead and traffic loads.

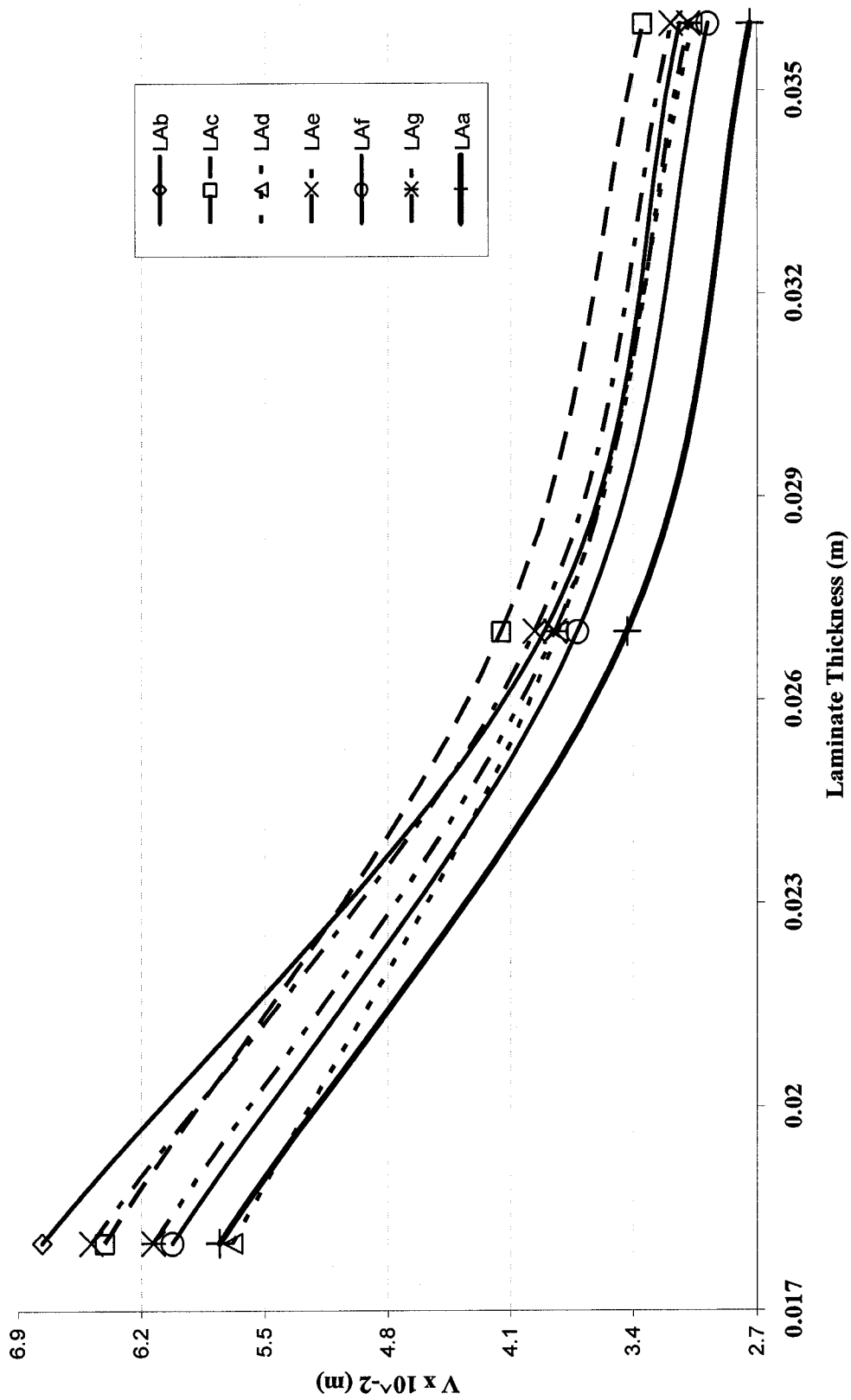


Figure 6.2.1.5-b Absolute maximum deflection, or vertical displacement ( $V$ ), in  $Y$  or  $Z$  coordinate direction) of the deck top surface of the advanced composite box girder simply supported bridge versus the laminate thickness ( $LTh$ ), under dead and traffic loads.

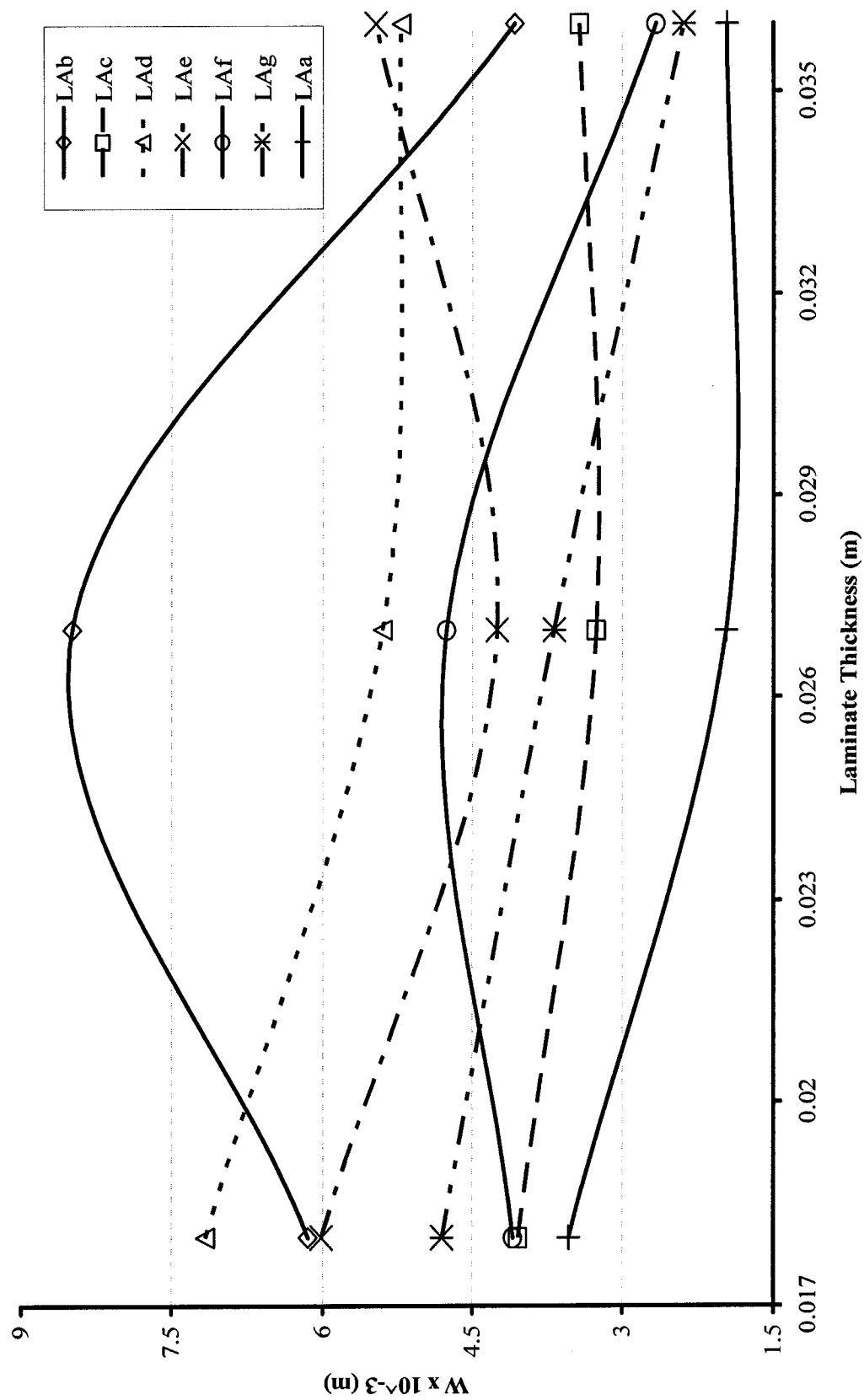


Figure 6.2.1.5-c Absolute maximum longitudinal displacement ( $W$ , in Z or 3 coordinate direction) of the deck top surface of the advanced composite box girder simply supported bridge versus the laminate thickness (LTh), under dead and traffic loads.

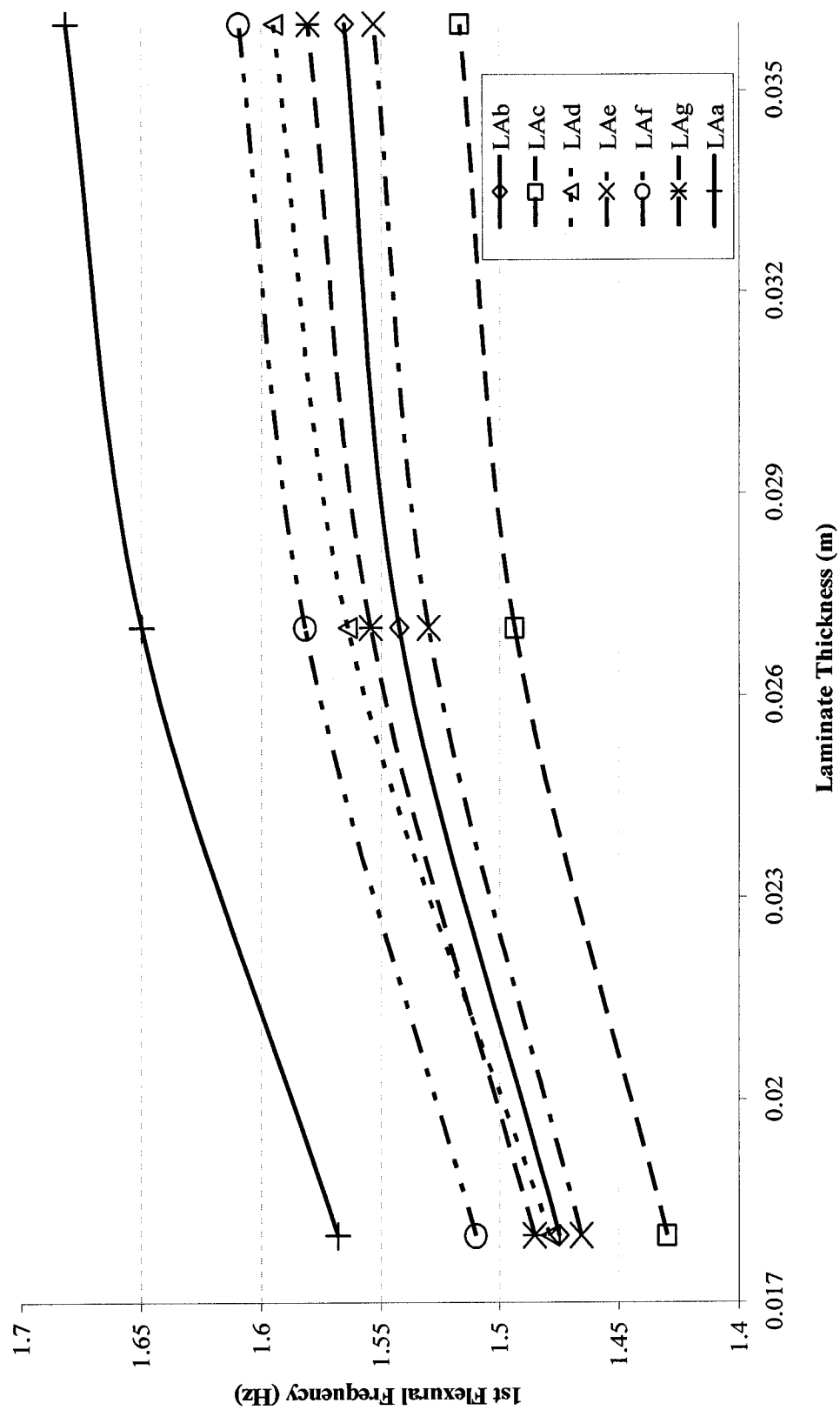


Figure 6.2.1.6 Relation between First Vertical Flexural Natural Frequencies (1stVNF) and the laminate thickness (LTh) of the advanced composite box girder simply supported bridge, for different laminate macro design arrangements.

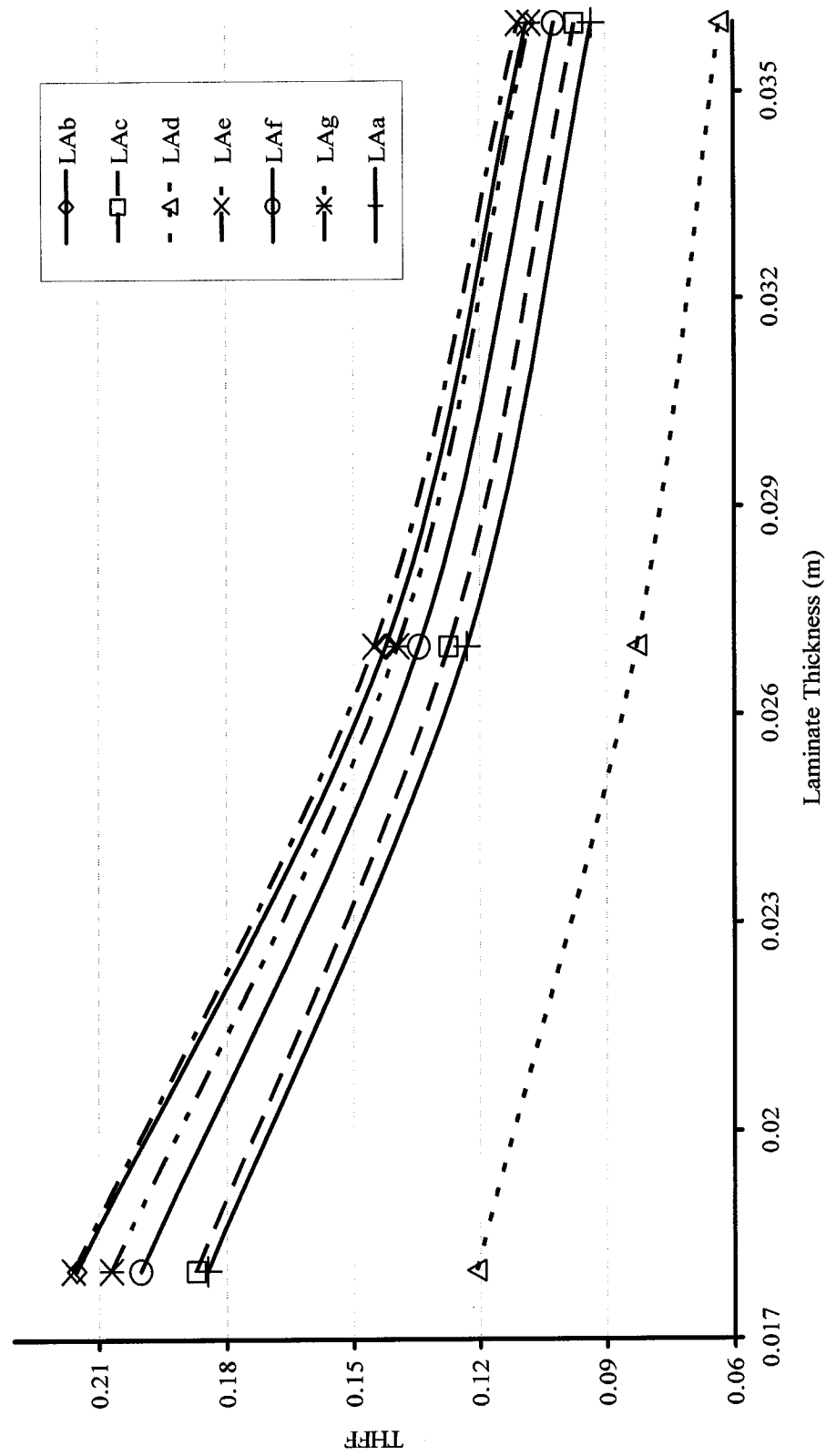


Figure 6.2.1.7 Relation between Tsai-Hill Failure Function and the laminate thickness (LTh) of the advanced composite box girder simply supported bridge, for different laminate macro design arrangements

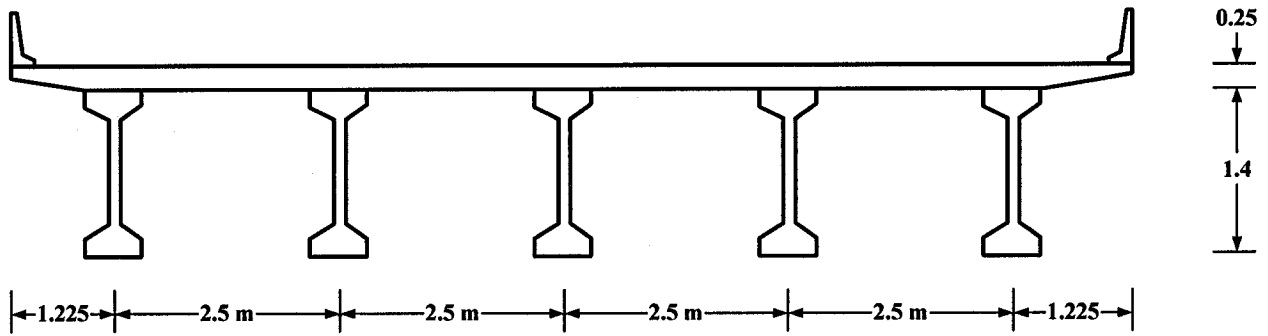


Figure 6.2.8 Typical 25 m span- slab on prestressed girder bridge.

### 6.2.1.2 Laminate Macro Design for HLSCSB:

As shown earlier, the composite material formed from uni-directional E-Glass Fiber and Vinylester Matrix (GFVM), which is the most economical choice for the lamina, is to be used for all the laminas of different orientations, for both the inner and the outer laminates, and for all the four models of each of the three hybrid bridges corresponding to the three reference bridges.

The optimized Lamina Alignment angles arrangement-a, (LAa), shown in the previous section, will be investigated throughout the performance study in this chapter as one of three lamina orientation designs. The other two LA designs, LAb and LAc, are derived from LAa by increasing the Laminate Dominant Fiber Alignment (LDFA) from  $10^\circ$  to  $20^\circ$  and  $30^\circ$  respectively. All the other laminas' orientations in the laminate are increased similar to LDFA. The lamina orientation is measured with respect to the longitudinal axis of the bridge (or the traffic direction) and takes an angle between  $-90^\circ$  to  $+90^\circ$ . If the orientation  $\alpha$  exceeds  $90^\circ$ , the orientation reference will be switched to  $\alpha - 180^\circ$ , which basically represents the same orientation. In the performance study, the Laminas Dominant Fiber Alignments (LDFA) will be used instead of the Laminas Alignment angles arrangement (LA), since it represents the only variable in the LA arrangement. Figure 6.2.1.8 shows an example of a lamina orientation of  $110^\circ$  that can also correspond to  $-70^\circ$ . The figure also shows the LDFA for the three investigated laminate alignment arrangements LAa, LAb, and LAc together with their reference to the traffic direction. Those alignment arrangements will be referred in the performance study as LDFA-a, LDFA-b and LDFA-c respectively.



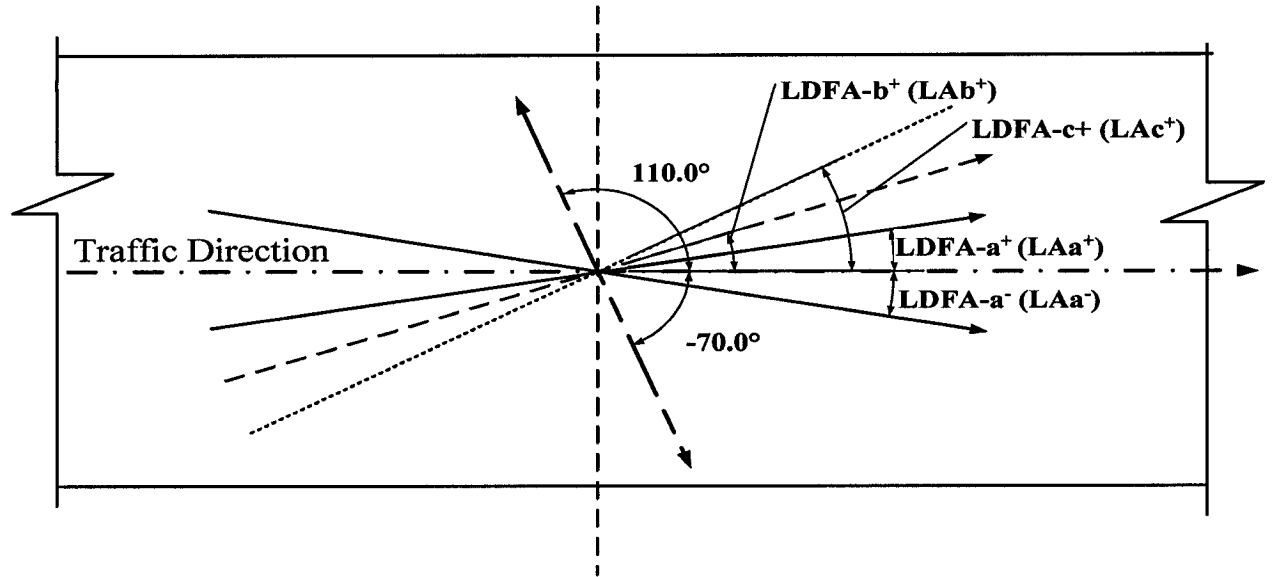


Figure 6.2.1.8 Laminar Dominant Fiber Alignment for different Laminate Alignments.

The laminate thickness  $LTh$  is also investigated as a variable in the performance study of the hybrid long span bridges. A set of three  $LTh$  values for each bridge, corresponding to the Anacis Island Bridge and the Second Chang River reference bridges, and a second set of another three  $LTh$  values for the bridge corresponding to the Tatara Bridge are studied. On the other hand, the thickness of every lamina is different from that for other laminas in the laminate, and has been optimized by a trial and error process, which results in an optimum bridge system performance. However, the ratio between each lamina thickness to the thickness of the laminate is a constant. Hence the thickness of lamina  $i$  is simply equal to the laminate thickness times the lamina  $i$  thickness ratio (constant), or;

$$l_{th(i)} = r_i * LTh$$

Where,  $l_{th(i)}$  : thickness of the lamina (i),  $r_i$  : ratio of the lamina thickness to the laminate thickness, which is independent of the laminate thickness,  $LTh$  : laminate thickness.

Table 6.2.2 shows the three Laminas Alignment angle designs and the three thickness designs for the HLSCS bridges corresponding to Annacis Island and the Second Chang River bridges. And Table 6.2.3 shows the three Laminas Alignment angle designs and the three thickness designs for the HLSCS bridge, corresponding to the Tatara Bridge.

| Lamina No | Laminate Thickness (m) |         |         | Laminas Alignment angle (with respect to the Bridge longitudinal direction)<br>(Degree) |                |                |                |                |                |
|-----------|------------------------|---------|---------|-----------------------------------------------------------------------------------------|----------------|----------------|----------------|----------------|----------------|
|           | LTha                   | LThb    | LThb    | LAa                                                                                     |                | LAb            |                | LAc            |                |
|           |                        |         |         | Inner Laminate                                                                          | Outer Laminate | Inner Laminate | Outer Laminate | Inner Laminate | Outer Laminate |
| 1         | 0.0033                 | 0.0066  | 0.0099  | -10                                                                                     | -10            | -20            | -20            | -30            | -30            |
| 2         | 0.00224                | 0.00448 | 0.00672 | 30                                                                                      | 45             | 40             | 55             | 50             | 65             |
| 3         | 0.00305                | 0.0061  | 0.00915 | 10                                                                                      | 10             | 20             | 20             | 30             | 30             |
| 4         | 0.00224                | 0.00448 | 0.00672 | -30                                                                                     | -45            | -40            | -55            | -50            | -65            |
| 5         | 0.00305                | 0.0061  | 0.00915 | 10                                                                                      | 10             | 20             | 20             | 30             | 30             |
| 6         | 0.00178                | 0.00356 | 0.00534 | 45                                                                                      | 90             | 55             | -80            | 65             | -70            |
| 7         | 0.00178                | 0.00356 | 0.00534 | 45                                                                                      | 90             | 55             | -80            | 65             | -70            |
| 8         | 0.00279                | 0.00558 | 0.00837 | 10                                                                                      | 10             | 20             | 20             | 30             | 30             |
| 9         | 0.00203                | 0.00406 | 0.00609 | -30                                                                                     | -45            | -40            | -55            | -50            | -65            |
| 10        | 0.00254                | 0.00508 | 0.00762 | 10                                                                                      | 10             | 20             | 20             | 30             | 30             |
| 11        | 0.00178                | 0.00356 | 0.00534 | 30                                                                                      | 45             | 40             | 55             | 50             | 65             |
| 12        | 0.00229                | 0.00458 | 0.00687 | -10                                                                                     | -10            | -20            | -20            | 30             | -30            |

Table 6.2.2 Laminates Macro Design of GFVM advanced composite with different laminas alignment and thickness sets- Annacis Island Bridge and Second Chang River Bridge.

| Lamina No | Laminate Thickness (m) |         |         | Laminas Alignment angle (with respect to the Bridge longitudinal direction)<br>(Degree) |                |                |                |                |                |                |                |
|-----------|------------------------|---------|---------|-----------------------------------------------------------------------------------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|
|           | LTha                   | LThb    | LThb    | LAa                                                                                     |                | LAb            |                | LAc            |                | Inner Laminate | Outer Laminate |
|           |                        |         |         | Inner Laminate                                                                          | Outer Laminate | Inner Laminate | Outer Laminate | Inner Laminate | Outer Laminate |                |                |
| 1         | 0.0066                 | 0.0099  | 0.0132  | -10                                                                                     | -10            | -20            | -20            | -30            | -30            |                |                |
| 2         | 0.00448                | 0.00672 | 0.00896 | 30                                                                                      | 45             | 40             | 55             | 50             | 65             |                |                |
| 3         | 0.0061                 | 0.00915 | 0.0122  | 10                                                                                      | 10             | 20             | 20             | 30             | 30             |                |                |
| 4         | 0.00448                | 0.00672 | 0.00896 | -30                                                                                     | -45            | -40            | -55            | -50            | -65            |                |                |
| 5         | 0.0061                 | 0.00915 | 0.0122  | 10                                                                                      | 10             | 20             | 20             | 30             | 30             |                |                |
| 6         | 0.00356                | 0.00534 | 0.00712 | 45                                                                                      | 90             | 55             | -80            | 65             | -70            |                |                |
| 7         | 0.00356                | 0.00534 | 0.00712 | 45                                                                                      | 90             | 55             | -80            | 65             | -70            |                |                |
| 8         | 0.00558                | 0.00837 | 0.01116 | 10                                                                                      | 10             | 20             | 20             | 30             | 30             |                |                |
| 9         | 0.00406                | 0.00609 | 0.00812 | -30                                                                                     | -45            | -40            | -55            | -50            | -65            |                |                |
| 10        | 0.00508                | 0.00762 | 0.01016 | 10                                                                                      | 10             | 20             | 20             | 30             | 30             |                |                |
| 11        | 0.00356                | 0.00534 | 0.00712 | 30                                                                                      | 45             | 40             | 55             | 50             | 65             |                |                |
| 12        | 0.00458                | 0.00687 | 0.00916 | -10                                                                                     | -10            | -20            | -20            | 30             | -30            |                |                |

Table 6.2.3: Laminates Macro Design of GFVM advanced composite with different laminas alignment and thickness sets- Tataara Bridge.

### **6.2.2 Fiber Fraction Effect on Static Deformations and Natural Frequencies of the HLSCSB:**

The effects of the Volumetric Fiber Fraction  $V_f$  variation in the range between 0.25 and 0.5 on the deck displacements and first natural frequencies, for all the four deck models and the three hybrid bridges corresponding to the three reference bridges, are investigated. The minimum value of this range is chosen such that the deck is capable to support the static loads and the resulting deformations and frequencies are within the acceptable ranges. On the other hand, the maximum value of the range is decided by the economical considerations, where the change of the structural parameters becomes small. The displacements and frequencies shown here are: (i) displacement U in lateral direction (parallel to X axis), (ii) V in the vertical direction (parallel to Y axis), (iii) W in the longitudinal direction (parallel to Z axis), (iv) the first Vertical flexural Natural Frequency (1<sup>st</sup> VNF), (v) the first Torsional Natural Frequency (1<sup>st</sup> TNF), and (vi) the first Horizontal flexural (or transverse bending) Natural Frequencies (1<sup>st</sup> HNF).

In order to avoid any coupling or interaction between the different macro or structural levels variables and the variable under consideration, i.e. the fiber fraction, it is necessary to assume: (i) the Laminas Alignment design type a, or LAa (or LDFA-a), is the macro design for all the studied cases, (ii) the laminate thickness, and hence the related lamina thicknesses, is the minimum for every case, or LTha (Table 6.2.3), (iii) the pylons are stiff, and (iv) the Carbon Cable Radii (CCR) of the three hybrid bridges are assumed to be constant and equal to 0.10m, 0.125m, and 0.15m for the Anacis Island, the Second Chang River Bridge and the Tatara bridge respectively.

It has been found that all the displacements are reduced and the natural frequencies are increased with the increase of the volumetric fiber fraction. While these changes are small with the shortest bridge central span (or Anacis Island Bridge central span), they are at their maximum level with the longest bridge span (or Tatara Bridge central span). The stiffness increase of the lamina due to the increase of the fiber content of its composite material, and hence the stiffness increase of different laminates explains the changes in the deformations and the natural frequencies. This effect becomes apparent with the Tatara Bridge. The deck deformations become more sensitive to the increase in the stiffness and the bridge natural frequencies are more sensitive to the resulting increase

in the deck stiffness to mass ratio. Model 3 shows higher sensitivity to  $V_f$  increase in general, while Model IV appears to be the least sensitive one and Models 1 and 2 lie in between them. The cross sectional area of the laminated part of Model III is the maximum while it is the minimum in Model IV. Model I has the second maximum cross sectional area of the laminated box girder and Model II has the second minimum.

For the hybrid system corresponding to the Tatara Bridge, Figure 6.2.2.1 shows the high reduction in the maximum lateral displacement of the deck of Model 3 with the increase of  $V_f$  in comparison with the other three models, that show less reduction. The Anacis Island and the Second Chang River Bridges give very similar patterns with lower variations.

For the maximum vertical displacement  $V$  (or the deflection), and for the Anacis Island Bridge, Models I, II and III show higher reduction in their maximum deflection (displacement  $V$ ) with the increase of  $V_f$  than Model 4 in General. This reduction pattern becomes significant, especially with Model III, as the bridge central span become very long and extremely long with the Second Chang Bridge, and the Tatara Bridge (Figure 6.2.2.2).

For the longest examined central span (Tatara Bridge), all models show a reduction in their longitudinal displacements ( $W$ ) as ( $V_f$ ) increases (Figure 6.2.2.3), however Model 3 shows very high reduction. Similar reduction patterns with less variation are related to less central span lengths of the other two reference bridges, the Annacis Island Bridge and the Second Chang Bridge.

For the Tatara Bridge, the first Vertical Flexural Natural Frequency (1<sup>st</sup> VNF) of Models I, II, III show high increase with increase of  $V_f$ , while Model IV shows much less rate of increase (Figure 6.2.2.4). Similar patterns of the VNF-increase, but with less variation, are related to the shortest bridge central span lengths of the other two reference bridges.

The first Torsional Natural Frequency (1<sup>st</sup> TNF) of Models I, II and III increases very slightly as  $V_f$  increases for the Anacis Island and the Second Chang River (Figure 6.2.2.5) Bridges. However, they show significant increase as the bridge span becomes very high with the Tatara Bridge (Figure 6.2.2.6). Model IV always gives very high Torsional Natural Frequencies and usually their values are incomparable with the other three models for all bridge central span lengths. On the other hand, and for all the studied cases, the first Horizontal flexural Natural Frequencies (1<sup>st</sup> HNF) show very slight

increases with increase of  $V_f$ , and Figure 6.2.2.7 shows this type of variation for the Tatara Bridge.

From these results, it can be concluded that: (i) Since all the deformations and natural frequencies are affected very slightly with the increase of the Volumetric Fiber Fraction of Model IV, then  $V_f$  of this model can be reduced to a minimum value, provided that all the static and dynamic characteristics are among the required properties for a specified central span length, (ii) all the deformations and the natural frequencies of Model 3 are highly changed with the increase of  $V_f$  and hence deciding an optimum value is very difficult and needs a careful study of the resulting bridge system characteristics, (iii) Models I and II give low variation of all deformations and natural frequencies except (TNF), hence the required ratio of the TNF to the VNF is the base to decide the optimum  $V_f$  for those models.

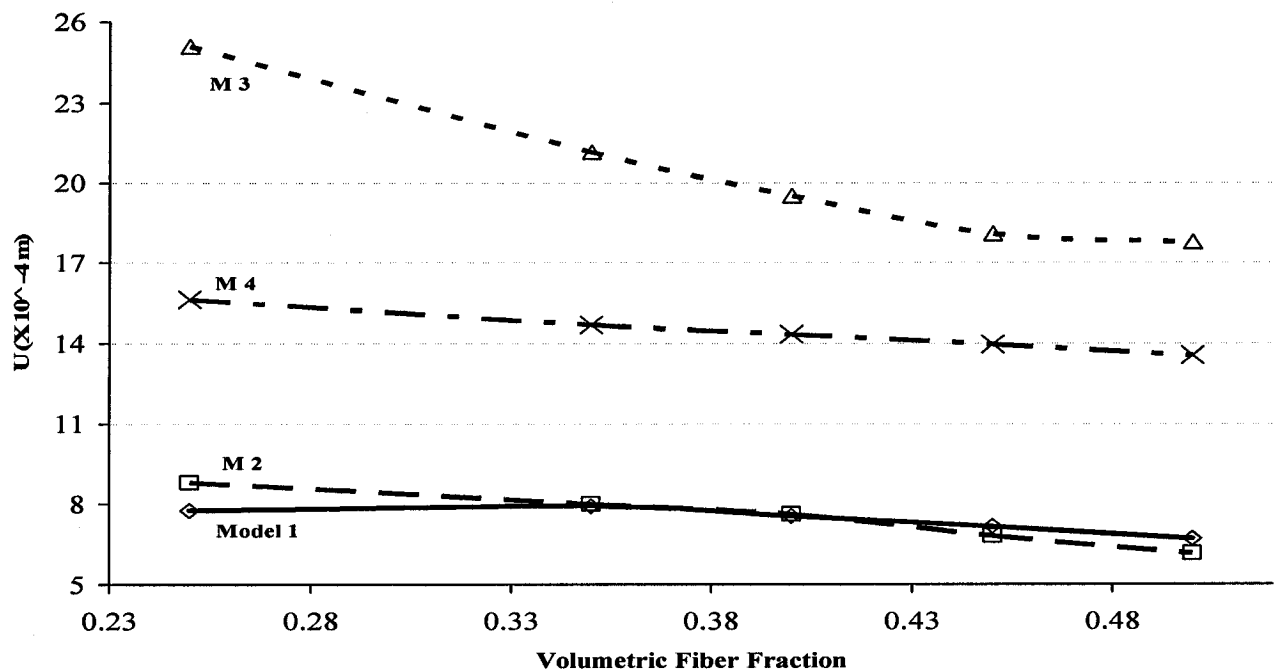


Figure 6.2.2.1 Fiber Fraction Effect on maximum lateral displacement ( $U$ ) of the deck- LDFA-a, Tha, CCR 0.150m, Tatara Bridge

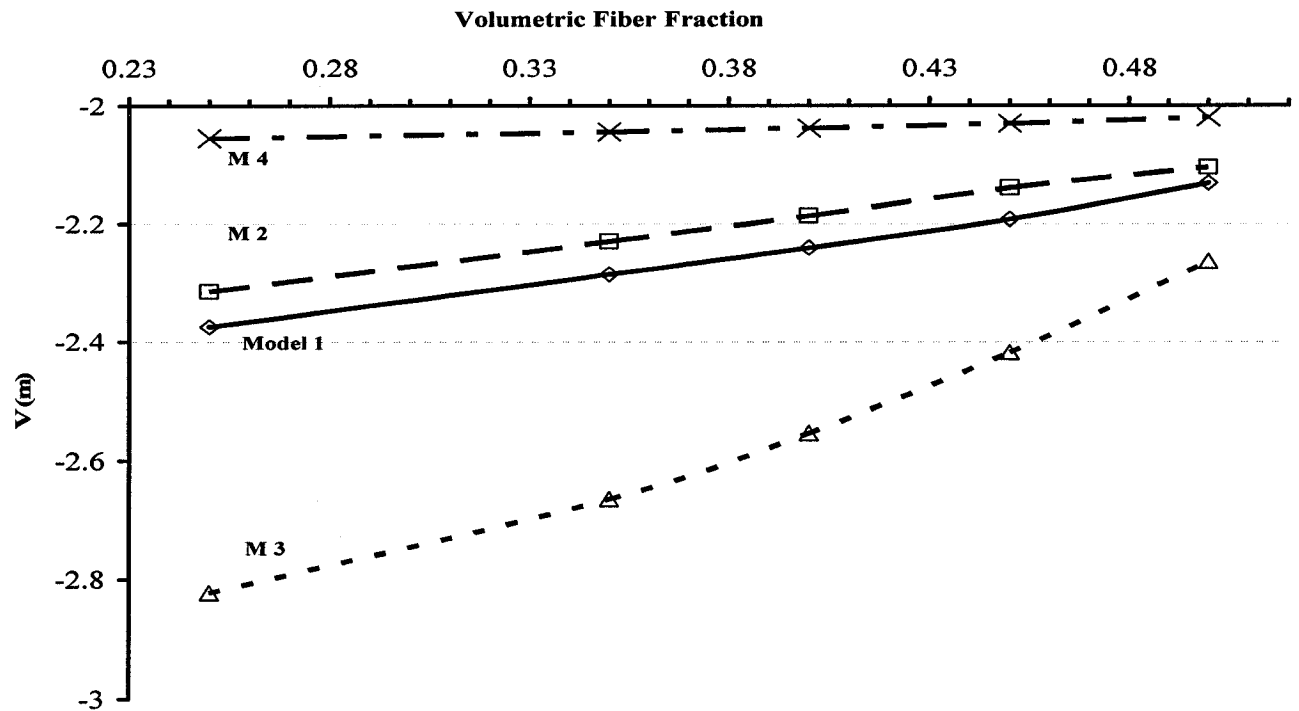


Figure 6.2.2.2 Fiber Fraction Effect on maximum Vertical displacement (V) of the deck, or the deflection, LDFA-a, Tha, CCR 0.150m- Tataru Bridge

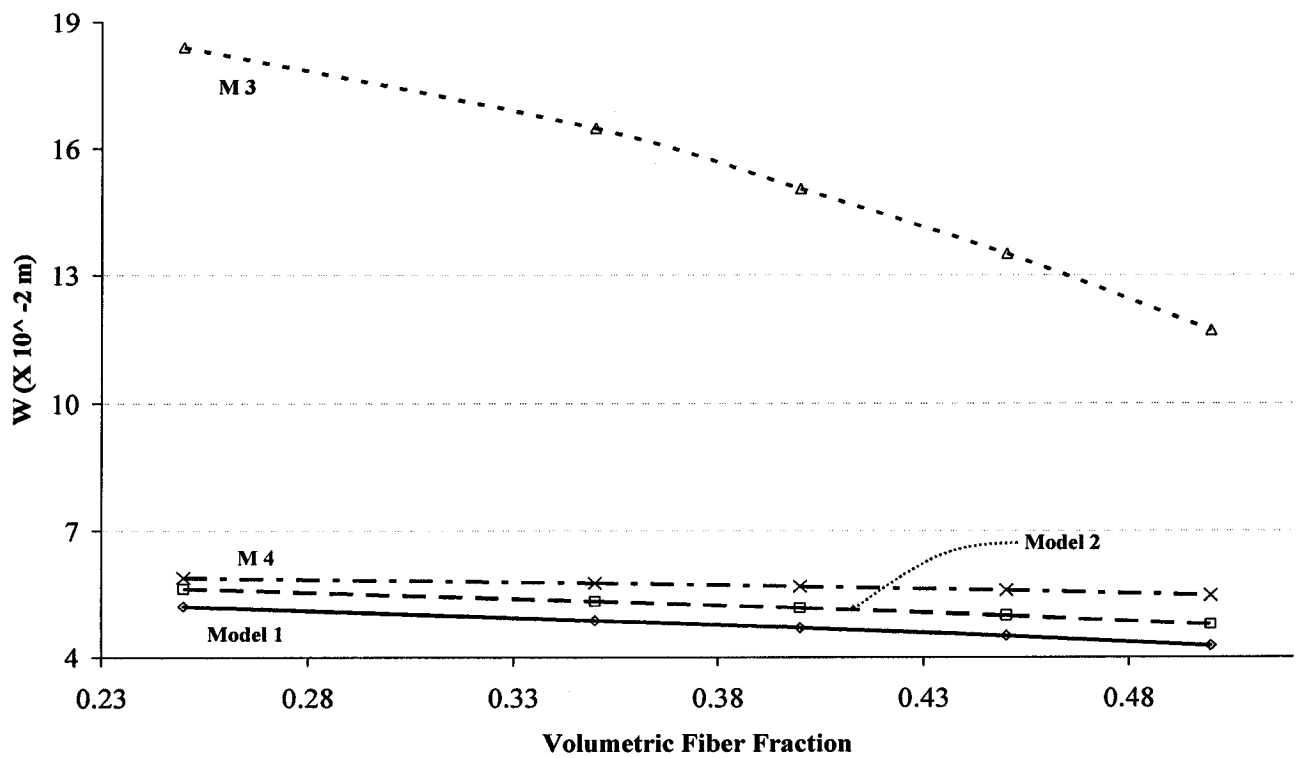


Figure 6.2.2.3 Fiber Fraction Effect on maximum longitudinal displacement (W) of the deck, LDFA-a, Tha, CCR 0150m- Tataru Bridge

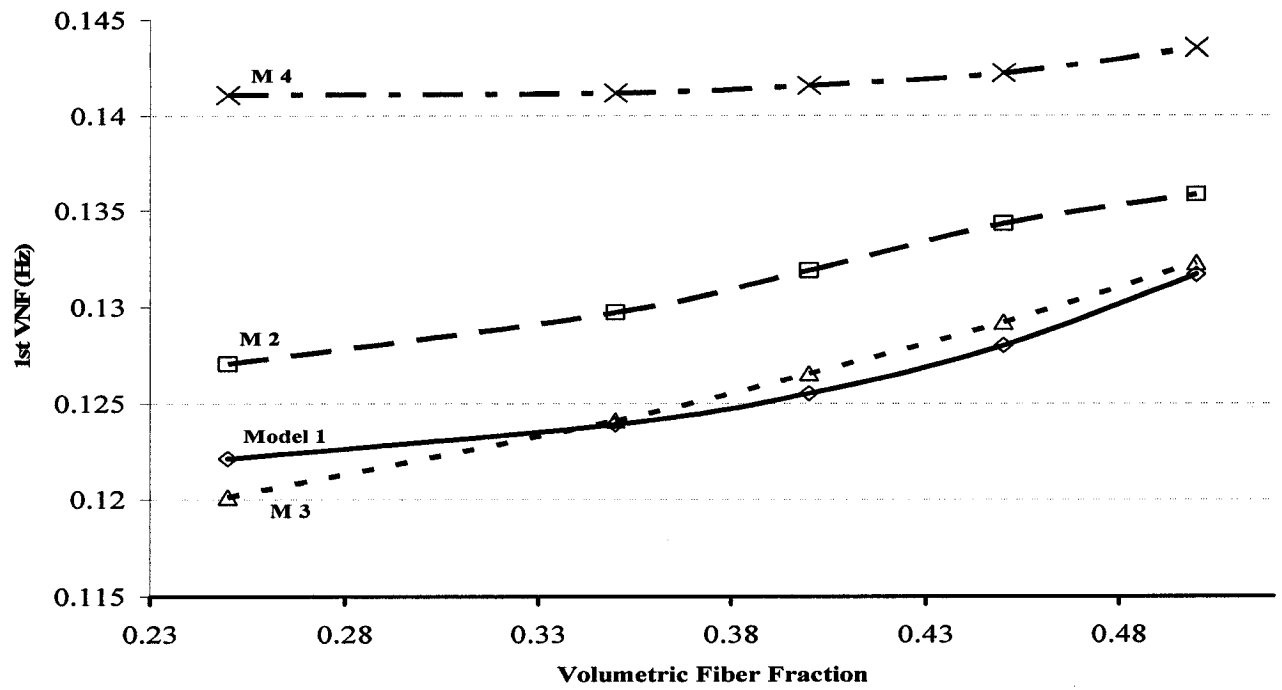


Figure 6.2.2.4 Fiber Fraction Effect on First Vertical Natural Frequency, LDFA-a, Tha, CCR 0.150m- Tatara Bridge

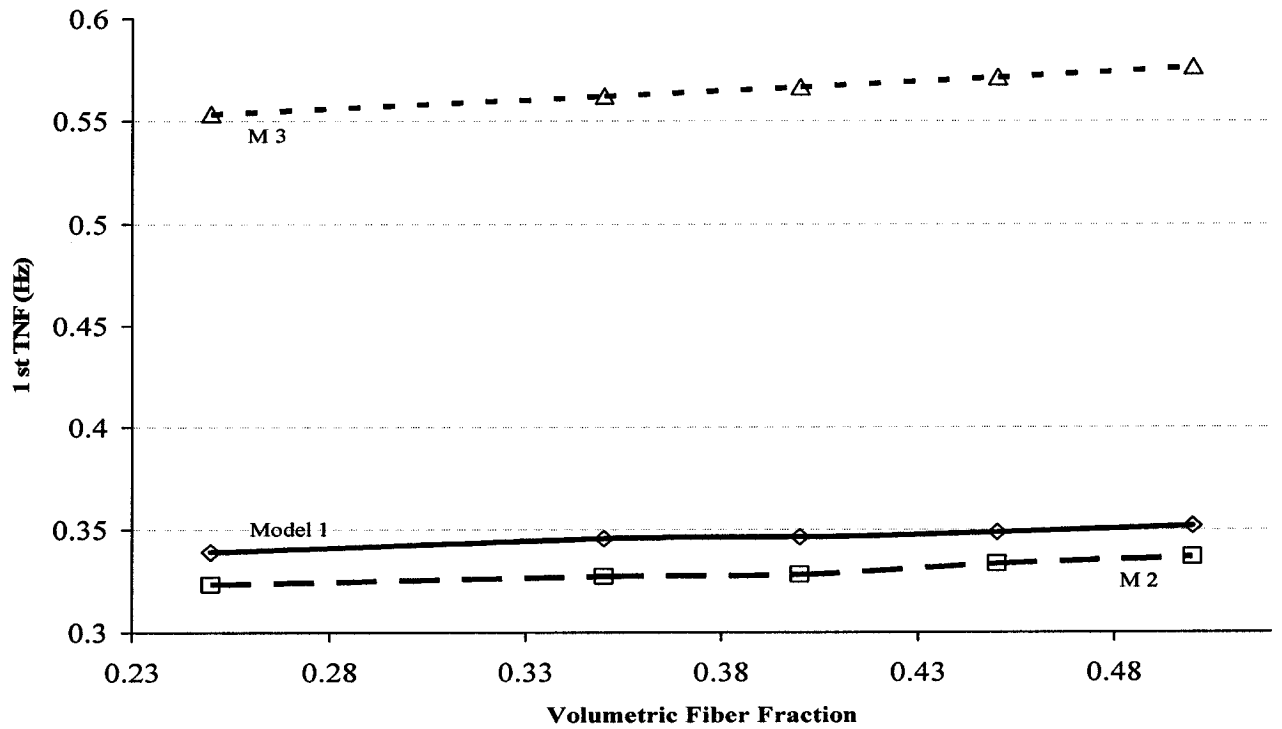


Figure 6.2.2.5 Fiber Fraction Effect on First Torsional Natural Frequency, LDFA-a, Tha, CCR125- Second Chang River Bridge



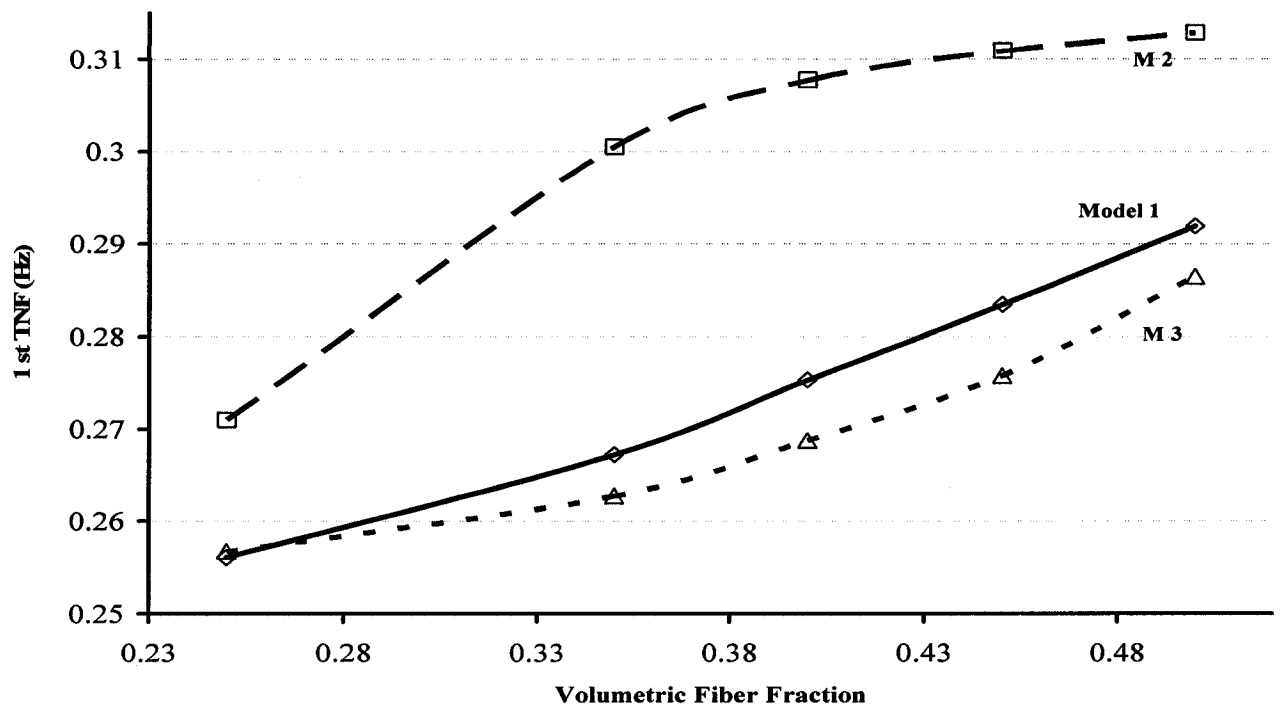


Figure 6.2.2.6 Fiber Fraction Effect on First Torsional Natural Frequency, LDFA-a, Tha, CCR150- Tataara Bridge

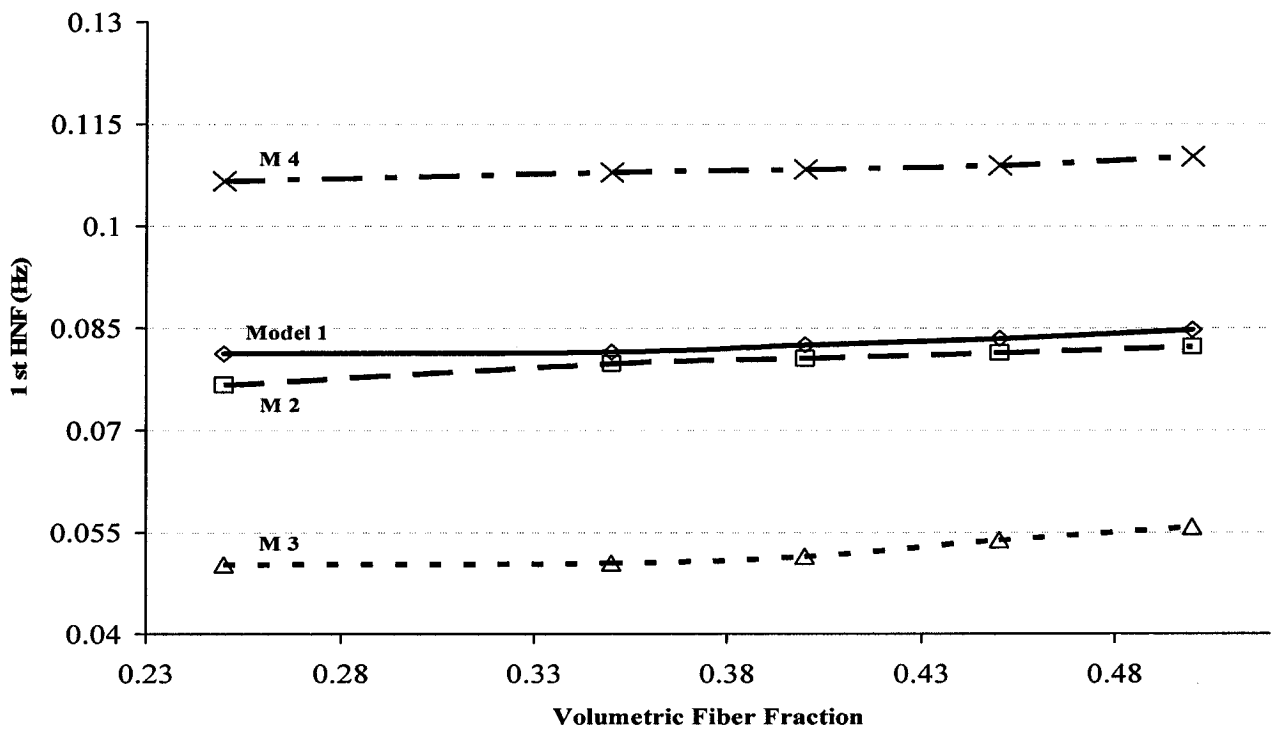


Figure 6.2.2.7 Fiber Fraction Effect on First Horizontal Natural Frequency, LDFA-a, Tha, CCR15- Tataara Bridge

### **6.2.3 Effect of Micro/Macro Structure Design on the Distribution of Deck Displacements and the Failure Function of Different Laminates, and Principal Stresses in the Chopped FRP:**

Samples of the displacements and Tsai Hill Failure Function (THFF) distributions over the deck are shown in this section. It has been observed that the distributions of these variables are generally unsymmetrical although the structure and the external applied loads are symmetrical. Any change in the laminate thickness or the laminate dominant fiber alignment would slightly change these distributions. This can be explained by the laminated box material anisotropy and the deck section lightweight. However the distributions are different from one model to another and are significantly affected by the increase of the CFRP cable radius and the central-span length (and the central span to the side span ratio) of different bridges. The cables are serving as elastic supports of the deck, and the increase in their section radius (hence their stiffness, strength and pretension stress...etc) would lead to a significant change in the distribution of the decks' deformations. The central span gives the major loads on the bridge system, and its loads are balanced by the side spans through the cable stays. The cable stayed-bridge system is sensitive to the increase of the central span length due to the increase of the loads applied on the entire bridge system, and the balance of the central span to side span becomes sensitive. This is apparent in the proposed bridge system with the critical load case LLA, where the central span is loaded by the full traffic loads and the side spans are not. The ratio of the central span to the side span length is a major variable of the mentioned load balance. As a result, the changes of this critical load balance of the bridge system leads to major changes in the distribution of different variables. On the other hand, the effect of the deck dead loads are small for the proposed hybrid long span cable stayed system in contrast to the traditional bridge system, where it plays the dominant rule in the mentioned load balance.

The lateral displacement in the x-direction  $U$  has unsymmetrical distributions, which are different from model to model. These distributions are also changed with the increase of the bridge span. For a central span of 465 (the Annacis Island Bridge), the maximum values for  $U$  are mainly covering a wide unsymmetrical zone close to the mid-span center. This "maximum  $U$  zone" is vanishing as the span is increased up to 890 m (the

Tatara Bridge) for all models. With the very long span case, the distribution has local regions of extreme high or low  $U$  magnitudes close to the deck external supports and the cable anchorage points. Figures 6.2.3.1 to 6.2.3.3 show sample contours of the displacement in the  $x$ -direction  $U$ , for the top surface of the deck of Models I for the Annacis Island, the Second Chang River and the Tatara Bridges.

The vertical displacement  $V$  (or the deflection) has symmetrical distributions for all models and all spans and its values are much higher than the two other displacements ( $U$  and  $W$ ). This symmetrical pattern has a maximum negative zone at the mid-span and two maximum positive zones between the two pylons and the bridge ends. The two maximum zones are moving apart toward the two ends as the central-span increases, accomplished with a successive increase in magnitude of all the three maximum zones. Indeed, all the applied static loads, including the bridge components self loads, are gravity loads, which are parallel to the  $Y$ -axis. Since the deck longitudinal dimension is significantly higher than the other two dimensions, and accordingly all the accumulated loads levels over the length are very high, therefore: (i) the magnitude of the deflection is very high, while other displacements are low, and results mainly from the anisotropy of the laminated material of the deck, (ii) the distribution of the deflection is directly related to the applied loads and the deck flexural stiffness in the longitudinal direction, (iii) it is also related to the bridge stay-system arrangement, the ratio of the central to the side spans, the cable pretension and the pylon stiffness. Figures 6.2.3.4 and 6.2.3.5 show sample results of the displacement  $V$  distribution, for the top surface of the deck slab of Model I for the Second Chang River and Tatara Bridges.

The longitudinal displacement  $W$  has unsymmetrical distributions and tends to show a “waved” pattern, formed from high and low values, at approximately  $45^\circ$  from the bridge longitudinal center line. This pattern becomes more apparent as the bridge central span is greatly increased or greatly decreased. It is also noted that the maximum positive or negative  $W$  values are at either of the deck ends (in the traffic direction). The longitudinal displacements distribution pattern can be imagined as if the material of the deck is moved in waves, inclined approximately at  $45^\circ$  to the traffic direction, towards the bridge center. The movement is faster far from the bridge centre, while it is slower and its fluctuation is vanished and mixed in a uniform stream close to the bridge centre. The central to side spans ratios are 2.4 for the Annacis Island Bridge, 2.0 for the Second Chang River Bridge

and 2.8 for the Tataru bridge. With the high ratios, the deck seemed as if it was strongly stretched by the increasing loads of the central span. However the deck supports restrict this stretching action. The deck material anisotropy distorts the stretched laminates and gives the inclined and wavy shapes in a way analogous to the currents and eddies in a rocky shore during the tidal movement. Figure 6.2.3.6 shows sample contours of the displacement in z-direction (W), for the top surface of the deck slab model III, the Tataru Bridge.

At the top laminate of the laminated box girder, the Tsai-Hill Failure Function follows approximate-symmetrical distributions for all models and all spans. This distribution pattern has two zones of maximum values close to the bridge two ends and a wide zone of minimum values covers the central span. The deck supports are increasing the stresses as expected and hence the distribution shows the maximum zones. Model III shows an expansion in the area of the two maximum zones and a reduction in the magnitude of minimum zone. This can be explained by the relatively higher stiffness of this model deck. Figure 6.2.3.7 shows sample contours of the THFF, for the top laminate of Laminate Box Girder of Model I of the Tataru Bridge.

At the bottom laminates of the laminated box girder, the Tsai-Hill Failure Function has an unsymmetrical distribution. This distribution pattern has two zones of maximum magnitudes close to the deck ends and a third local maximum zone located close to the deck center, with values lower than the two extreme zones. The maximum deflection of the bridge deck is located at the mid zone of the central span. Also at the same zone the high tensile stress field due to the maximum tensile stress of the cable stays is located at this central zone. The effect of both, the high deflection and the related high rotations and the high tensile stress field due to cable tension, results in higher stresses on the lower face of the deck box girder than at the top surface. This is apparent by the local rise of THFF in this zone. The minimum magnitude zones are distributed between the zones of maximum magnitude. Model III shows different locations for the high magnitude zones (Figure 6.2.3.10), specifically in the cylindrical sector connecting the horizontal and the inclined parts of the bottom laminate, at the two ends of the bridge. These cylindrical sectors are variable and can switch their stress field from tension to compression. The axial strength of the lamina in compression is very low (Table 4.4), and as a result, THFF

rapidly increases at these zones. Figures 6.2.3.8 shows sample contours of the THFF, for the bottom laminate of the Laminate Box Girder of Model I of the Tatara Bridge.

A different pattern for the THFF at the vertical laminates of the laminated box girder for each of the three models (with box girders) is observed. For Model I, the two maximum magnitude “spot” zones are close to the two deck ends in the longitudinal direction. For Model II, the spots of the maximum THFF are wider than those of Model I and lie closer to the longitudinal bridge centerline. Figures 6.2.3.9-a shows sample distribution of the Tsai-Hill Failure Function, for the vertical laminates of Laminate Box Girder of Model I of the Tatara Bridge, and Figure 6.2.3.9-b shows sample distribution of the THFF, for the vertical laminates of Laminate Box Girder of Model II.

Tsai-Hill Failure Function distribution patterns can be related to the bridge deck boundary conditions, the distribution of the axial stress fields in the deck and the deck section geometry. For all the three types of laminates, top, bottom and vertical, the zones of maximum THFF are usually located between the deck supports and the pylons, while the minimum THFF are distributed at the central part of the bridge deck. Close to the bridge centre, the deck is elastically supported by the cables and affected by a tensile field resulting from the high cables tension. Since the tensile strength of the lamina is the highest among its other strengths, the expected failure function is hugely reduced. However, the high levels of failure function at the support zone can be explained by the existence of the compression fields of the pylon zones that moved towards the deck ends because of the intermediate supports and the high ratio between the central span to the side spans of the deck. The compression strength of the lamina is low and accordingly the failure function rise to the maximum. High THFF zones are also related to the rounded edges of the Model III bottom laminate and the vertical laminates close to the two sides of the deck cross section of models I and II. This can be clarified by the stress concentration that has to be avoided by a proper choice of the side slope and the curvature of such critical parts. However, the aerodynamic-section-shape and deck dimensions requirements for minimum vortex shedding vibration could not allow for such changes to the section sides.

The maximum Principal Stress distributions of the Chopped FRP “deck slab” of Models I, II, and III are symmetrical and there are only two maximum regions located between the pylon and the bridge end for both sides. These regions are moving towards the two

bridge ends as the bridge span is increased. The values of maximum Principal Stress drop to a very low level out of the two extreme maximum regions. On the other hand, The Minimum Principal Stress distributions are again symmetrical and have only two very low value regions located between the pylon and the bridge end for both sides. Elsewhere, a sequence of extreme high values regions and low value regions are shown for all models and all spans. Figures 6.2.3.11 and 6.2.3.12 shows the distribution of Maximum and Minimum Principal Stresses at the Chopped FRP “Deck Slab” of Model I, of the Tatara Bridge.

For Model IV, again two extreme maximum regions of the Principal Stresses at the Chopped FRP part on the top surface of the two side boxes of the section are observed, while very low values are distributed elsewhere. Also with the very long bridge central span, those two regions are moved towards the deck ends in the longitudinal direction. On the other hand, the Minimum Principal Stress distribution is again symmetrical and has only two very low value regions located between the pylon and the bridge end for both sides, but at the top surface of the section. Elsewhere, a sequence of extreme high value regions and high value regions are observed, but now located at “the slab-grid” part of Model IV together with the top surface of the two side-boxes. Figure 6.2.3.13 and 6.2.3.14 show the distribution of Maximum and Minimum Principal Stresses at the Chopped FRP part of Model IV of the Tatara Bridge.

If the “maximum” and the “minimum” principal stress distributions of the chopped FRP parts of the deck sections are combined in one image, (For instance, Figures 6.2.3.11 – 6.2.3.12 and 6.2.3.13 – 6.2.3.14), the resulting image can be compared to the distributions of the deflection and the tensile stresses in the cable stays (which is maximum far from the pylons and minimum close to the pylons) of a specified case. Then, the principal stresses (PS) of the chopped FRP look analogous to the bending stresses of a beam on multi-elastic supports. For Models I, II and III, the chopped FRP “deck slab” principal stress distributions are analogous to the bending stress distribution in the top fiber along the beam. For Model IV, the bending stresses of all the beam section along the beam length can be compared to the PS at different levels (or horizontal slices) of the deck section.

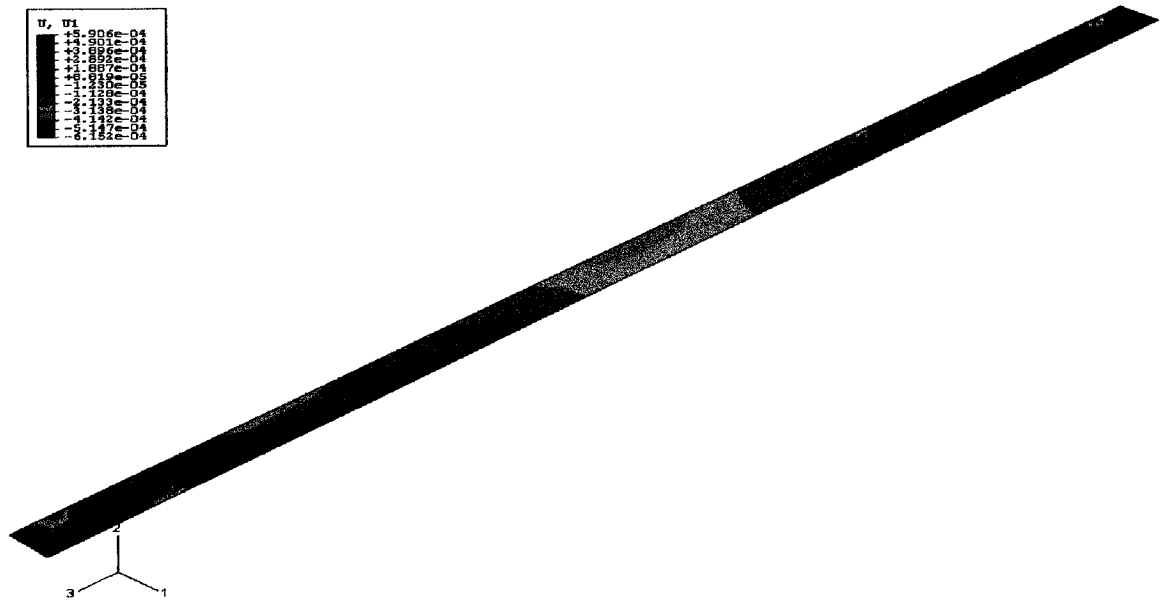


Figure 6.2.3.1 Distribution of displacement  $U$  (m) in top surface of the deck, Model I, Stiff pylon, LDFA-a, LTha, CCR 0.100 m and Live Load A, Anacis Island Bridge.

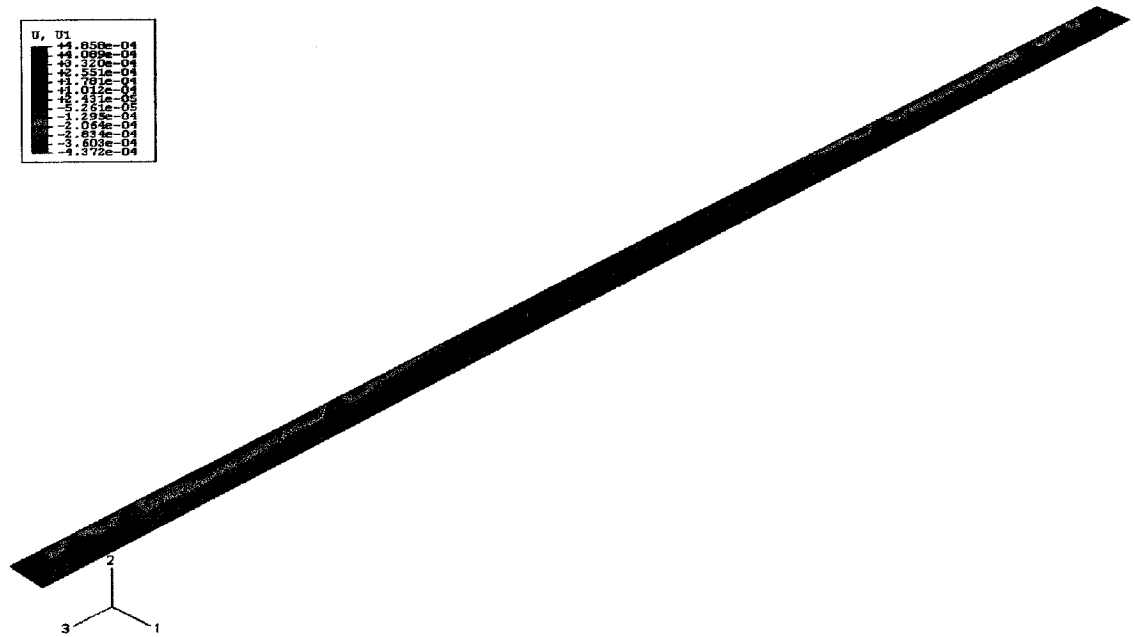


Figure 6.2.3.2 Distribution of displacement  $U$  (m) of the top surface of deck A, Model I, Stiff pylon, LDFA-a, LTha, CCR 0.125 m, LLA, the Second Chang River Bridge.

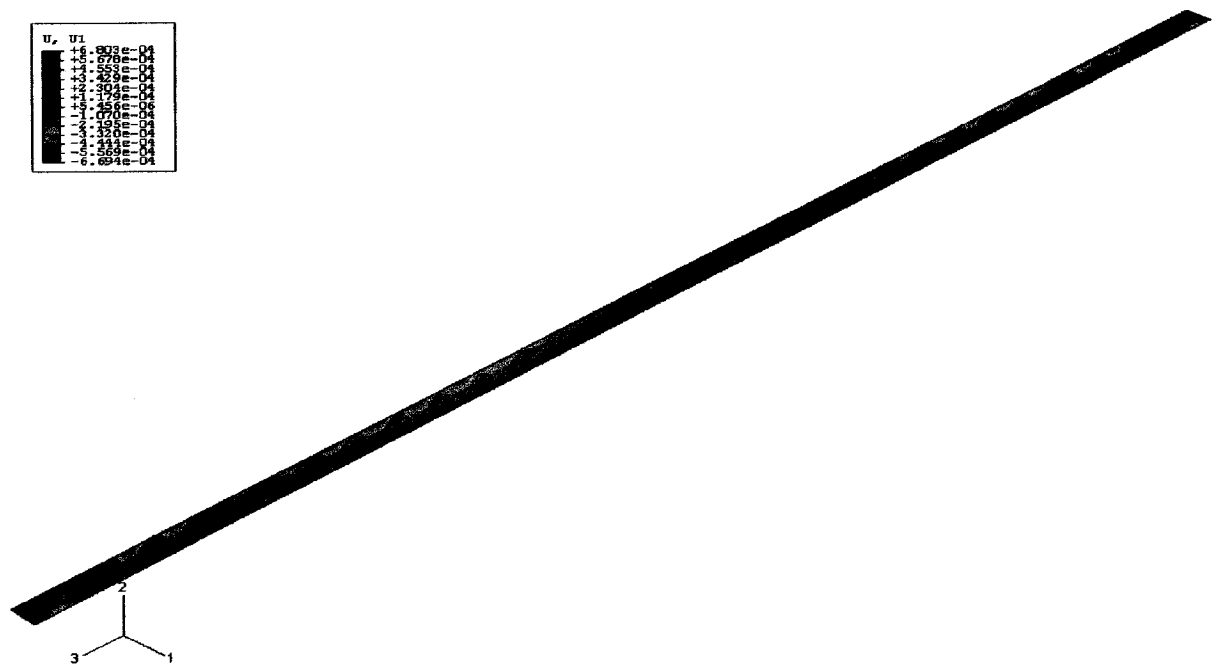


Figure 6.2.3.3 Distribution of displacement  $U$  (m) of the top surface of the deck, Model I, Stiff pylon, LDFA-a, LTha, CCR 0.150 m, LLA, the Tatara Bridge.

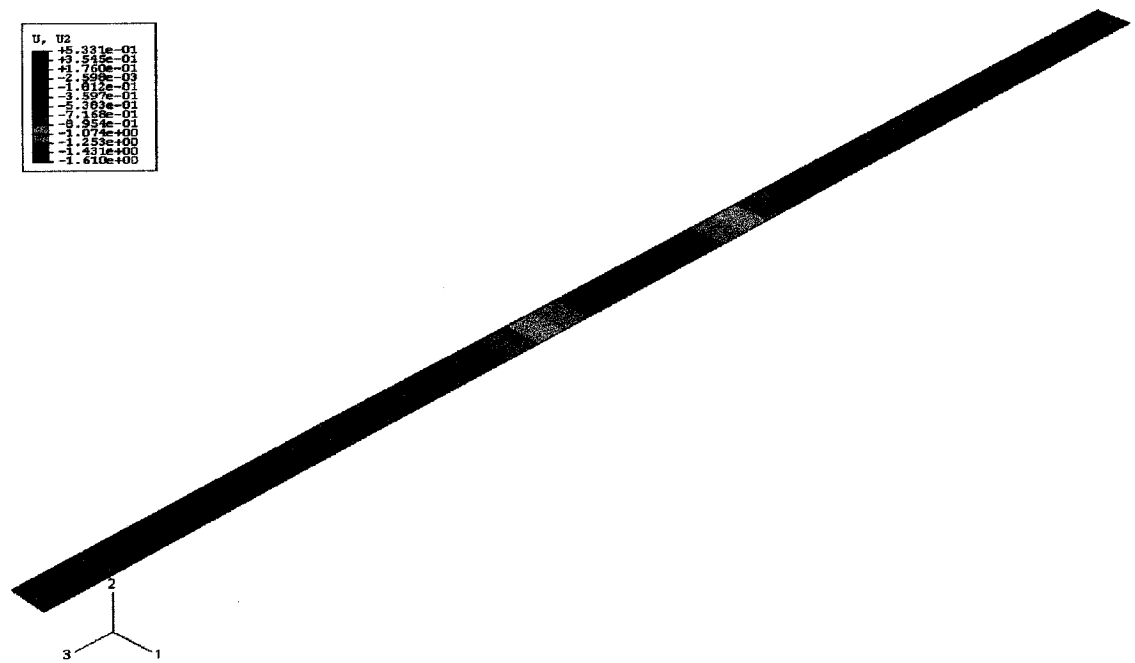


Figure 6.2.3.4 Distribution of displacement  $V$  (m) of the top surface of the deck, Model I, Stiff pylon, LDFA-a, Tha, CCR 0.125 m and LLA, the Second Chang River Bridge.



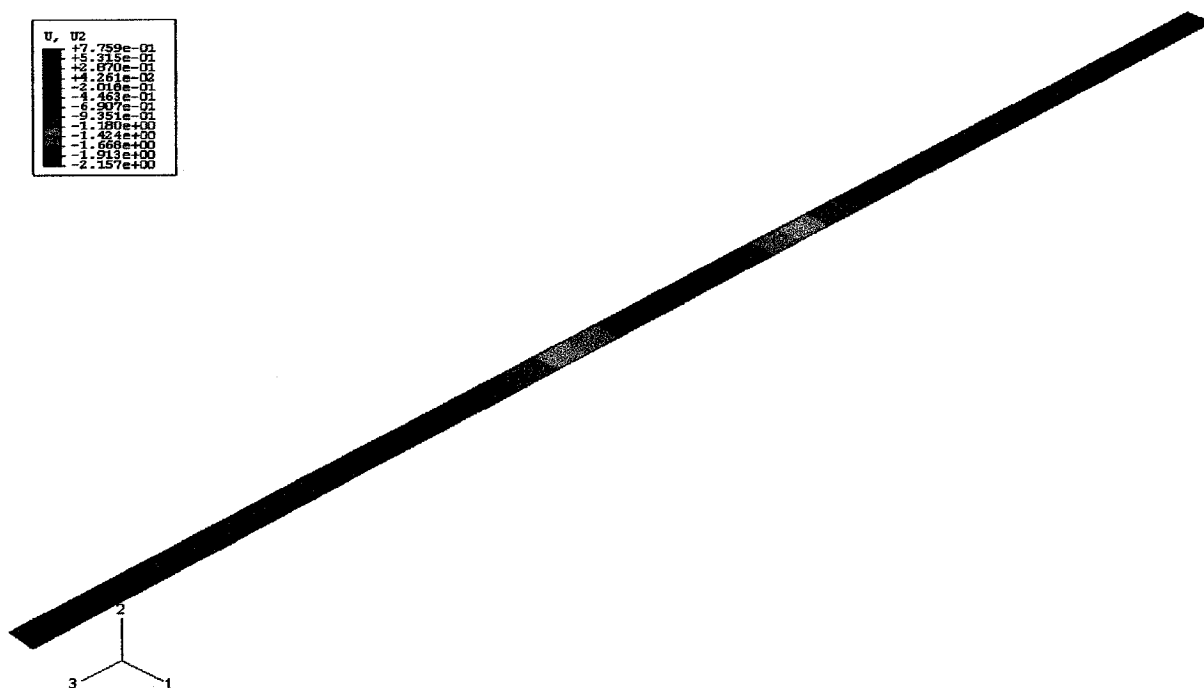


Figure 6.2.3.5 Distribution of displacement V (m) of the top surface of the deck, Model I, Stiff pylon, LDFA-a, Tha, CCR 0.150 m and LLA, the Tatar Bridge.

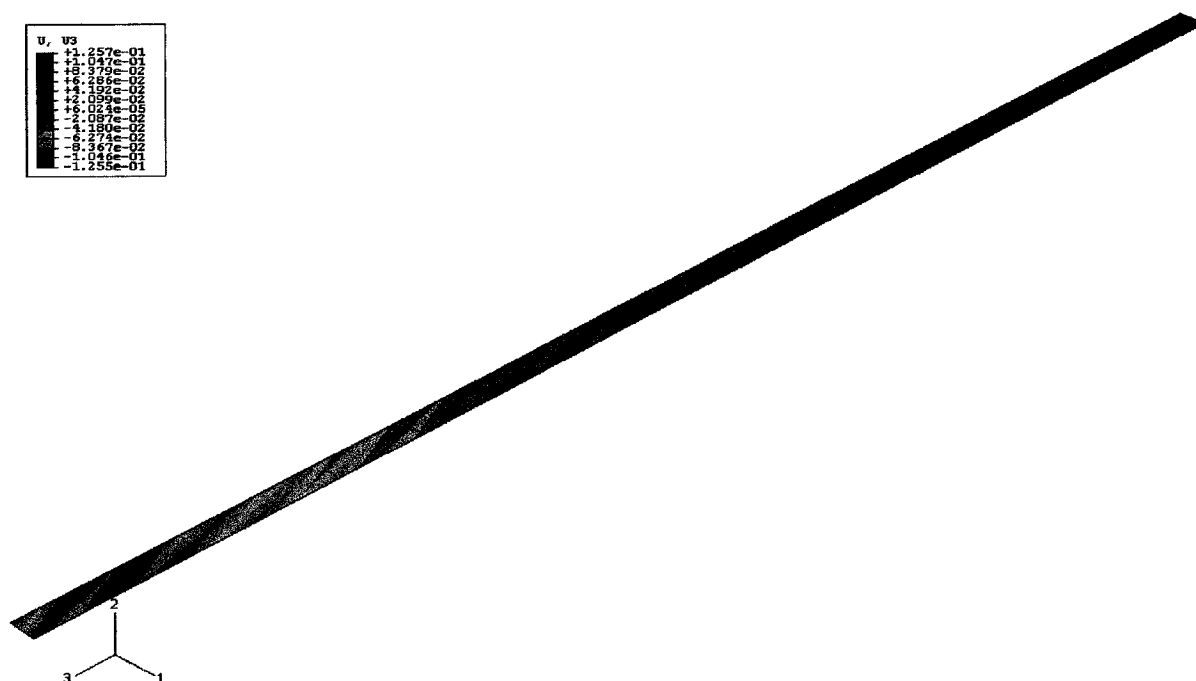


Figure 6.2.3.6 Distribution of displacement W (m) of the top surface of the deck, Model III, Stiff pylon, LDFA-b, LTha, CCR 0.150 m and LLA, the Tatar Bridge.



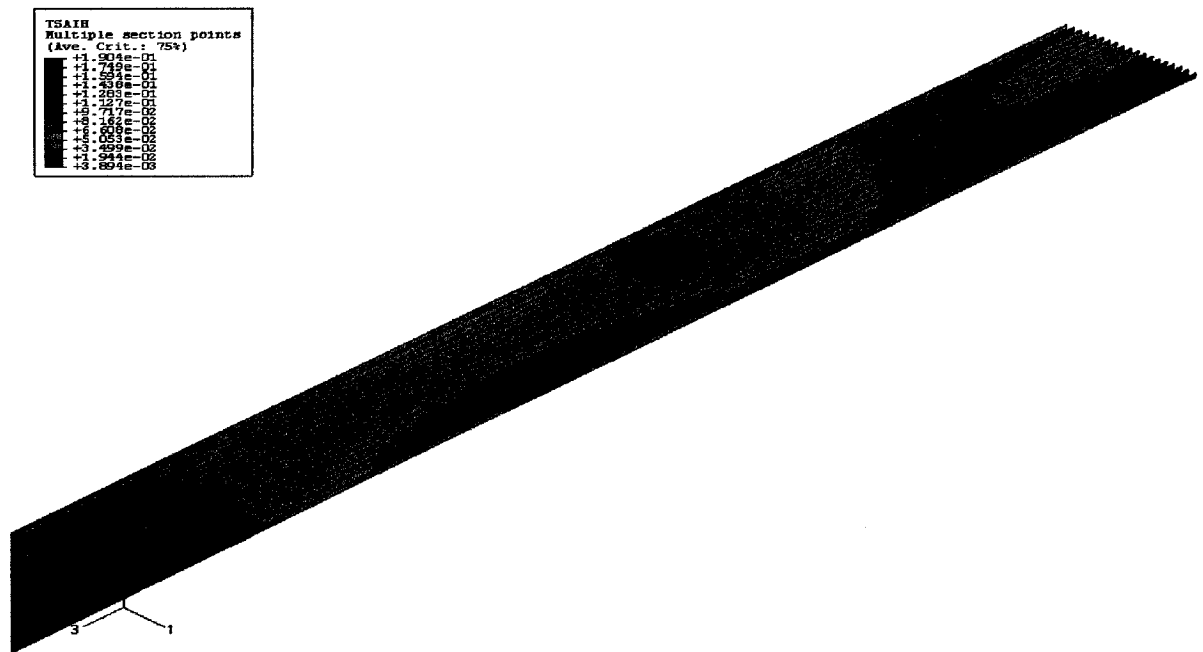


Figure 6.2.3.9-a Distribution of Tsai-Hill Failure Function of the vertical Laminates of the Laminated Box Girder, Model I, Stiff pylon, LDFA-b, LTha, CCR 0.150 m and LLA, the Tataru Bridge.

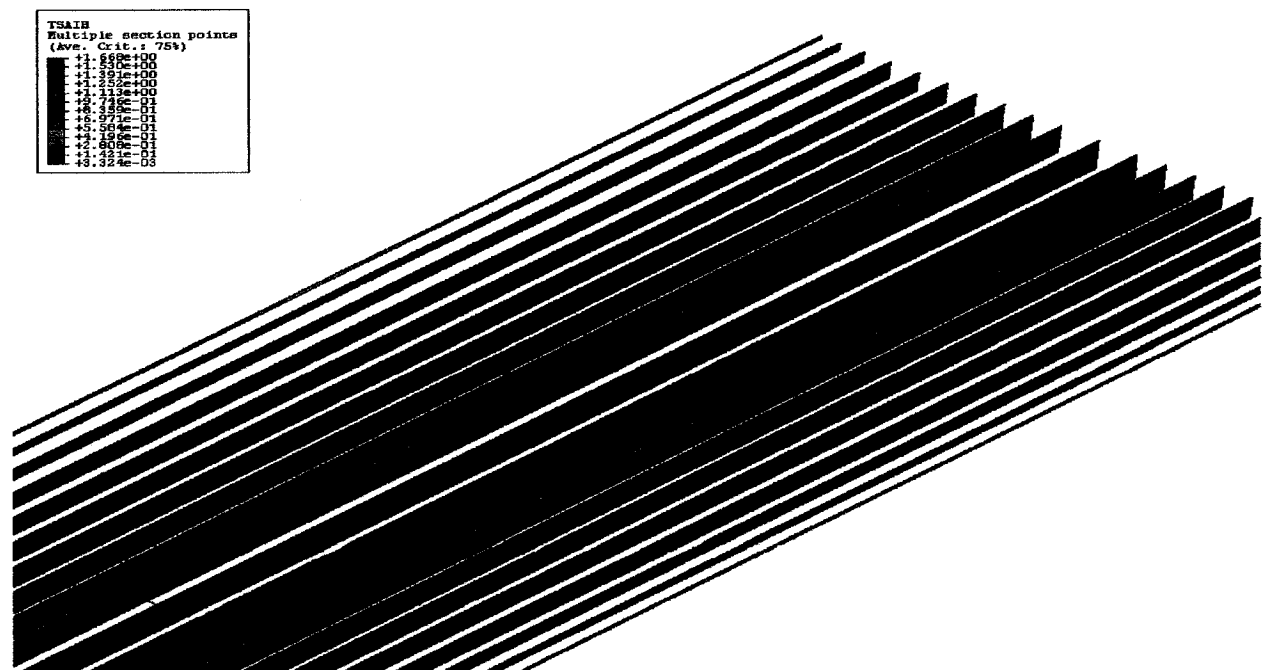


Figure 6.2.3.9-b Distribution of Tsai-Hill Failure Function of the vertical Laminates of the Laminated Box Girder, Model II, Stiff pylon, LDFA-b, LThb, CCR 0.100 m and LLA, the Second Chang River Bridge.



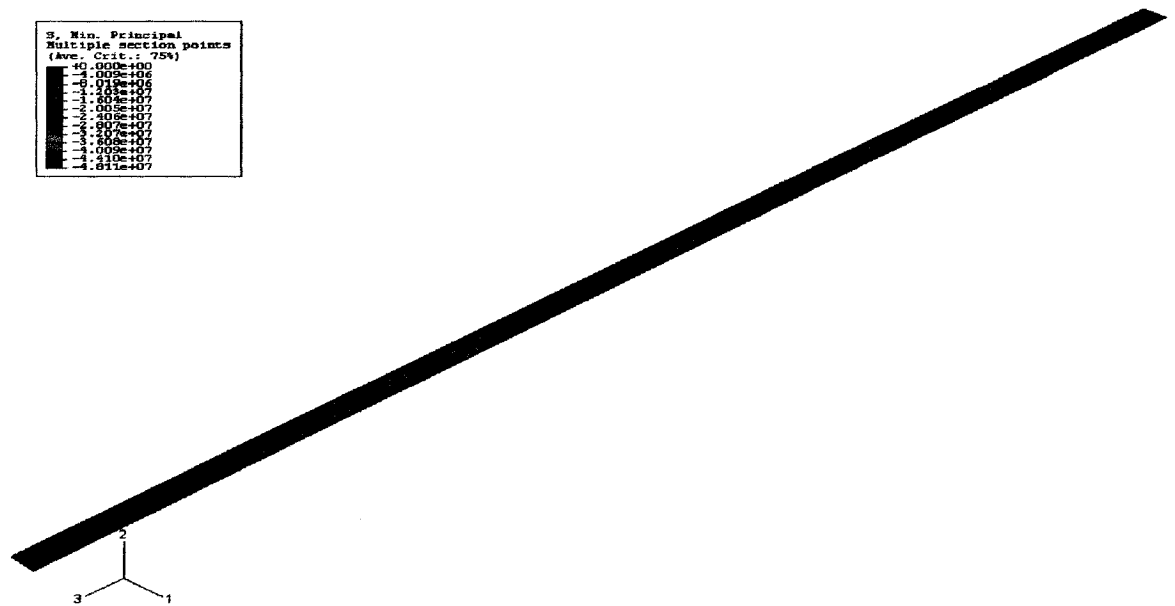


Figure 6.2.3.12 Distribution of the Minimum Principal Stresses ( $\text{N/m}^2$ ) at the Chopped FRP "Deck Slab", Model I, Stiff pylon, LDFA-a, LTha, CCR 0.150 m and LLA, the Tatara Bridge.

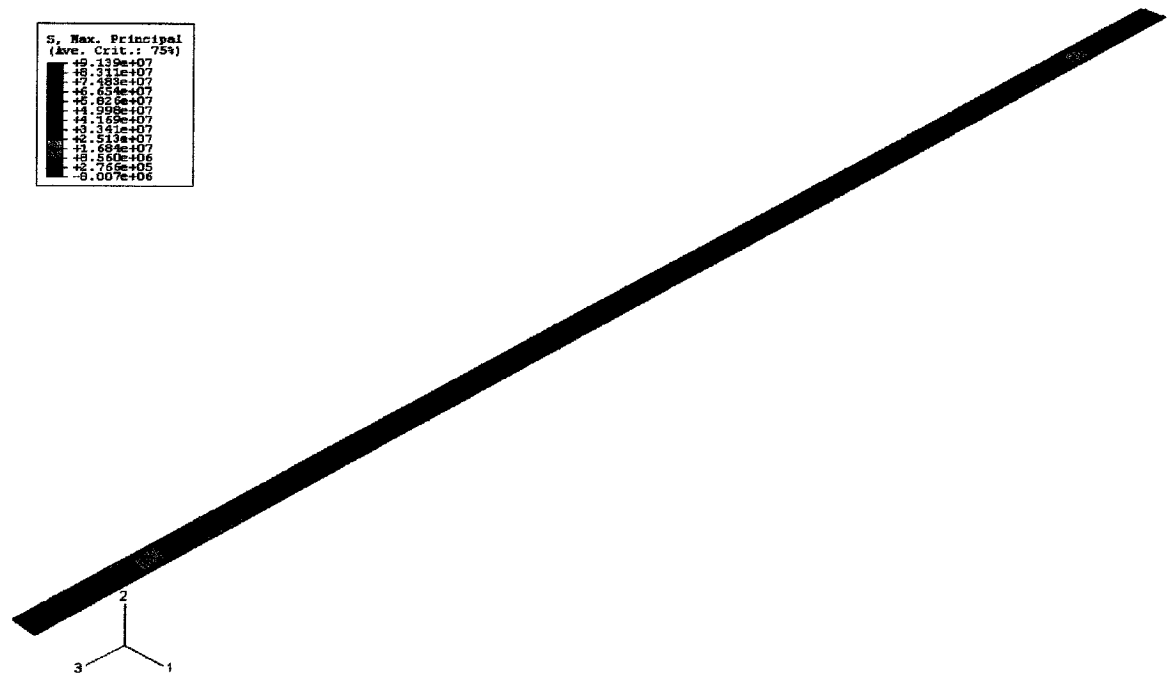


Figure 6.2.3.13 Distribution of the Maximum Principal Stresses ( $\text{N/m}^2$ ) at the Chopped FRP part of Model IV, Stiff pylon, LDFA-a, L Tha, CCR 0.150 mm and LLA, the Tatara Bridge.



#### **6.2.4 Effect of Laminas Dominant Fiber Alignment:**

In order to realize the complicated relation between the increase of the Laminas Dominate Fiber Alignment (LDFA) and the bridge structural response, it is important to relate the changes in the deck deformations and stiffnesses to the changes in mechanical properties of its laminated components. The complicated bending behavior of the laminated shell due to the effects of the bending-twisting, shear-extension, bending-extension coupling, can explain the deformation results of the proposed bridge system. On the other hand, the change in the deck stiffnesses due to the change in laminate macro-structure explains the results of the natural frequencies of the system and the related mode shapes. The laminated part of the four proposed bridge deck models is mainly composed from two types of laminates, flat and cylindrical. The flat laminates are of two types, vertical and horizontal. Here, the inclined laminates can be imagined as a resultant of both the horizontal and the vertical laminate types. Consequently, the changes of any of the deck stiffnesses are resultants of the changes of the related laminates contributions. The concentration here is on the deck flexural stiffnesses, the vertical flexural stiffness and the lateral flexural stiffness, the torsional stiffness and the axial stiffness. However, the deck geometry and the deck length are considered constant for every case. The lamina material properties on its principal axes and the laminas thicknesses in the laminate are also constant. Then the deck stiffnesses are functions of the laminates stiffnesses, which are a function of their laminas orientations. On the other hand, the micro/macro design is such that the majority of the laminas (60% of the laminate thickness), of both the inner and the outer laminates, are oriented in the  $\pm$ LDFA directions. The other “minor” laminas have their own orientation, and absolute values of those orientations are greater than the absolute value of LDFA, as shown in Section 6.2.3. The inner laminates include the “low-to medium” angles of “minor” orientations (28% in the orientation of  $30^\circ$  and 12% in the orientation of  $45^\circ$ ). The outer laminates include the “medium to high” angles of “minor” orientations (28% in the orientation of  $45^\circ$  and 12% in the orientation of  $90^\circ$ ). Any laminate stiffness is “dominantly” lead by the stiffnesses of the laminas oriented in  $\pm$ LDFA directions but the effects of the “minor” laminas contributions are very important in some cases, where the changes in their moduli are significant.

The influence of increasing the angle of a lamina orientation on the stiffness of the laminate can be explained by a simplified situation of a laminate that has a single lamina stressed at an angle to the lamina principal direction. The “structural” stresses are assumed to be in the directions of the laminate axes,  $x$  and  $y$ , and in the  $x$ - $y$  plane. If the lamina principal axes are rotated by an angle ( $\theta$ ) from the laminate axes (Figure 6.2.4.1), it is necessary to find the changes in the laminate stiffnesses due to this rotation. If the laminate section and length are constant, then the only variables in the laminate stiffnesses are its moduli, which are functions of the angle of orientation,  $\theta$ . The problem is then reduced to finding the relation between the lamina moduli of elasticity in its principal plane (1-2) to those of the laminate in its “structural” plane ( $x$ - $y$ ), or the plane of laminate stresses.

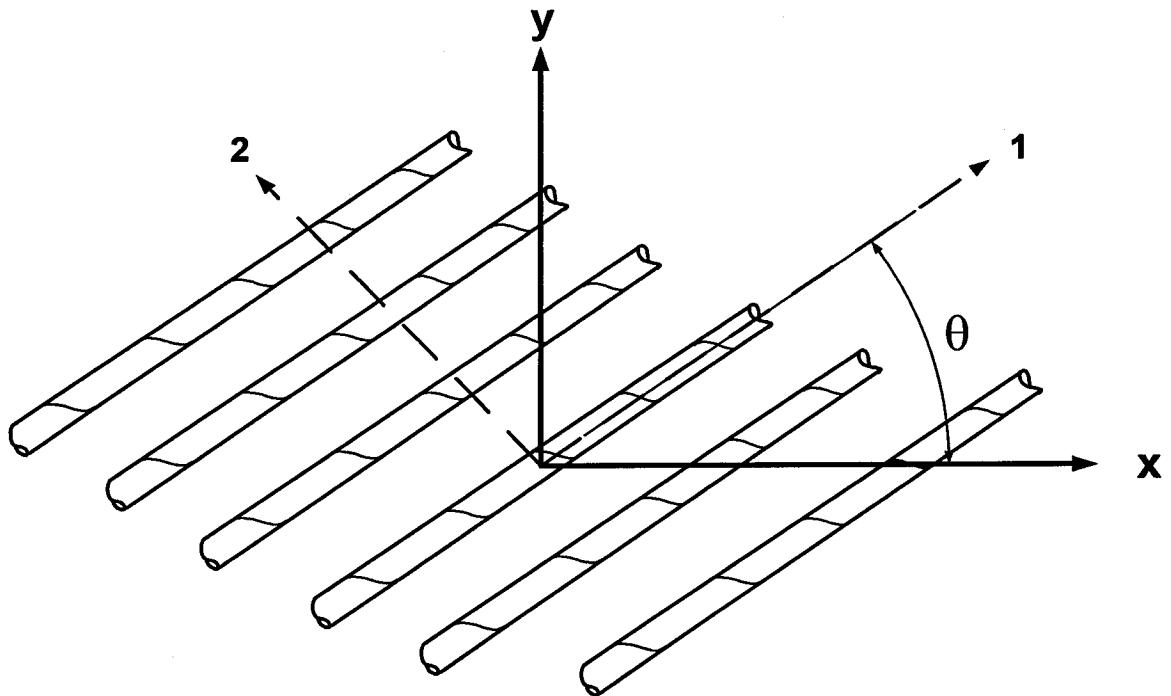


Figure 6.2.4.1 Positive rotation of the lamina principal material axes (1-2) from the laminate axes ( $x$ - $y$ ). The third lamina axis “3” is coinciding with the laminate “ $z$ ” axis.

The relation between the lamina and the laminate stresses and strains are:



$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} = [T]^{-1} \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \end{Bmatrix} \quad (6.1)$$

and

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} = [T]^{-1} \begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \gamma_{12} \end{Bmatrix} \quad (6.2)$$

Where [T] is the transformation matrix and given by,

$$[T] = \begin{bmatrix} \cos^2 \theta & \sin^2 \theta & 2 \sin \theta \cos \theta \\ \sin^2 \theta & \cos^2 \theta & -2 \sin \theta \cos \theta \\ -\sin \theta \cos \theta & \sin \theta \cos \theta & \cos^2 \theta - \sin^2 \theta \end{bmatrix} \quad (6.3)$$

The stain-stress relation of the orthotropic lamina used in this research is:

$$\begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \gamma_{12} \end{Bmatrix} = \begin{bmatrix} S_{11} & S_{12} & 0 \\ S_{12} & S_{22} & 0 \\ 0 & 0 & S_{66} \end{bmatrix} \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \end{Bmatrix} \quad (6.4)$$

Where

$$S_{11} = \frac{1}{E_1} \quad S_{12} = \frac{-\nu_{12}}{E_1} \quad S_{22} = \frac{1}{E_2} \quad S_{66} = \frac{1}{G_{12}} \quad (6.5)$$

And the stain-stress relation of the laminate of a single lamina, with  $\theta$  angle of orientation between the lamina principal axes and the laminate axes, is:

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} = \begin{bmatrix} \bar{S}_{11} & \bar{S}_{12} & \bar{S}_{16} \\ \bar{S}_{12} & \bar{S}_{22} & \bar{S}_{26} \\ \bar{S}_{16} & \bar{S}_{26} & \bar{S}_{66} \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} \quad (6.6)$$

Where

$$\begin{aligned} \bar{S}_{11} &= S_{11} \cos^4 \theta + (2S_{12} + S_{66}) \sin^2 \theta \cos^2 \theta + S_{22} \sin^4 \theta \\ \bar{S}_{12} &= S_{12} (\sin^4 \theta + \cos^4 \theta) + (S_{11} + S_{22} - S_{66}) \sin^2 \theta \cos^2 \theta \\ \bar{S}_{22} &= S_{11} \sin^4 \theta + (2S_{12} + S_{66}) \sin^2 \theta \cos^2 \theta + S_{22} \cos^4 \theta \\ \bar{S}_{16} &= (2S_{11} - 2S_{12} - S_{66}) \sin \theta \cos^3 \theta - (2S_{22} - 2S_{12} - S_{66}) \sin^3 \theta \cos \theta \\ \bar{S}_{26} &= (2S_{11} - 2S_{12} - S_{66}) \sin^3 \theta \cos \theta - (2S_{22} - 2S_{12} - S_{66}) \sin \theta \cos^3 \theta \\ \bar{S}_{66} &= 2(2S_{11} + 2S_{22} - 4S_{12} - S_{66}) \sin^2 \theta \cos^2 \theta + S_{66} (\sin^4 \theta + \cos^4 \theta) \end{aligned} \quad (6.7)$$

Note that the laminate compliance matrix  $\bar{S}_{ij}$  has terms in all its nine elements in contrast to the  $S_{ij}$  of the lamina, which has zero elements in the third line and column. However there are still only four independent material constants because the lamina is orthotropic. In the laminate coordinate system(x and y), there is coupling between shear strain and normal stresses and between shear stress and normal strains. In other words, there is shear extension coupling in the laminate. Thus, in its own coordinate, the laminate appears to be anisotropic, although its single lamina is orthotropic. Furthermore, it is found that this “apparent anisotropic laminate” has similar properties as the anisotropic material under the condition of plane stress. The strain-stress relation of such anisotropic material is:

$$\begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \gamma_{12} \end{Bmatrix} = \begin{bmatrix} \tilde{S}_{11} & \tilde{S}_{12} & \tilde{S}_{16} \\ \tilde{S}_{12} & \tilde{S}_{22} & \tilde{S}_{26} \\ \tilde{S}_{16} & \tilde{S}_{26} & \tilde{S}_{66} \end{bmatrix} \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \end{Bmatrix} \quad (6.8)$$

Where

$$\begin{aligned} \tilde{S}_{11} &= \frac{1}{E_1} & \tilde{S}_{22} &= \frac{1}{E_2} & \tilde{S}_{16} &= \frac{\eta_{12,1}}{E_1} \\ \tilde{S}_{12} &= -\frac{\nu_{12}}{E_1} & \tilde{S}_{66} &= \frac{1}{G_{12}} & \tilde{S}_{26} &= \frac{\eta_{12,2}}{E_1} \end{aligned} \quad (6.9)$$

And

$\eta_{ij,i}$ : Shear-extension coupling coefficient, characterizing shearing in ij-plane by normal stress in i-direction.

Comparing the apparent anisotropic compliances in Equation (6.7) and the anisotropic compliances in Equation (6.9), the following expressions are found for the elastic moduli and the shear-extension coupling coefficient of the laminate:

$$\begin{aligned}
E_x &= \left[ \frac{1}{E_1} \cos^4 \theta + \left\{ \frac{1}{G_{12}} - \frac{2\nu_{12}}{E_1} \right\} \sin^2 \theta \cos^2 \theta + \frac{1}{E_2} \sin^4 \theta \right]^{-1} \\
E_y &= \left[ \frac{1}{E_1} \sin^4 \theta + \left\{ \frac{1}{G_{12}} - \frac{2\nu_{12}}{E_1} \right\} \sin^2 \theta \cos^2 \theta + \frac{1}{E_2} \cos^4 \theta \right]^{-1} \\
G_{xy} &= 2 \left[ \frac{2}{E_1} + \frac{2}{E_2} + \frac{4\nu_{12}}{E_1} - \frac{1}{G_{12}} \right] \sin^2 \theta \cos^2 \theta + \frac{1}{G_{12}} (\sin^4 \theta + \cos^4 \theta) \\
\eta_{xy,x} &= E_x \left[ \left\{ \frac{2}{E_1} + \frac{2\nu_{12}}{E_1} - \frac{1}{G_{12}} \right\} \sin \theta \cos^3 \theta - \left\{ \frac{2}{E_2} + \frac{2\nu_{12}}{E_1} - \frac{1}{G_{12}} \right\} \sin^3 \theta \cos \theta \right]
\end{aligned} \tag{6.10}$$

The importance of the presence of the shear-extension coupling coefficient is that an axial stress in a non-principal material direction for a lamina results in shear deformation in addition to the usual axial extension. Equation (6.10) gives the apparent anisotropic moduli for a laminate of a single lamina (oriented by  $\theta$  from the laminate axes) and stressed in the laminate directions. Figure 6.2.4.2 views the variation of the laminate (of a single lamina) moduli in the range of the laminas orientations involved in the micro/macro design of the deck laminates, or  $-90^\circ \leq \theta \leq 90^\circ$ . The results of  $E_x$ ,  $E_y$ , and  $G_{xy}$  are symmetrical, and  $\eta_{xy,x}$  is skew symmetrical on  $\theta = 0$ . Hence, the reference will be only to the positive orientation for the moduli, but for the shear-extension coupling coefficient, both signs have to be mentioned. However, the beauty of using “skew-symmetric” orientation arrangement of laminas in the micro/macro design of the laminate in this research is that the shear-extension coupling effects due to the positive and negative orientations are cancel each other.

The reduction of the longitudinal modulus  $E_x$  is apparent for the orientation  $10^\circ \leq \theta \leq 30^\circ$ , which is in the range of LDFA variation of the present investigation. While in the same range of LDFA variation, the lateral Modulus  $E_y$  decreases slightly, and the in-plane shear modulus  $G_{xy}$  increases. For the “minor” laminas of the inner laminate, in which the orientation angles vary in the range  $-30^\circ \leq \theta \leq 65^\circ$ ,  $E_x$  decreases then increases and  $E_y$  increases slightly but  $G_{xy}$  increases then decreases very highly. For the “minor” laminas of the outer laminates, in which its orientation angles vary between  $45^\circ \leq \theta \leq 90^\circ$ ,  $E_x$  decreases slightly up to an orientation angle of  $50^\circ$  then increases. On the other hand,  $E_y$  increases very highly except for the laminas with orientation angles of  $90^\circ$ ,

where it decreases very highly as with the angle increase this orientation angle moves to  $-80^\circ$  and  $-70^\circ$ , while  $G_{xy}$  is decreased.

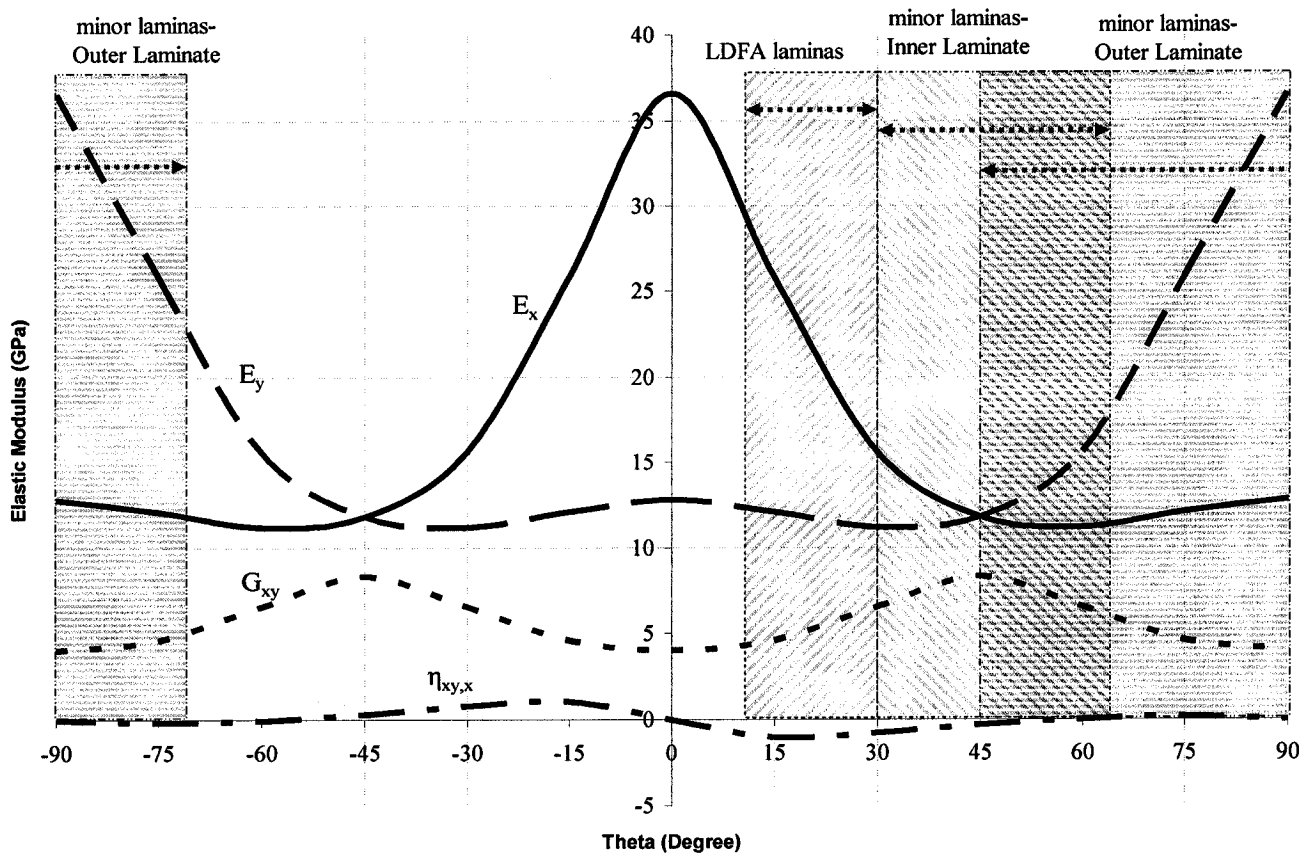


Figure 6.2.4.2 Elastic moduli and the shear-extension coupling coefficient of a laminate of a single E-Glass fiber and Vinylester Matrix lamina as functions of the lamina principal axes orientation angle ( $\theta$ ) to the laminate axes

As mentioned earlier, the deck section and length are constants for every bridge and every deck model. The contributions of each laminate to the deck stiffnesses are then dependent on the laminate variable moduli and the laminate constant moments of inertia. The moments of inertia, as constants, either enhance the moduli variation effect or reduce it. For both flexural stiffnesses, vertical and the horizontal,  $E_x$  is the modulus that is in the major bending axis while  $E_y$  is in the minor axis. The length of any laminate (length of the bridge) is far more than its lateral dimension, hence to simplify the analysis of the results, the interaction between the two moduli effects on the flexural stiffnesses will be

ignored. In the light of different laminates characteristics, moduli variations, and their moments of inertia, it has been found that LDFA increase (also the increase of the “minor laminas” orientation in a similar pattern) affect different laminates contributions as follows:

- i) For the horizontal flat laminates, LDFA increase leads to: (a) Decrease of the vertical flexural stiffness of the deck as the laminate modulus in the longitudinal direction  $E_x$  is decreased for the inner and the outer laminates. The changes of  $E_x$  is guided by the “major” laminas in LDFA orientation of both laminate types (inner and outer). However, the changes are reduced by the effect of increasing  $E_x$  of the “minor” laminas. (b) Decrease of the lateral flexural stiffness of the deck. The laminate modulus in the flexure direction  $E_x$  is decreased as before. (c) The outer laminates slightly decrease the deck torsional stiffness and the inner laminates are give fluctuating contributions to the deck torsional stiffness. The resultant in-plane laminate shear modulus  $G_{xy}$  either slightly decreases or increases then decreases.  $G_{xy}$  increases slightly for the “major” laminas, but it increases then decreases very highly for the “minor” laminas of the inner laminates and decreases highly with outer laminates. The slight decrease of the deck torsional stiffness could lead to a fluctuation of its values depending on the ratio of the inner to the outer laminates of different models. It is also important to note that the outer horizontal laminates have relatively higher moments of inertia on the horizontal centerline of the deck, as their distances from this centerline are greater.
- ii) The vertical flat laminates are only formed from the “inner” laminate types. The longitudinal and the lateral directions of the lamina have been interchanged in comparison with those of the horizontal laminates. LDFA increase leads to: (a) Decrease of the vertical flexural stiffness of the deck. The laminate modulus  $E_x$  is decreased. Indeed,  $E_y$  decreases for both the “major” laminas and the “minor” laminas. (b) Decrease of the lateral flexural stiffness of the deck as the laminate modulus in the longitudinal direction  $E_x$  is decreased. (c) Similar to the horizontal inner laminate, it gives a fluctuating contribution to the torsional stiffness for the same reason explained earlier.
- iii) For a cylindrical laminate, the helical path of a fiber, with a higher angle of rotation, needs less number of rounds (and of course less path lengths) to cover the

same height of the cylinder (Figure 6.2.4.3). This would change the stiffnesses of the cylindrical laminate with a higher rate than a similar flat laminate. The coordinate reference for the laminate and the lamina are changed to the orthogonal- tangential-set for 1-2, and the longitudinal and tangential for x and y, while z or 3 are normal to the cylinder surface (or in the radial direction). The changes of the laminate moduli in the directions of the cylinder are combinations of the  $E_x$  and  $E_y$  according to the new axis transformation. LDFA increase leads to: (a) Increase of the vertical flexural stiffness as the contribution of  $E_y$  is reversing the reduction coming from  $E_x$ . (b) Similarly, it increases the lateral flexural stiffness. (c) Similar to the other inner laminates; it gives a fluctuating contribution to the torsional stiffness.  $G_{xy}$  increases slightly for the “major” laminas, but it increases then decreases very highly for the “minor” laminas of the inner laminates and decreases highly with outer laminates. However, most of the cylindrical laminates of Model III are of the “inner” types and hence a fluctuation or reverse behavior of the torsional stiffness is expected for such cases. On the other hand, Model IV side laminated tubes are formed from inner and outer cylindrical laminates. The outer laminates have slightly more radius and hence more area and moment of inertia.

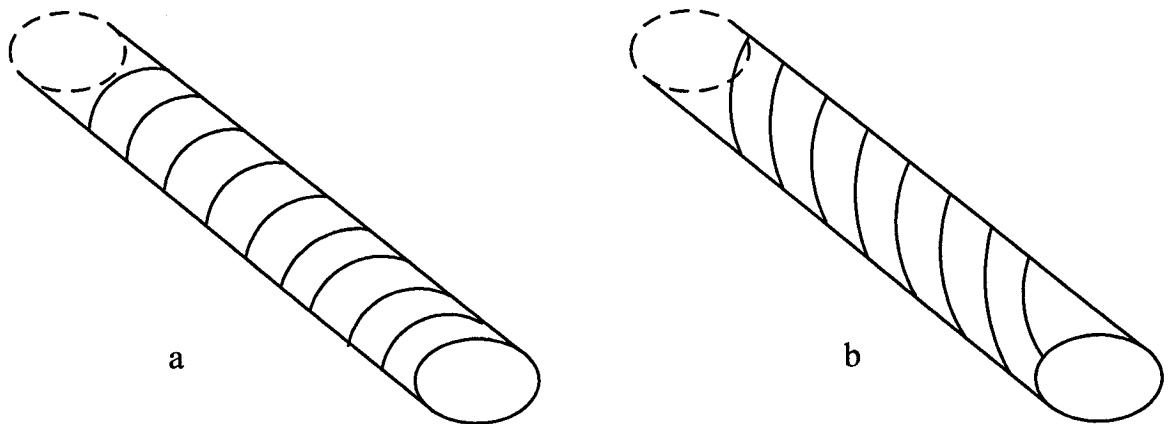


Figure 6.2.4.3 Increasing the lamina orientation angle ( $\theta$ ) on the cylindrical laminate.

#### **6.2.4.1 Effect of Laminas Dominant Fiber Alignment on the Deck Top Surface Maximum Displacements:**

It has been found that the change of the lateral displacement  $U$  is very small, due to the increase of the Laminas Dominant Fiber Alignment (LDFA) (Figures 6.2.4.1.1 to 6.2.4.1.3). This change can be related to the bending-twisting coupling, which gives the out of bending-plane deformation of the laminated structure, as the laminas orientations are not parallel to the bending-plane. Models I and II show a very slight decrease in their maximum lateral displacement for Annacis Island Bridge and the Second Chang Bridge, and they show a slight increase with the Tatara Bridge. The two models laminated box girders are formed from only flat laminates. Model III shows an increase of its maximum  $U$  displacement for all bridge central spans. This model is formed from horizontal and semi-cylindrical laminates. Model IV, shows almost no change for the shortest and medium bridge central span, and a very slight increase with the longest examined span. The small laminated part of this model explains the slight effect of LDFA increase on the lateral displacement.

On the other hand, the vertical displacements of all models are slightly and linearly decreased as the LDFA angle is increased (Figures 6.2.4.1.4 to 6.2.4.1.5). However this reduction becomes nonlinear as the bridge central span becomes very long. The moment of inertia of the horizontal laminates is more than the moment of inertia of the vertical and the cylindrical laminates. As LDFA increases, horizontal laminates cause a reduction of the vertical flexural stiffness as their  $E_x$  decreases and also the vertical laminates cause an increase, while the cylindrical laminates cause an increase in the stiffness. The resultant of these effects depends on the model section, but it cause a slight reduction in the vertical flexural stiffness in all the cases. This reduction makes the deck deflection more dependent on the cable stay system and results in more loading of the short cables (closer to the pylons). This is reflected in a reduction of the cable tension in the long cables and an increase of the cable tension in the short cables. Accordingly, the distribution of the deck deflection of a more flexible deck is locally affected and its peak levels are slightly reduced.

The maximum  $W$  displacements (Figures 6.2.4.1.6 to 6.2.4.1.8) on the top surface of the deck of all models show a slight drop as LDFA increases. The slight reduction in the deck

vertical flexural stiffness slightly reduces the stretching effect due to the high load difference between the central and the side spans. The bridge system is transferring the central span load more through the stay cable system and less through the deck, and this explains the slight reduction in the maximum longitudinal displacements.

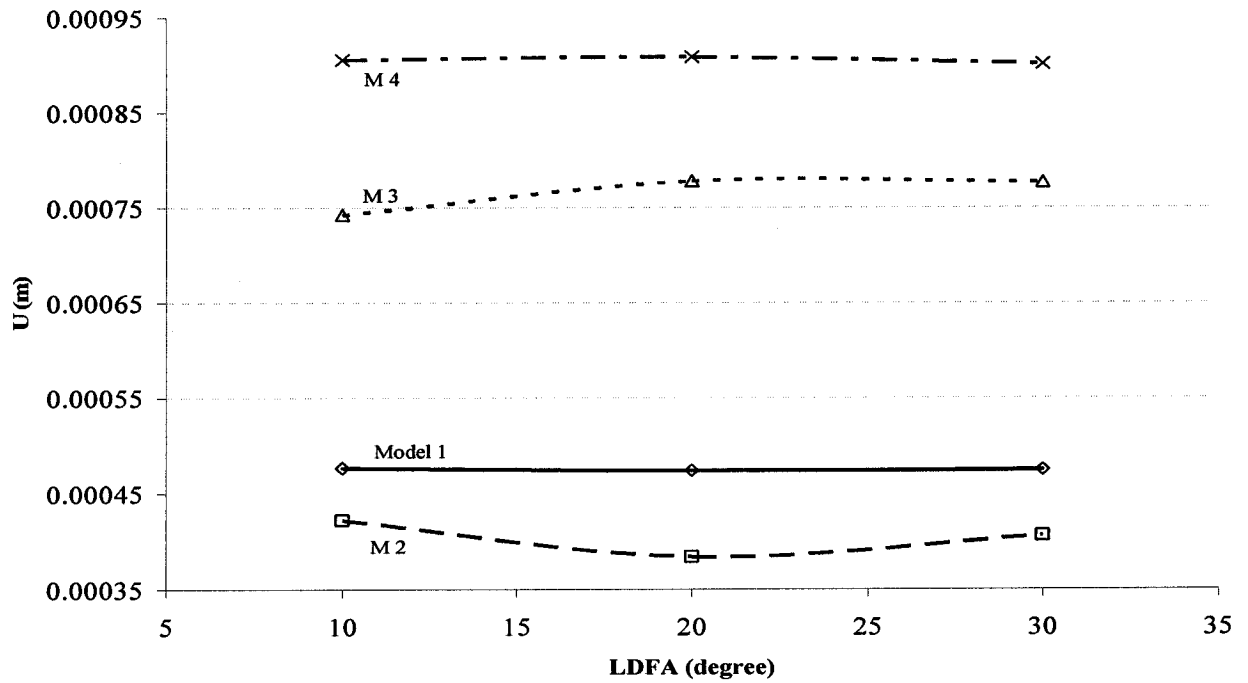


Figure 6.2.4.1.1 Laminas Dominant Fiber Alignment Effect on Lateral Displacement (U)  
 – Thb- CCR 0.100m - Annacis Island Bridge



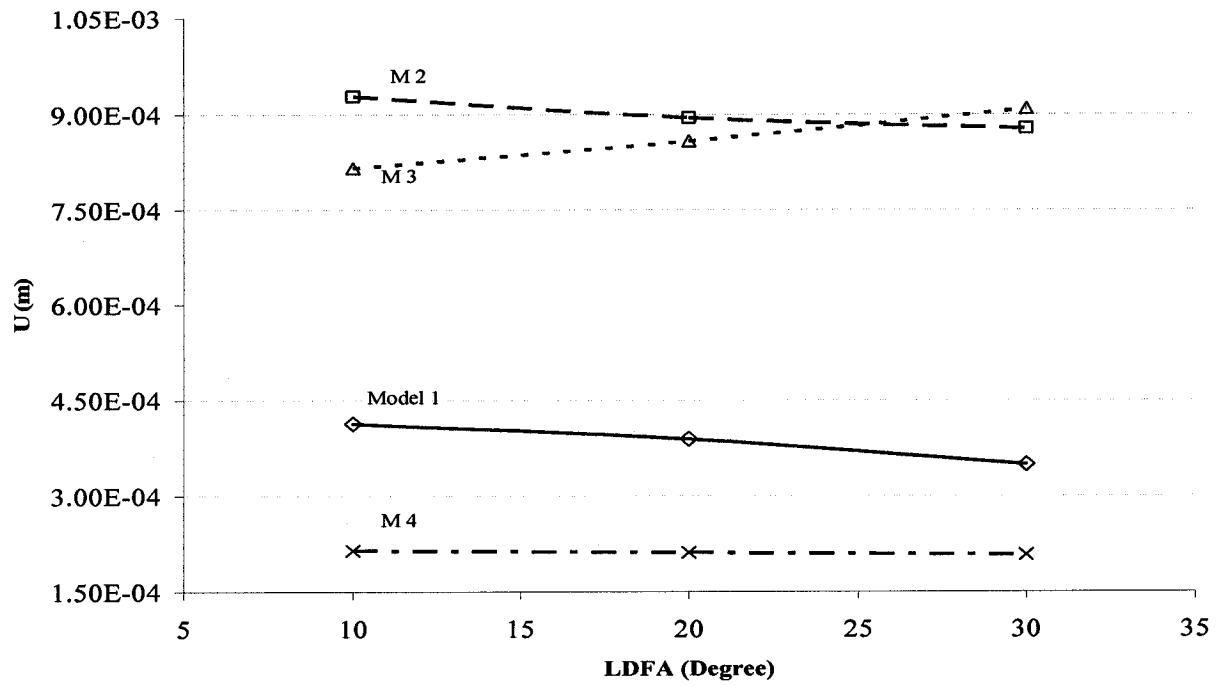


Figure 6.2.4.1.2 Laminas Dominant Fiber Alignment Effect on Lateral Displacement (U)  
 – Thb- LLA- CCR 0.125 – Second Chang River Bridge

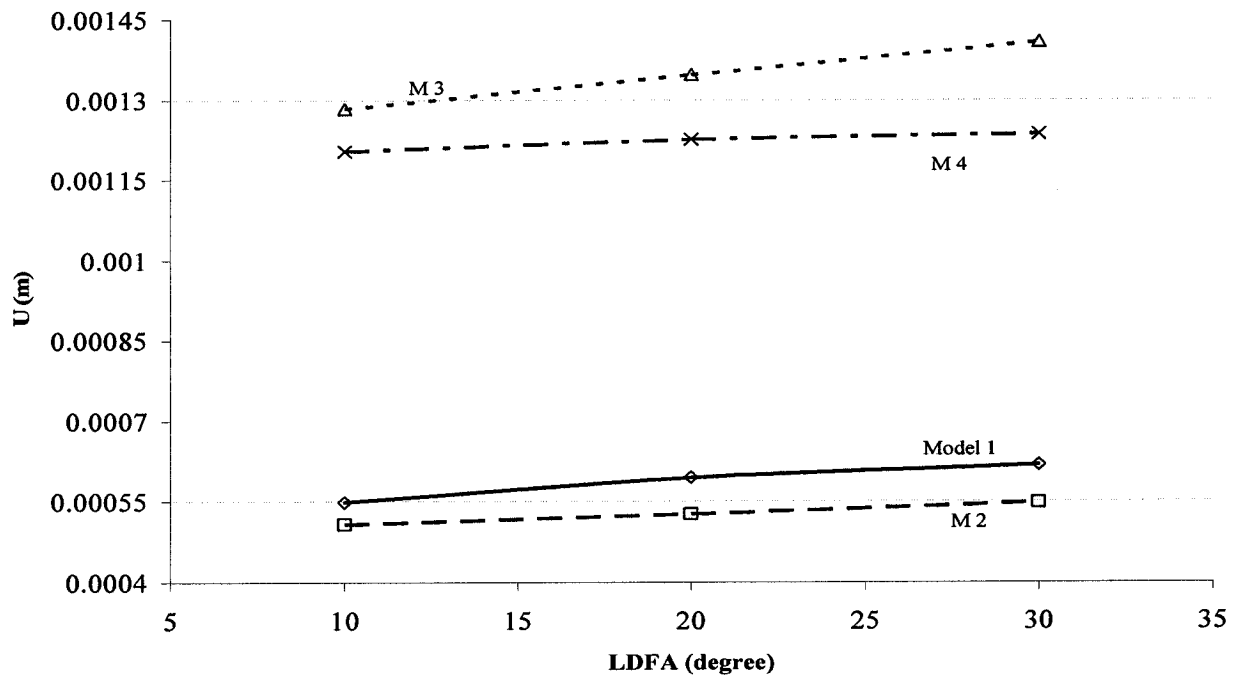


Figure 6.2.4.1.3 Laminas Dominant Fiber Alignment Effect on Lateral Displacement (U)  
 – Thb- LLA- CCR 0.150 – Tatara Bridge

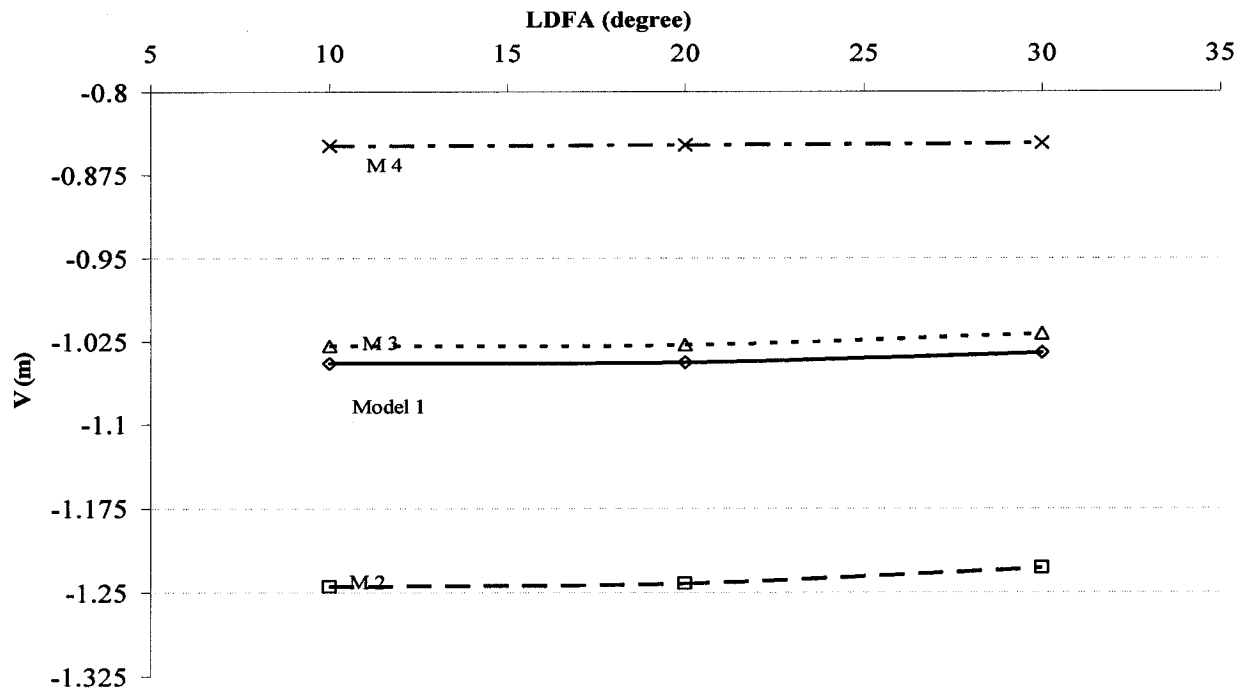


Figure 6.2.4.1.4 Laminas Dominant Fiber Alignment Effect on Lateral Displacement (V)  
– Thb- CCR 0.100m - Annacis Island Bridge

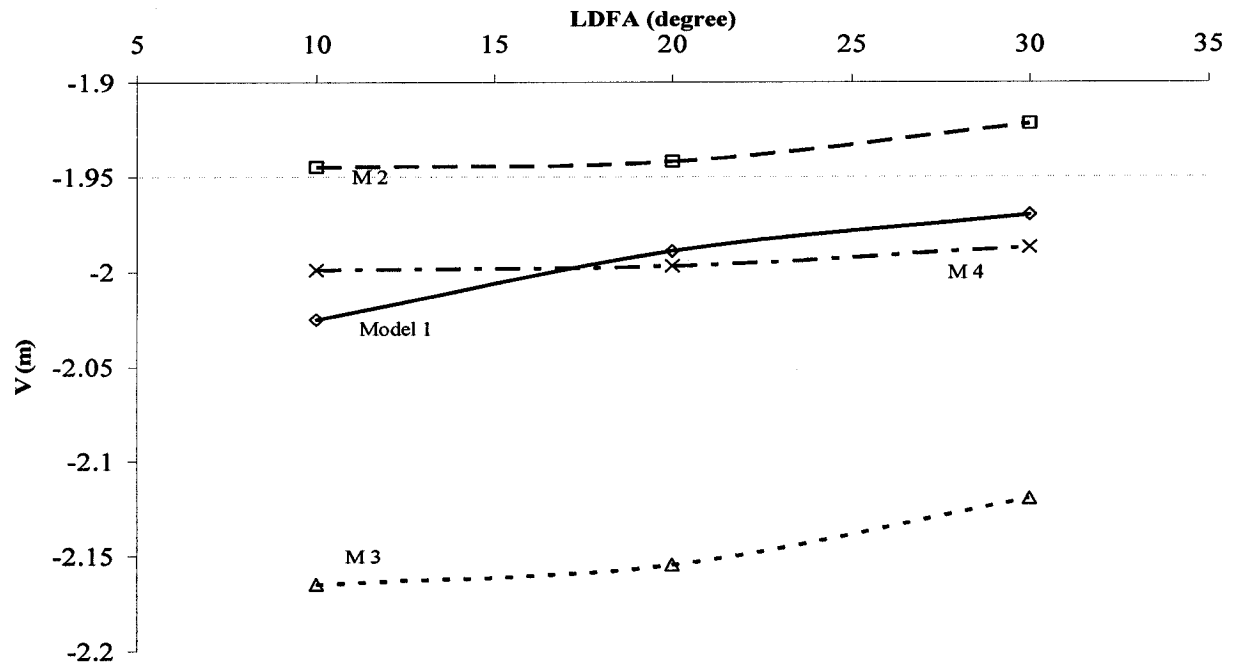


Figure 6.2.4.1.5 Laminas Dominant Fiber Alignment Effect on Vertical Displacement (V)  
– Thb- LLA- CCR 0.150m – Tatara Bridge

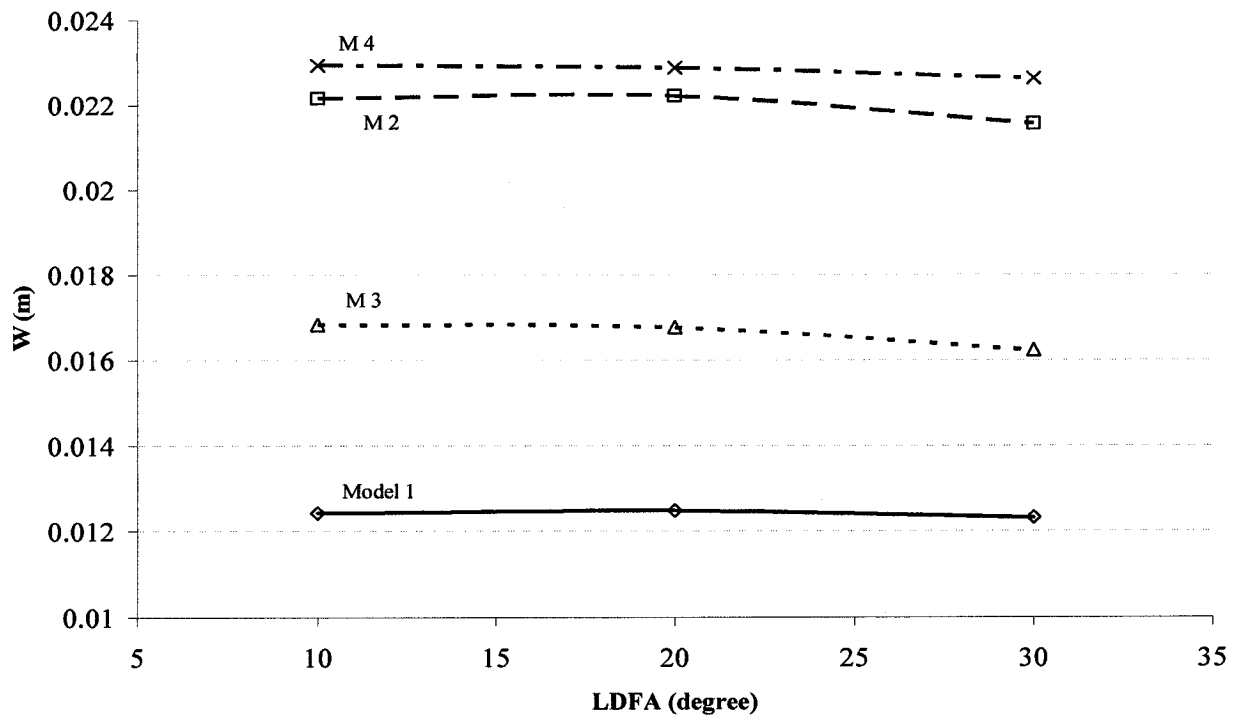


Figure 6.2.4.1.6 Laminas Dominant Fiber Alignment Effect on Lateral Displacement (W)  
– Thb- CCR 0.100m - Annacis Island Bridge

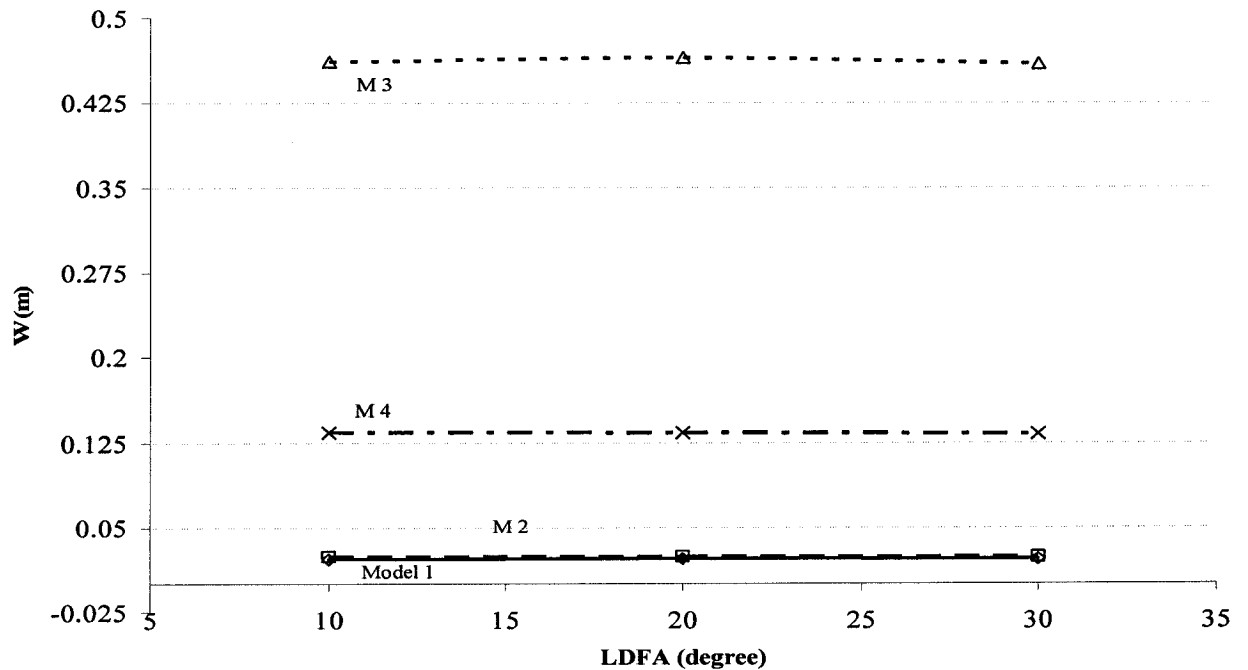


Figure 6.2.4.1.7 Laminas Dominant Fiber Alignment Effect on Longitudinal Displacement (W) – Thb- LLA- CCR 0.125m – Second Chang River Bridge

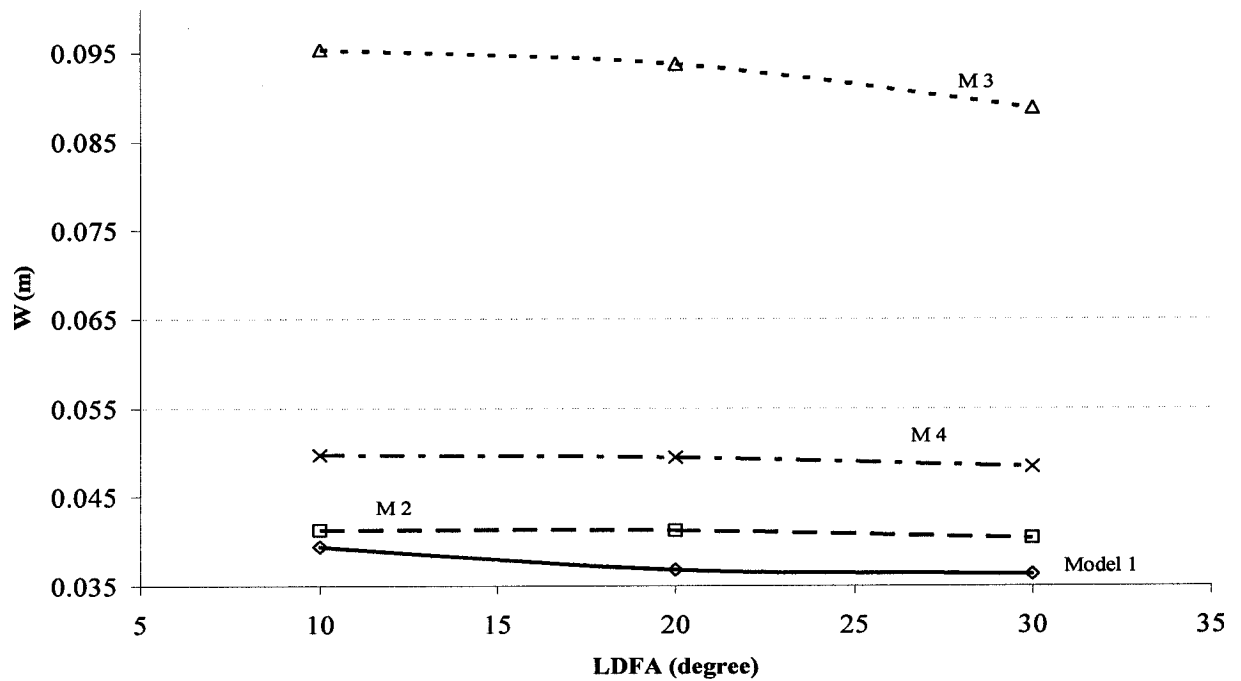


Figure 6.2.4.1.8 Laminas Dominant Fiber Alignment Effect on Longitudinal Displacement (W) – Thb- LLA- CCR 0.150m – Tatara Bridge

#### 6.2.4.2 Effect of Laminas Dominant Fiber Alignment on the Maximum Tsai-Hill Failure Function of Different Laminates, the Principal Stresses of the Chopped FRP and the Tensile Stresses of the Cable Stays:

The effect of Laminas Dominant Fiber Alignment (LDFA) on the maximum Tsai Hill Failure Function (THFF) can be summarized as follows: (i) the top laminate of the laminated box girder of Models I, II, and III and the stiffening laminate of Model IV, or Laminate B (Figures 6.2.4.2.1 to 6.2.4.2.3) has the lowest THFF that is slightly decreased with the increase of LDFA for all models, (ii) The bottom laminate of the laminated box girder of Models I, II, and III and the left cylindrical laminate of Model IV, or Laminates C (Figures 6.2.4.2.4 to 6.2.4.2.6) has the maximum THFF among the other laminates (except for Model II with  $LDFA \geq 12^\circ$ ), and it also shows a general slight reduction of its level with the increase of LDFA. Model II gives a fluctuated THFF with the medium bridge central span (or the Second Chang River Bridge), and Model III shows an increase of its THFF level for the range ( $10^\circ \leq LDFA \leq 20^\circ$ ), (iii) The vertical laminates of the

laminated box girder of Models I, II, and III and the right cylindrical laminate of Model IV, or Laminates D (Figures 6.2.4.2.7 to 6.2.4.2.9) show a slight decrease or slight increase, except with Model II for the Second Chang River Bridge, where the increase is very rapid and very high.

The changes in the moduli of different laminates, due to the increase of LDFA and the resulting changes in the deck stiffness, leads to changes in the distributions of different stresses. Hence, the Tsai-Hill Failure Function distributions over every laminate in the deck are also changed. The static stresses in the deck are mainly due to the flexure, torsion and the concentrated loads. The most critical loading case which generates the worst THFF distributions is the Live Load A, where all the loads are gravity loads and the bridge central span is loaded with the full traffic loads. The vertical flexure (in the global YZ plane) due to the gravity loads is governing the stresses. The lateral flexure, the torsion and the stress concentration have minor effects on THFF distribution. The stresses due to the vertical flexure are axial stresses normal to the deck cross section. As LDFA increase causes a reduction of the moduli of the top and lower laminates, and since the strains are not changed very much (as the deformations changes are very low), then the axial stresses are lead by this laminate moduli reduction. This explains the general reduction of THFF over the top and the bottom laminates. On the other hand, the vertical and the cylindrical laminates show a decrease in their moduli in the longitudinal direction as LDFA increases. However, the vertical flexural strains are distributed from maximum extension to maximum contraction in the cross section. The strain distribution is also governed by the compatibility of strains in the section. As a result, the effect of the decreasing moduli on the maximum THFF of the vertical and the cylindrical laminates is either reduced or enhanced according to the strain distribution at each vertical laminate.

The high sensitivity of Model II is due to the sharp edges of the laminated box girder of this model (Figure 3.3). The very high THFF regions are observed in the vertical laminates of the minimum height at both deck section sides, and only close to the bridge deck ends in the longitudinal direction (Figure 6.2.3.9-b). The high THFF covers a very small region of these laminates, while elsewhere its values are low. The axial tensile stress field is switched to compression, and this explains the high THFF at those small regions on the edge of the vertical laminates. This sudden and huge change of THFF in the short vertical laminates, which is happening specifically with the Second Chang River

Bridge, can be explained by the large width of this bridge deck section and its sharp edges. Indeed, the boundary deck depth is the same for all the three bridges but the total deck width of the Second Chang River Bridge is 37.2 m, while it is 32 m for the Annacis Island Bridge and 30.8m for the Tatara Bridge. The increase in the deck section width would reduce the side edge vertical laminate height and further decrease the sharp angles of the deck section sides. Increasing LDFA results in a decrease in the longitudinal modulus of the vertical laminates. However, once the strains are increased greatly in those short edge laminates, due to the reduction of the deck stiffness, the resulting stress will be increased rapidly. Indeed, the short vertical edge laminates are above the neutral plane of the deck. If the stress direction gives compression then THFC increases very highly.

The vertical laminates of the other models show less sensitivity to the LDFA change because they either have larger heights (Model I, and the intermediate vertical laminates of Model II) or they are semicircular (Models III and IV) with large height. The deck neutral plane intersects with such laminates and the strains then have a full range of the distribution, while with the short vertical laminates, the strains are either an extension or a contraction.

The chopped FRP “deck slab” of Models I, II, and III and the most of the deck section of Model IV (Figures 6.2.4.2..10 and 6.2.4.2.11) gives almost no change of their maximum and minimum principal stresses due to the increase of LDFA angle. The stresses of this part of the deck section (which is formed from linear elastic material) are linear functions of its strains. The strains and hence the stresses are similar to the deformations; they are changed slightly at the top surface of the deck as LDFA increases. On the other hand, the maximum cable tension (Figures 6.2.4.2.12 and 6.2.4.2.13) is slightly decreased by the LDFA increase as the decrease of the deck vertical flexural stiffness causes a redistribution of the stay cable tensile forces. The cable tension is increased in the short cables close to the pylons and decreases in the long cables far from the pylons where the maximum cable tension is.

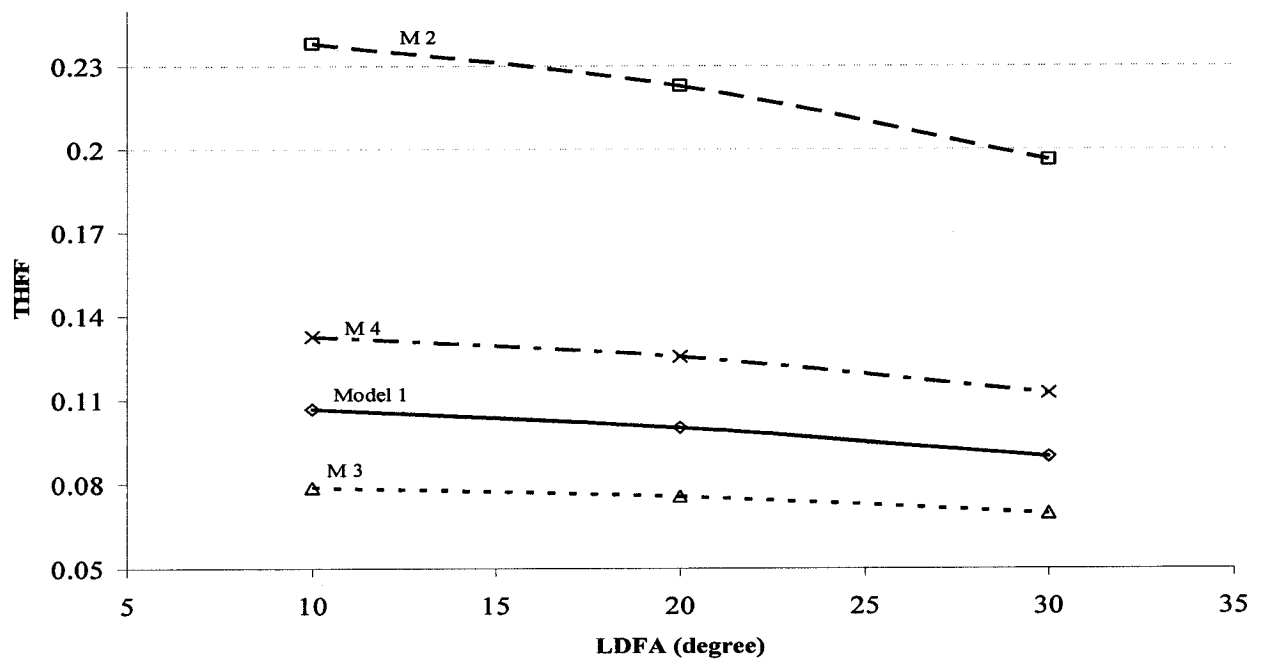


Figure 6.2.4.2.1 - Laminas Dominant Fiber Alignment Effect on THFF –Laminate B - Thb – CCR 0.100m -Annacis Island

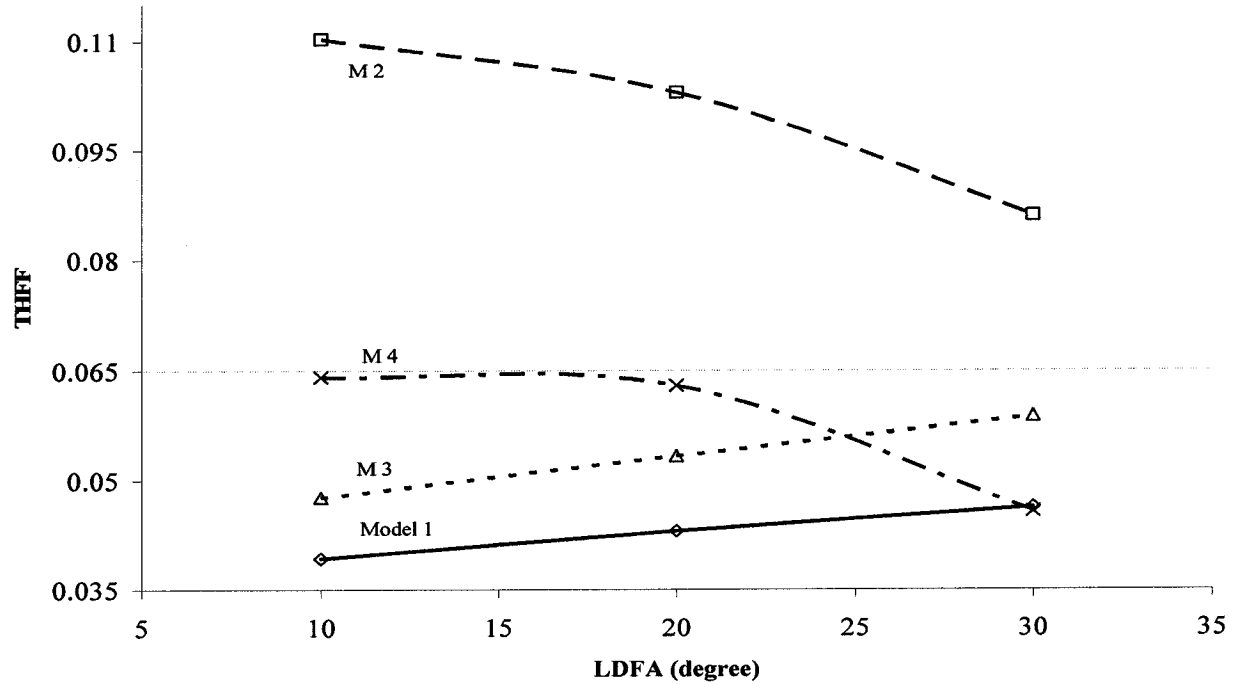


Figure 6.2.4.2.2 Laminas Dominant Fiber Alignment Effect on THFF – Laminate B - Thb - LLA- CCR 0.125m - Second Chang River Bridge

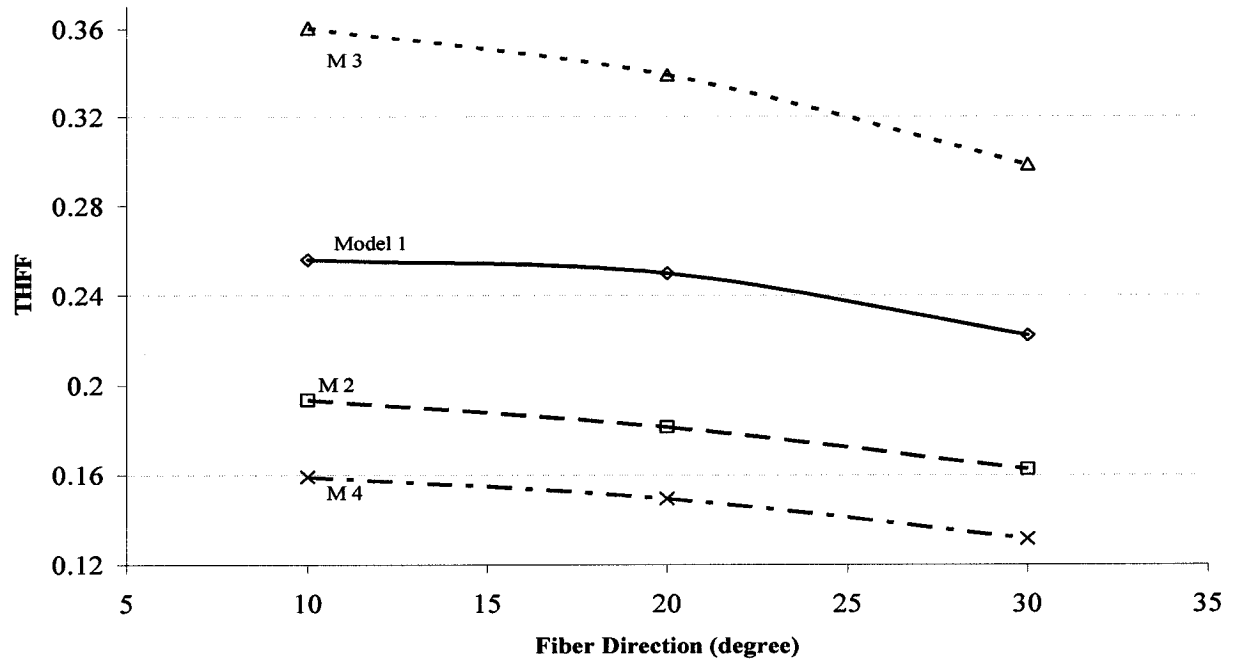


Figure 6.2.4.2.3 Laminas Dominant Fiber Alignment Effect on THFF – Laminate B-Thb- LLA- CCR 0.150m - Tatara Bridge

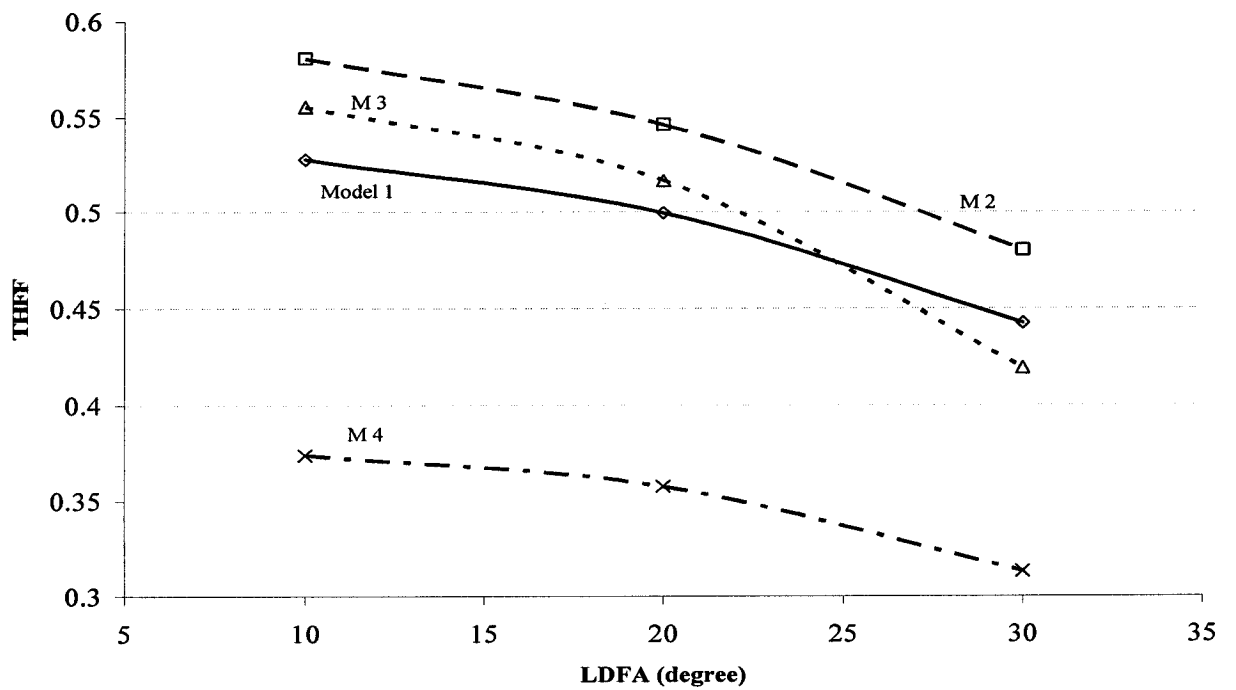


Figure 6.2.4.2.4 Laminas Dominant Fiber Alignment Effect on THFF – Laminate C - Thb – CCR 0.100m -Annacis Island Bridge



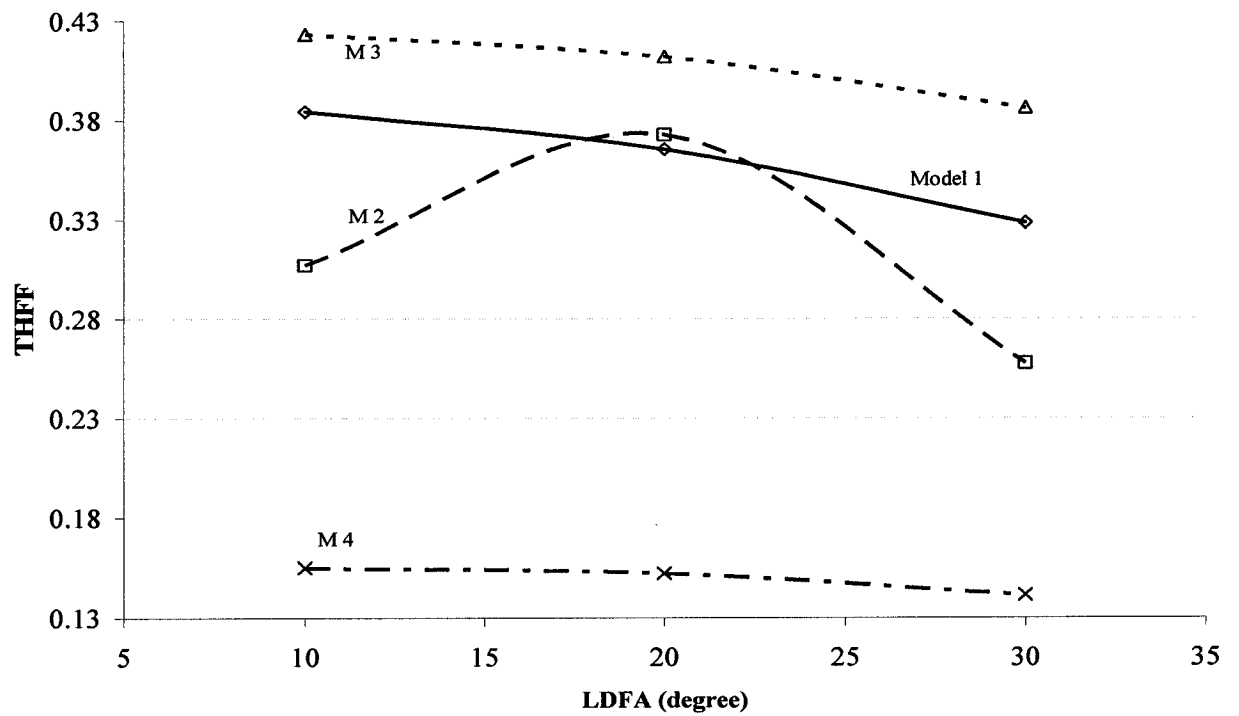


Figure 6.2.4.2.5 Laminas Dominant Fiber Alignment Effect on THFF – Laminate C - Thb - LLA- CCR 0.125m - Second Chang River Bridge

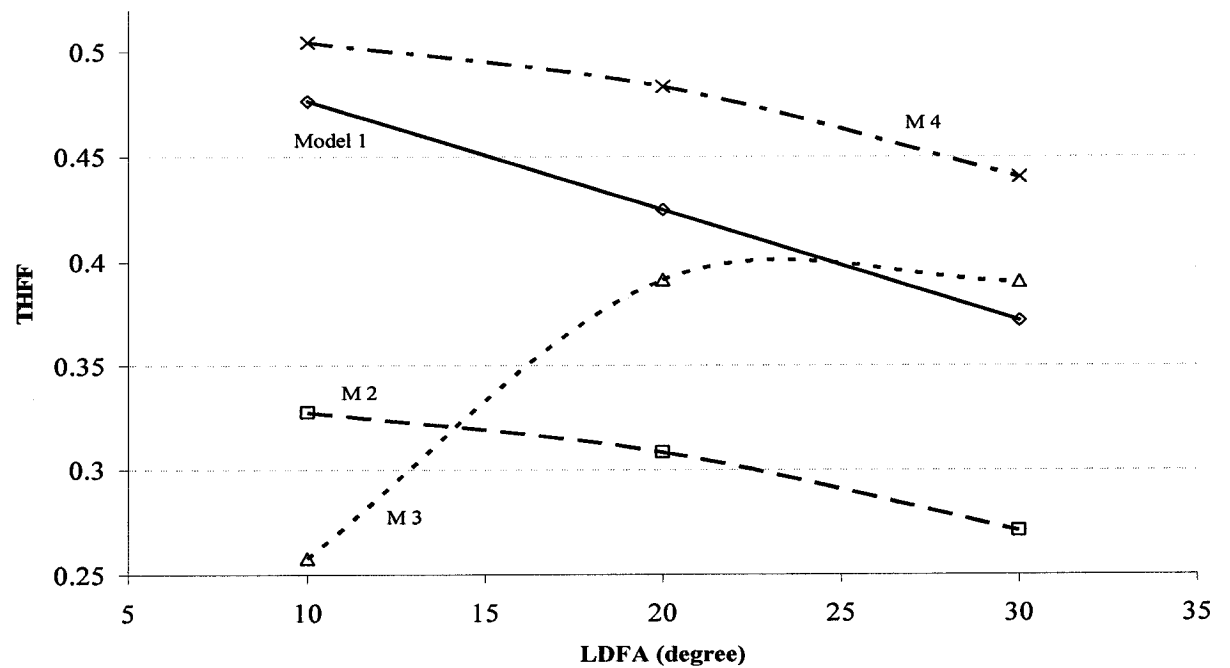


Figure 6.2.4.2.6 Laminas Dominant Fiber Alignment Effect on THFF – Laminate C- Thb- LLA- CCR 0.150m - Tatara Bridge

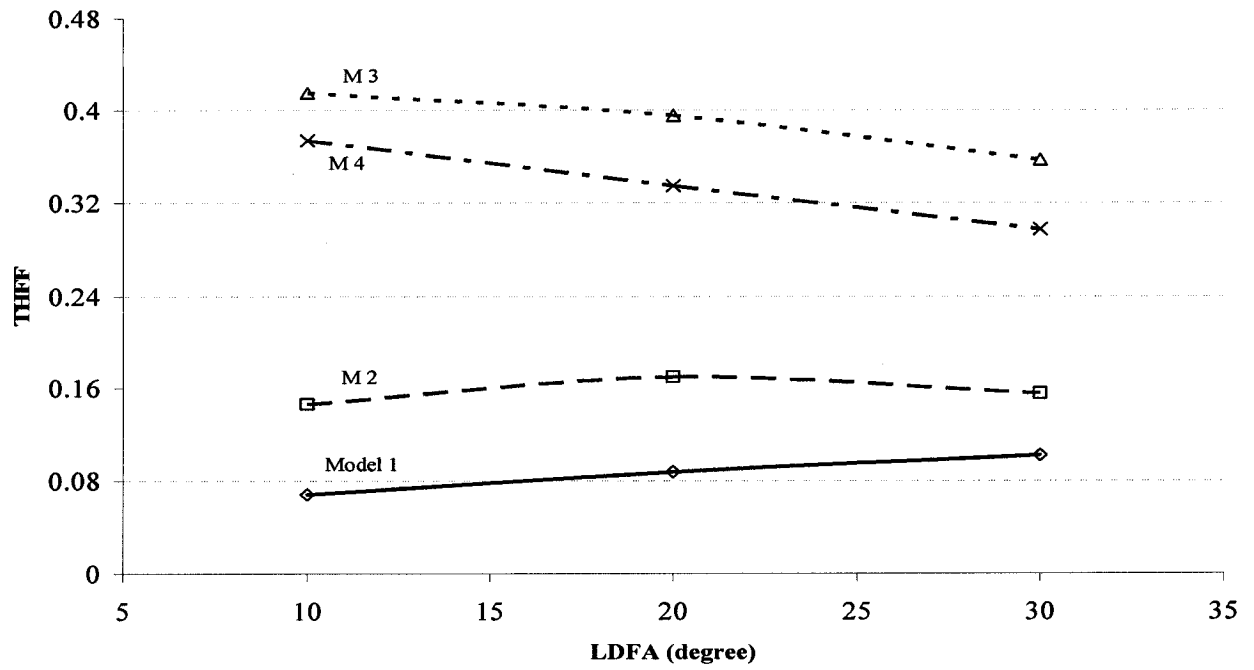


Figure 6.2.4.2.7 Laminas Dominant Fiber Alignment Effect on THFF – Laminate D - Thb – CCR 0.100m -Annacis Island Bridge

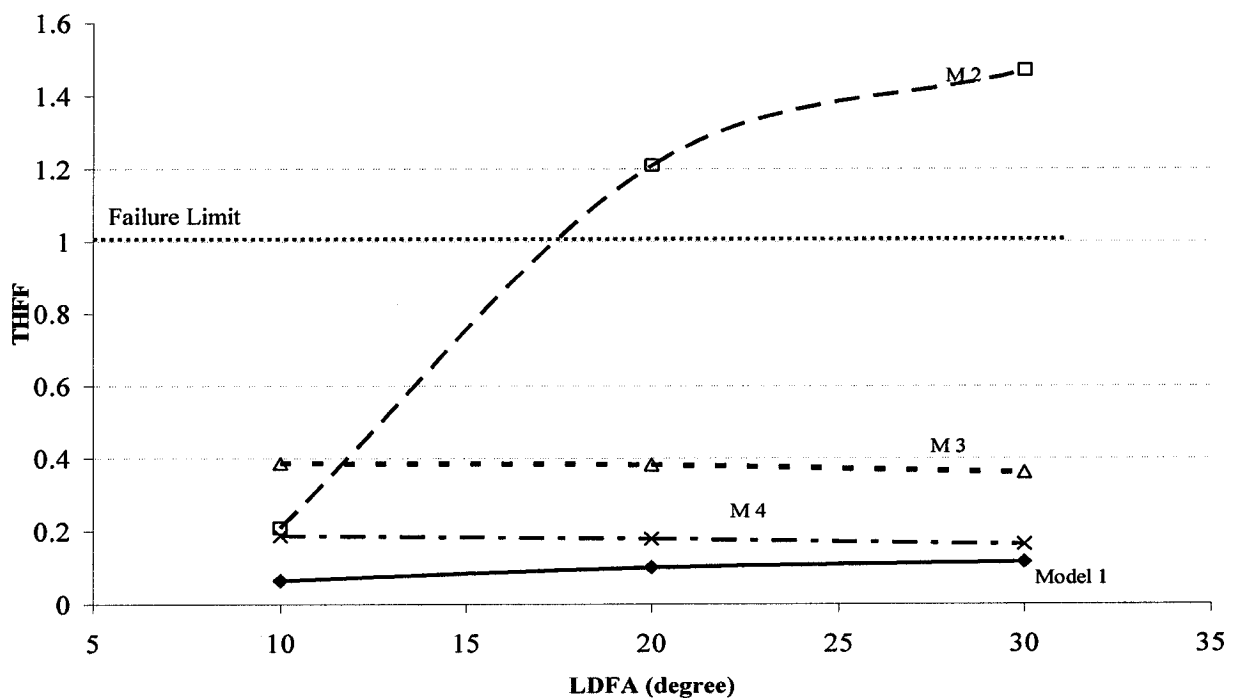


Figure 6.2.4.2.8 Laminas Dominant Fiber Alignment Effect on THFF – Laminate D - Thb - LLA- CCR 0.125m - Second Chang River Bridge

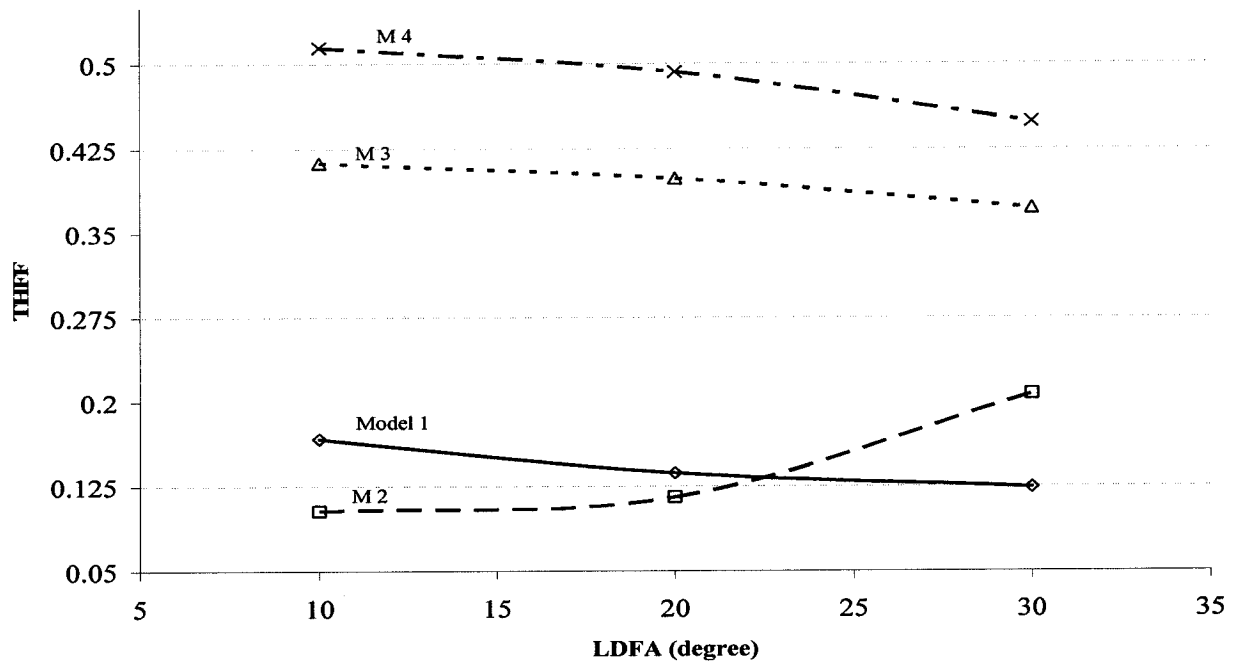


Figure 6.2.4.2.9 Laminas Dominant Fiber Alignment Effect on THFF – Laminate D-Thb- LLA- CCR 0.150 - Tatara Bridge

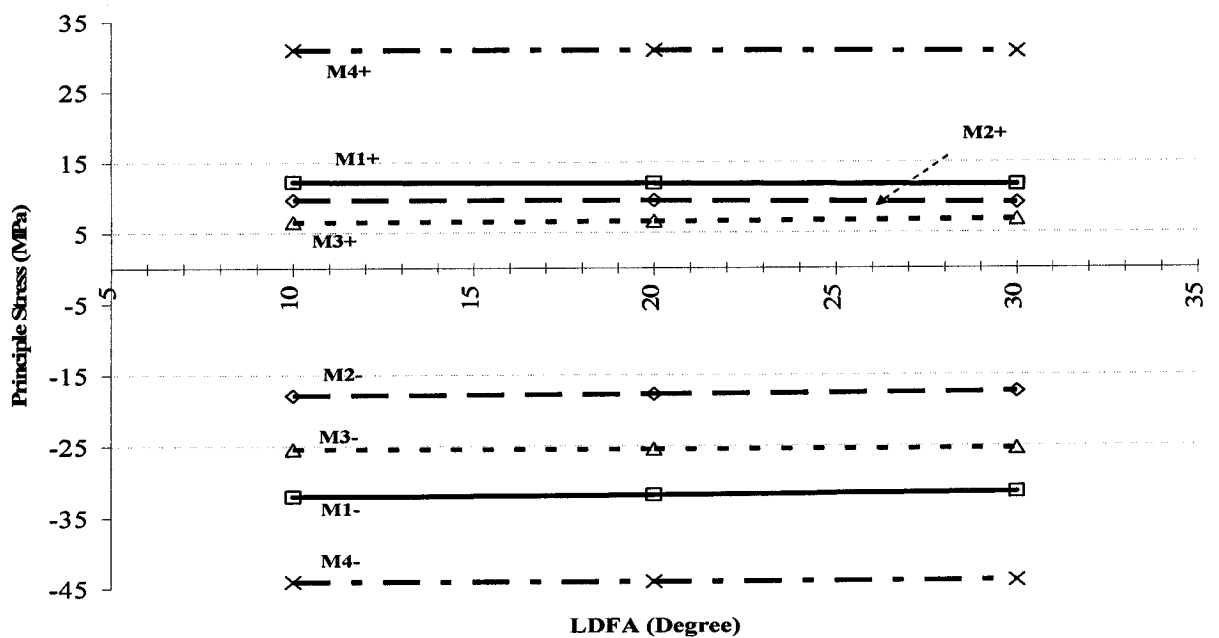


Figure 6.2.4.2.10 Laminas Dominant Fiber Alignment Effect on Principal Stresses at the Chopped FRP top layer (or “Deck Slab”) for Models I, II and III and the deck section for Model IV- Thb- CCR 0.125m- 2nd Chang River Bridge

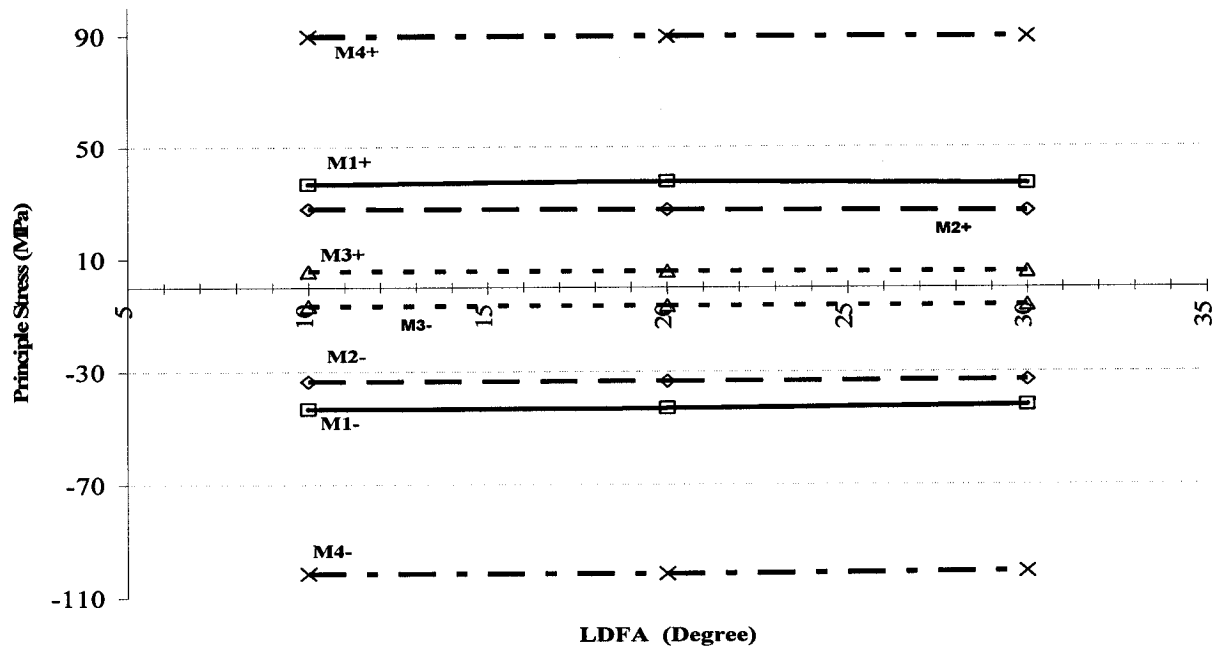


Figure 6.2.4.2.11 Laminas Dominant Fiber Alignment Effect on Principal Stresses at the Chopped FRP top layer (or “Deck Slab”) for Models I, II and III and the deck section for Model IV- Thb- CCR 0.150m- Tatar Bridge

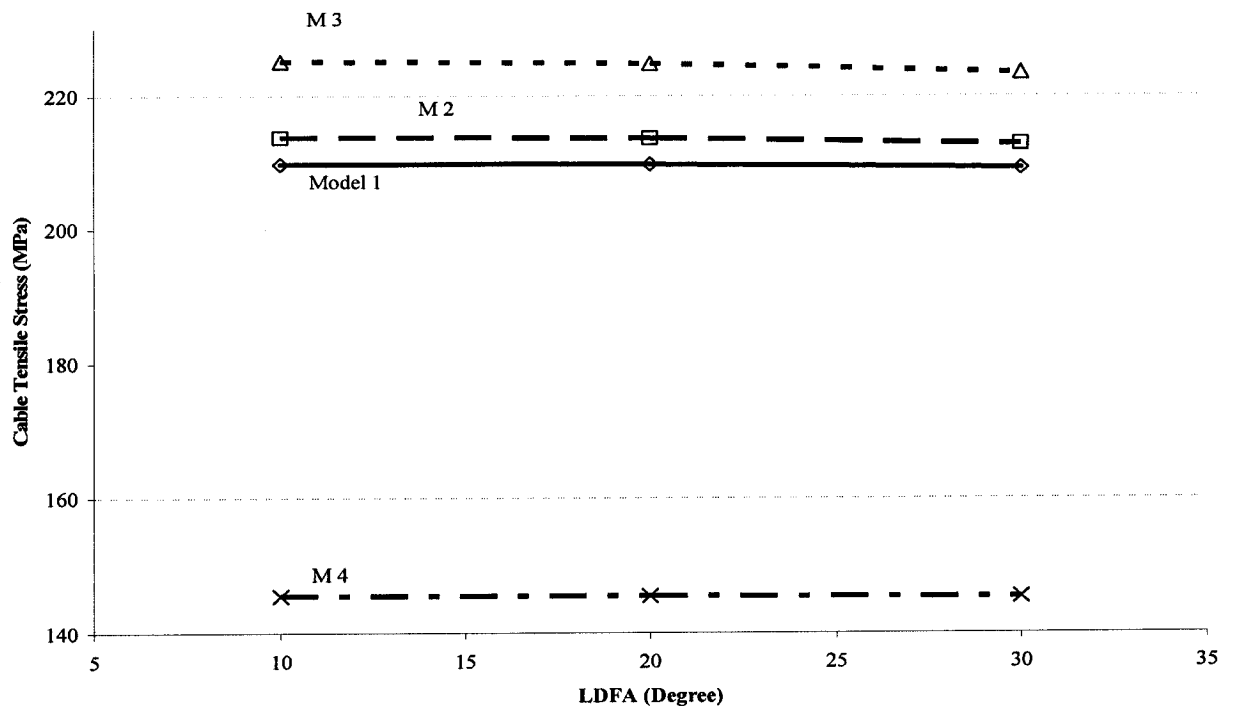


Figure 6.2.4.2.12 Laminas Dominant Fiber Alignment Effect on Maximum Cable Tension for different models, Thb- CCR 0.125m- 2nd Chang River Bridge

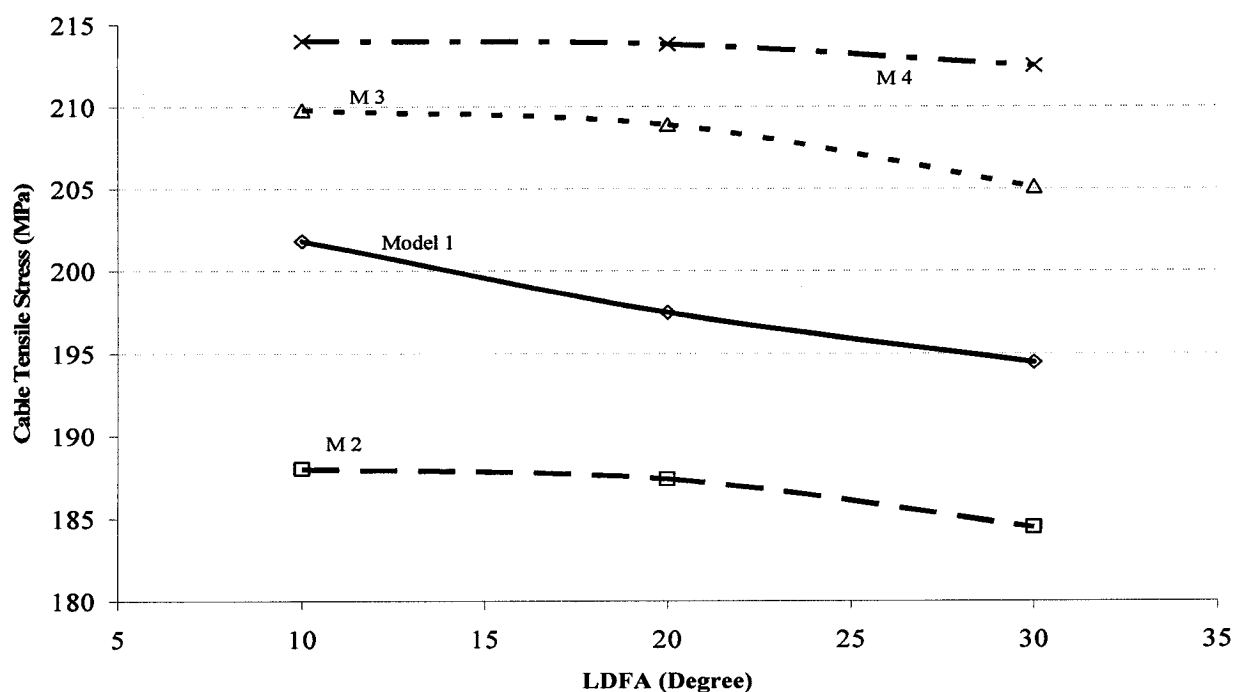


Figure 6.2.4.2.13 Laminas Dominant Fiber Alignment Effect on Maximum Cable Tension for different models, Thb- CCR 0.150m- Tataru Bridge

### 6.2.4.3 Effect of Laminas Dominant Fiber Alignment on the Vertical Natural Frequencies:

It has been found that the first three Vertical Natural Frequencies VNF are not affected by the increase of LDFA except Model I with the very long central span, where it slightly decreased. The high frequencies show high changes depending on the model and the bridge central span length.

Model I shows a decrease in the first four VNF with the Tataru Bridge, of the longest central span length (Figures 6.2.6.3, 4, 5 and 8). The stiffness to mass ratio becomes less as the bridge central span increases and the VNF becomes more sensitive to the deck stiffness change due to LDFA increase and the mass of this model is relatively high. Both the horizontal and the vertical laminates contribute in reducing the vertical flexural stiffness of the deck due to the increase of LDFA. This reduction in the vertical flexural stiffness explains the reduced VNF of this model.

Model II has relatively low flexural stiffness because of its very sharp edges and the lesser heights of many of its vertical laminates. However with the Second Chang River

Bridge, where the deck width is high, the moment of inertia of the horizontal laminates are greatly increased and overcome the slight reduction in the section moment of inertia. The modulus is decreased due to LDFA increase, and leads to reduce the deck stiffness. The reduction in the stiffness effect on the VNF is reduced because of the low mass of this model deck, and only causes a decrease in the fourth and fifth VNF (Figures 6.2.6.7 and 10). However, with the very long bridge central span, the low mass of Model II diminishes the effect of the stiffness reduction due to the increase of LDFA angle, and results in a relative increase of the high VNF (Figures 6.2.6.8 and 6.2.6.11).

On the other hand, the first three vertical natural frequencies of Models III and IV show a very slight increase due to the increase in the LDFA angle (Figures 6.2.6.1 to 6.2.6.6). However, the fourth and fifth VNF of Model III are increased in general (Figures 6.2.6.7 to 6.2.6.12). This can be related to the many internal tubular laminates of Model III, and the circular side tubular laminates of Model IV. The contribution to the vertical flexural stiffness of those tubular laminates increases with the rise of LDFA angle. However, the reducing effect of the horizontal laminates result in only a slight change in the high VNF.

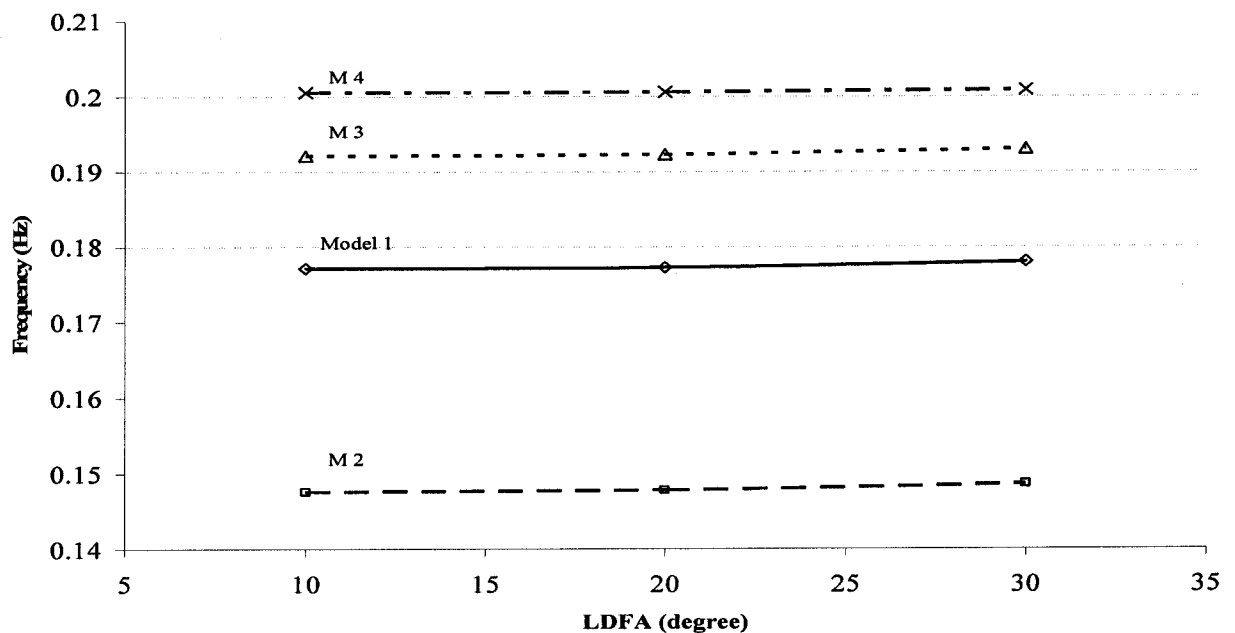


Figure 6.2.4.3.1 Laminas Dominant Fiber Alignment Effect on the 1st Vertical Natural Frequencies, Thb, CCR 0.100m - Annacis Island Bridge

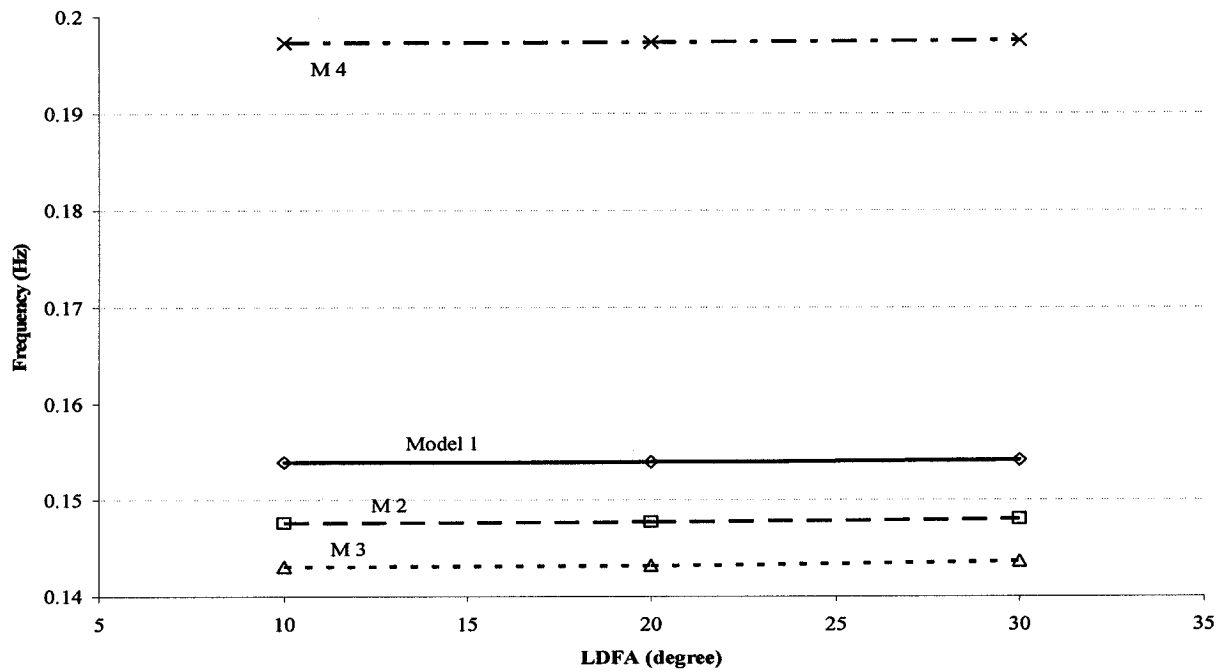


Figure 6.2.4.3.2 Laminas Dominant Fiber Alignment Effect on the 1st Vertical Natural Frequencies, Thb, CCR 0.125m- Second Chang River Bridge

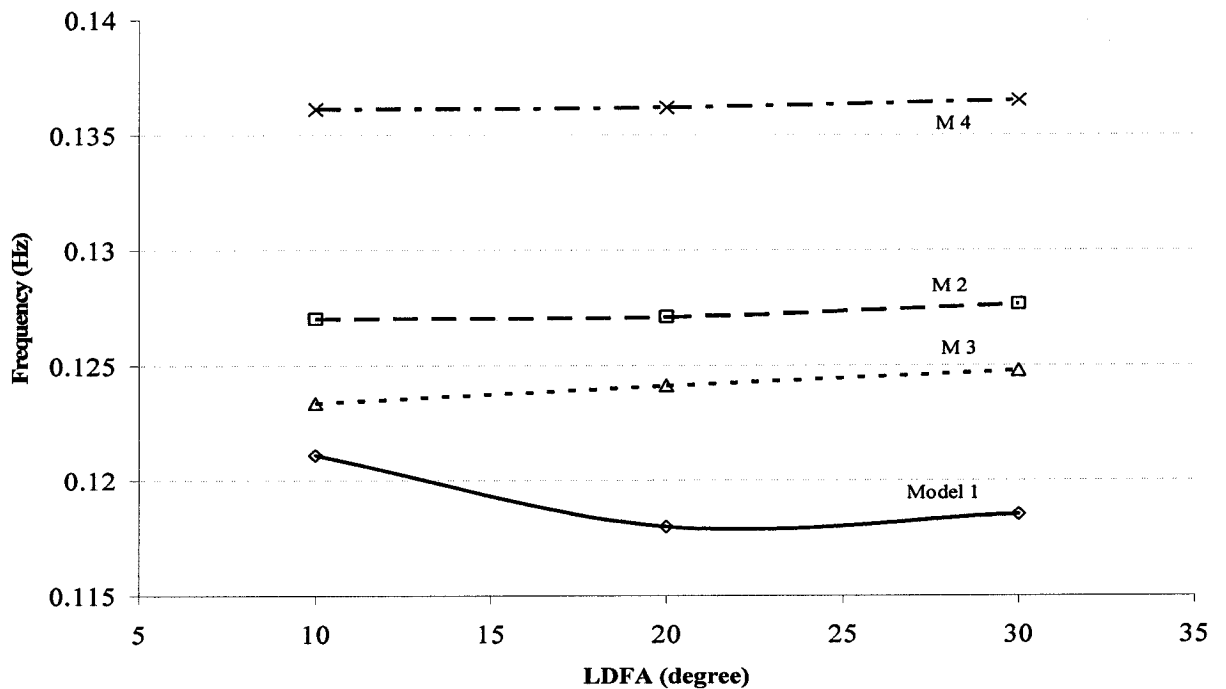


Figure 6.2.4.3.3 Laminas Dominant Fiber Alignment Effect on the 1st Vertical Natural Frequencies, Thb, CCR 0.150m- Tatara Bridge

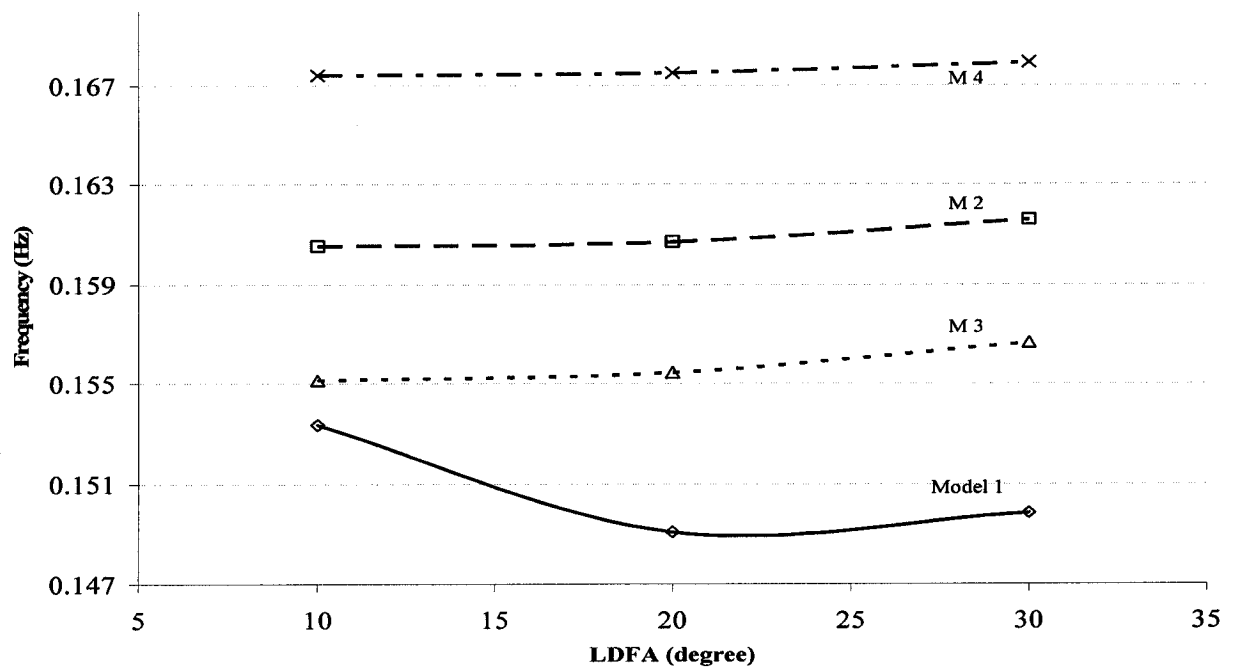


Figure 6.2.4.3.4 Laminas Dominant Fiber Alignment Effect on the 2nd Vertical Natural Frequencies, Thb, CCR 0.150m- Tatara Bridge Bridge

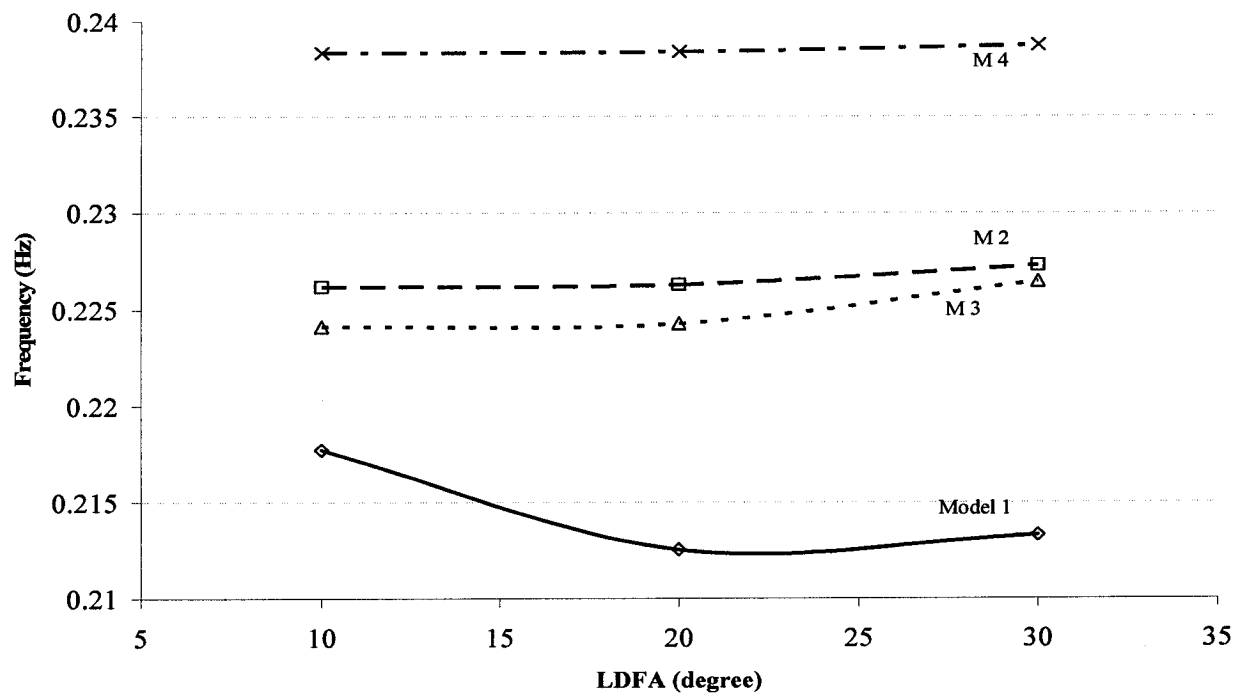


Figure 6.2.4.3.5 Laminas Dominant Fiber Alignment Effect on the 3rd Vertical Natural Frequencies, Thb, CCR 0.150m- Tatara Bridge Bridge



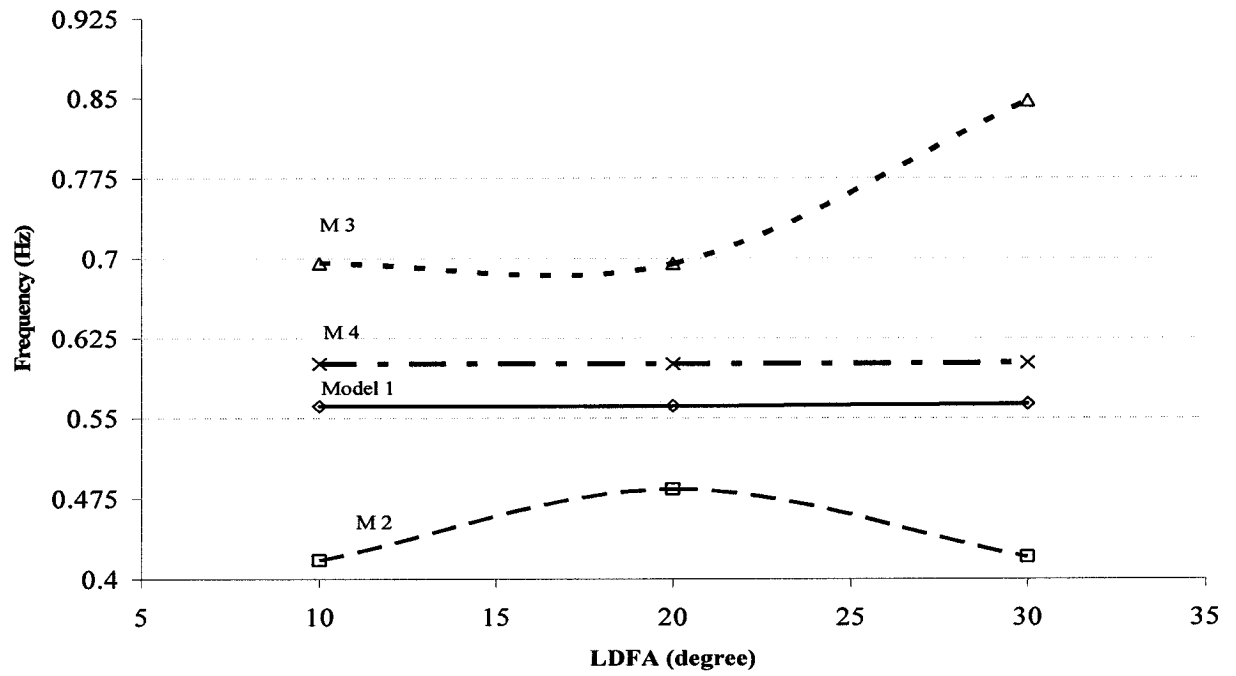


Figure 6.2.4.3.6 Laminas Dominant Fiber Alignment Effect on the 4th Vertical Natural Frequencies, Thb, CCR 0.100m- Annacis Island Bridge

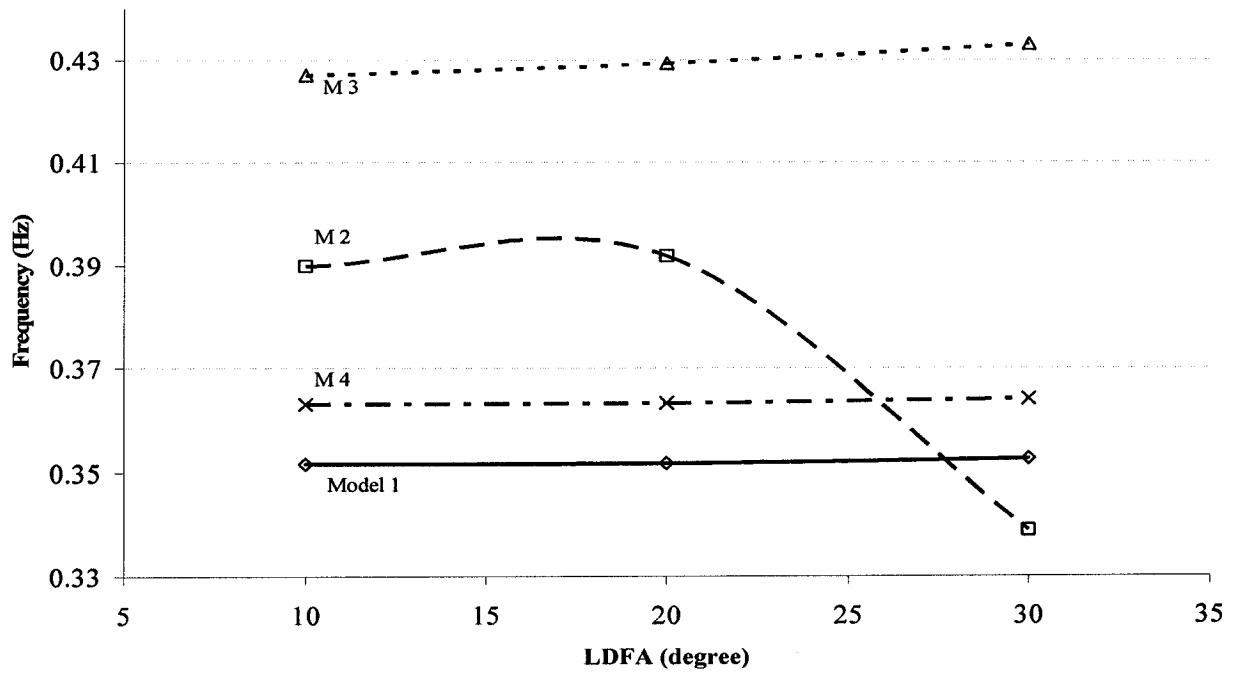


Figure 6.2.4.3.7 Laminas Dominant Fiber Alignment Effect on the 4th Vertical Natural Frequencies, Thb, CCR 0.125m- Second Chang River Bridge

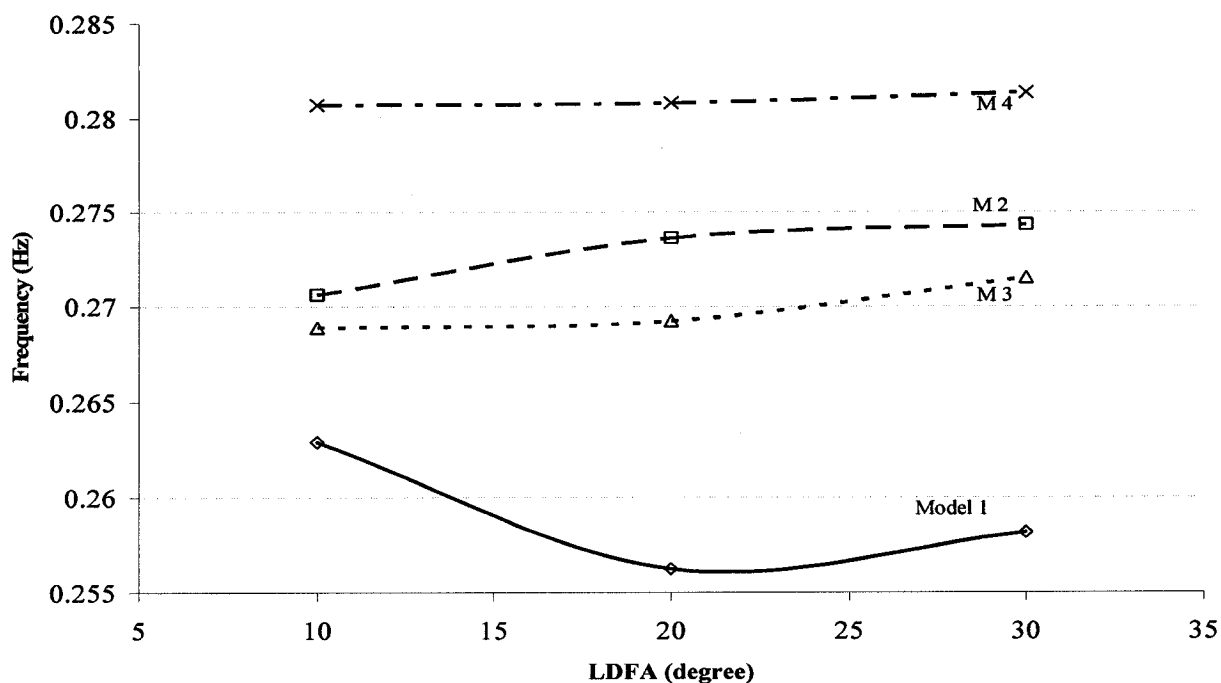


Figure 6.2.4.3.8 Laminas Dominant Fiber Alignment Effect on the 4th Vertical Natural Frequencies, Thb, CCR 0.150m- Tataru Bridge

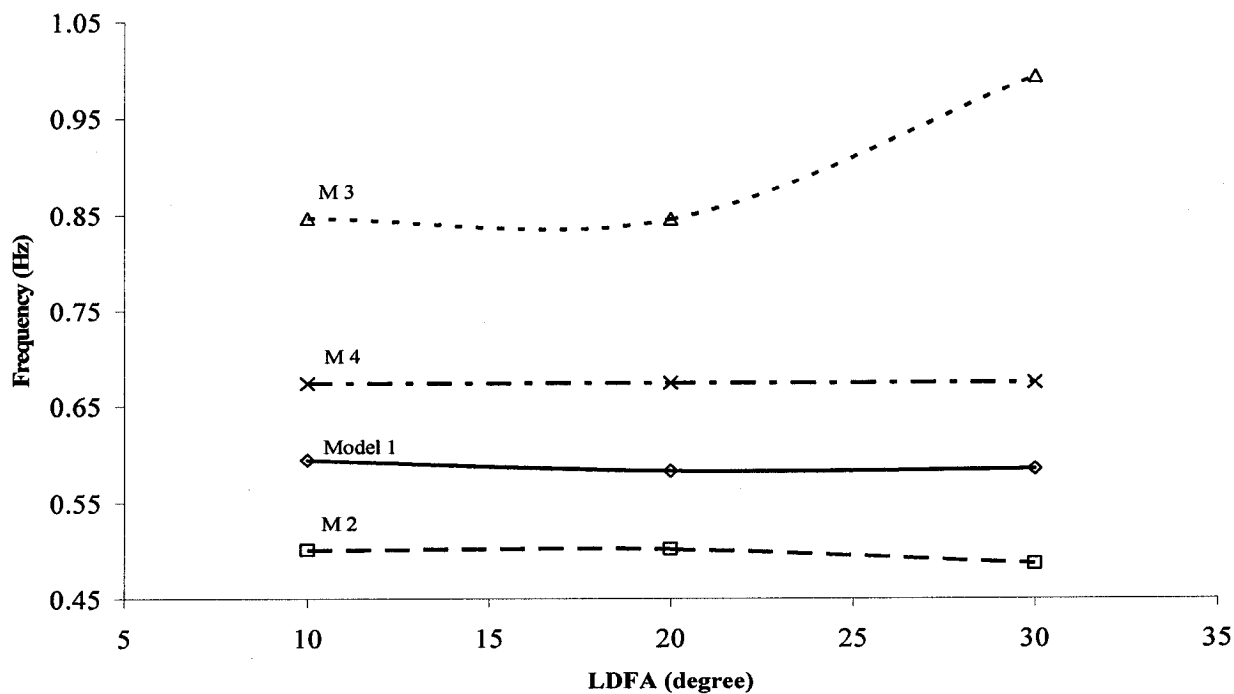


Figure 6.2.4.3.9 Laminas Dominant Fiber Alignment Effect on the 5th Vertical Natural Frequencies, Thb, CCR 0.100m- Annacis Island Bridge

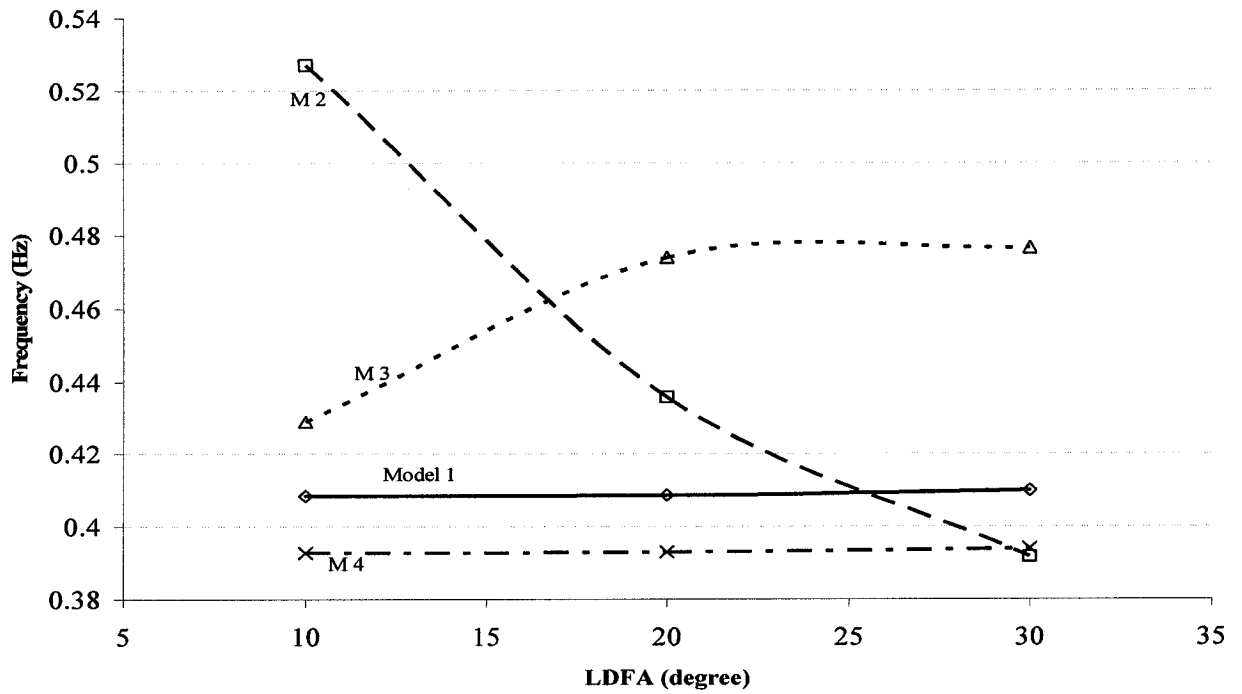


Figure 6.2.4.3.10 Laminas Dominant Fiber Alignment Effect on the 5th Vertical Natural Frequencies, Thb, CCR 0.125m- Second Chang River Bridge

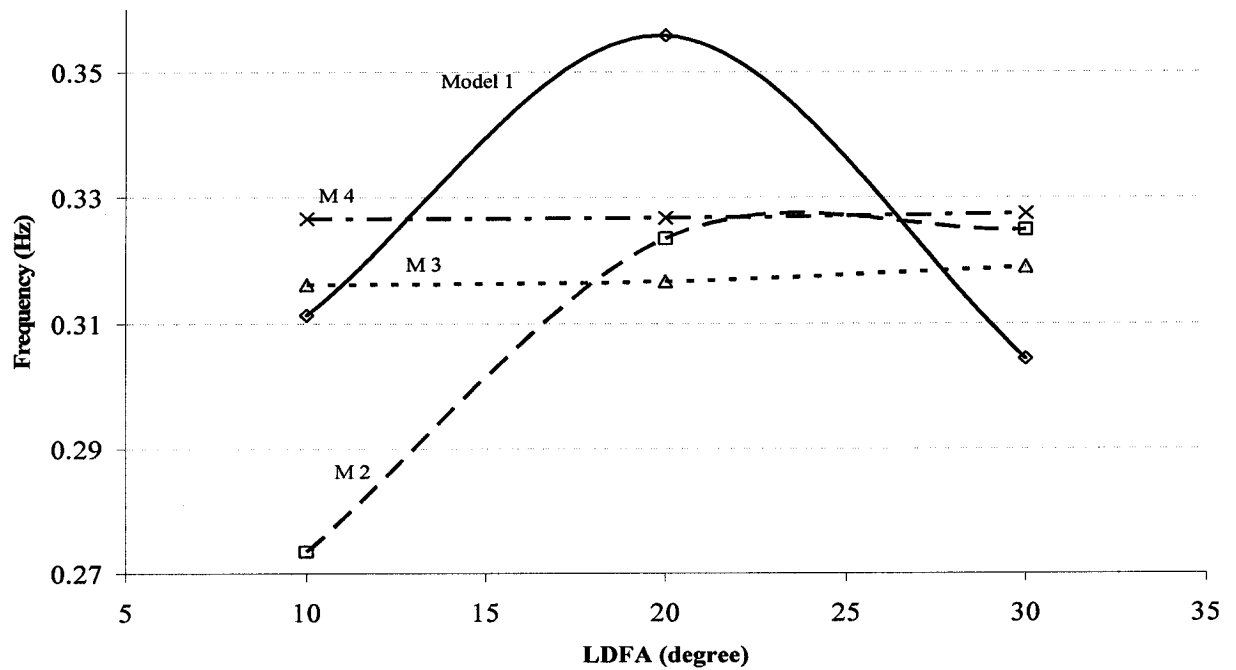


Figure 6.2.4.3.11 Laminas Dominant Fiber Alignment Effect on the 5th Vertical Natural Frequencies, Thb, CCR 0.150m- Tatara Bridge

#### **6.2.4.4 Effect of Laminate Dominant Fiber Alignment on the Torsional Natural Frequencies:**

The changes in the torsional natural frequencies TNF of different bridge models, due to the increase of LDFA, are small in general. With the longest bridge span length, TNF are almost constant, and only slightly dropped with the Model I. With the shortest and medium bridge central span lengths, Model II shows some changes starting with the Second TNF and Model III shows changes starting from the third TNF.

The TNF of all the proposed hybrid long-span cable-stayed bridge models are not affected by the LDFA angle increase when the bridge central-span is very long, with an exception related to the TNF of Model I, where it decreases by 2-4%. Figures 6.2.4.4.3 and 6.2.4.4.6 show typical TNF of the longest central span bridge or Tatara Bridge. With the shortest and medium bridge central span lengths, Model II shows some changes starting with the Second TNF and Model III shows changes starting from the third TNF. The balance of the deck torsional stiffness and mass contributions to the TNF plays important rule in the TNF variation. On the other hand, other parameters affect the influence of LDFA on the torsional stiffness, such as the width of the deck and the ratio between the central to the side spans. The changes in the torsional stiffness of different bridge models, due to the increase of LDFA, however they are small in general. The decrease in TNF of Model I for a very long central span, can be explained by the decrease of the torsional stiffness to mass ratio, as the central span increases, and makes the model more sensitive to slight changes of the torsional stiffness due to the increase of LDFA. The majority of laminas of all laminates and the “minor laminas” of the outer laminates gives reductions of torsional stiffness due to LDFA increase and this reduces the fluctuation of the contributions of the “minor laminas” of the inner laminates. On the other hand, Model I shows a constant TNF (Figures 6.2.4.4.1, 2, 4, 5, 7-10 and 12) for the shortest and medium bridge central span, except its fifth TNF of the Annacis Island Bridge (Figure 6.2.4.4.11), which shows an increase of 10%.

Model II shows a decrease in its TNF for short and medium bridge central spans (Figures 6.2.4.4.2, 4, 5, 7, 8, 10, 11 and 12) with one exception, that is Figure 6.2.4.4.9), where the fourth TNF increases then decreases with the Annacis Island Bridge. The change in TNF

is reduced as the frequency increases, i.e. the high change in the second TNF (up to 50%) is reduced with the third, fourth and fifth TNF to 15% then to 7%. Model II has relatively low torsional stiffness and the lowest mass. Similar to Model I, the majority of laminas of all laminates and the “minor laminas” of the outer laminates gives reductions of tortional stiffness due to LDFA increase and this reduces the fluctuation of the contributions of the “minor laminas” of the inner laminates. This becomes effective when the bridge central span is relatively short, and the TNF in this case is controlled by the deck stiffness. Model II also has sharp edges and very short (or low-height) vertical side laminates, as mentioned earlier, which reduces the ratio of the inner laminates to the outer laminates. When the central span increases, the low mass of the section leads the TNF, while the reduction of the stiffness becomes less effective.

Model III gives constant first and second TNF for the shortest and medium bridge central span (Figures 6.2.4.4.1, 2, 4, 5). However, its third, fourth and fifth TNF shows fluctuations, decreases then increases by 2-7% with shortest span, and increases then decreases by 5-18% with the medium bridge central span length (Figures 6.2.4.4.7, 8, 9, 10, 11, 12). Model III-TNF appears to be sensitive to the LDFA for the shortest and medium span, while the results are almost constant with the very long span. The ratio of the inner laminates to the outer laminates is high for this section. The section has many cylindrical laminates, which are of the inner type, and their stiffness change is relatively higher than the vertical laminates of models I and II, and hence they lead the stiffness fluctuation for the effect of their “minor laminas”. The changes of the stiffness due to the LDFA increase leads to a change of TNF in the short and medium span lengths. However for the longest span, the torsional stiffness of the cylindrical parts of the section decreases and its fluctuation effect vanishes. For the wide deck section of the Second Chang river, the effect of LDFA increase on the deck tortional stiffness is more, as it has more cylindrical laminates, and hence the TNF fluctuations for this bridge is higher than that of Annacis Island Bridge.

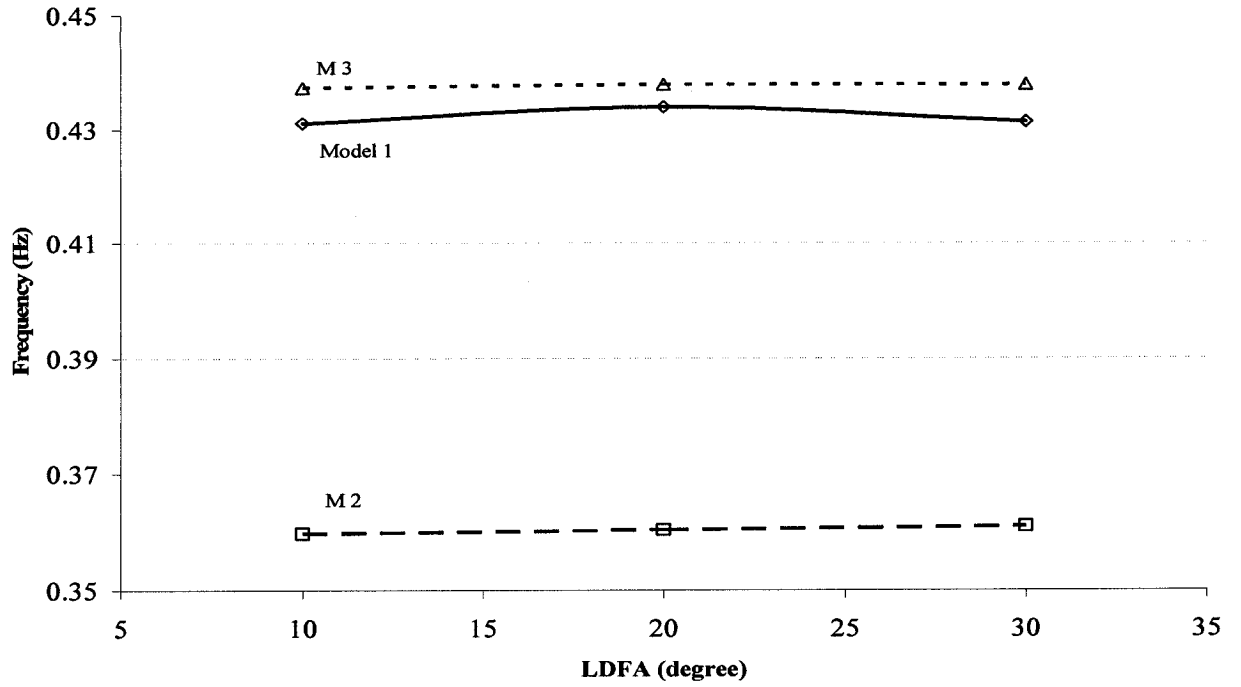


Figure 6.2.4.4.1 Laminas Dominant Fiber Alignment Effect on the 1st Torsional Natural Frequencies, Thb, CCR 0.100m- Annacis Island Bridge

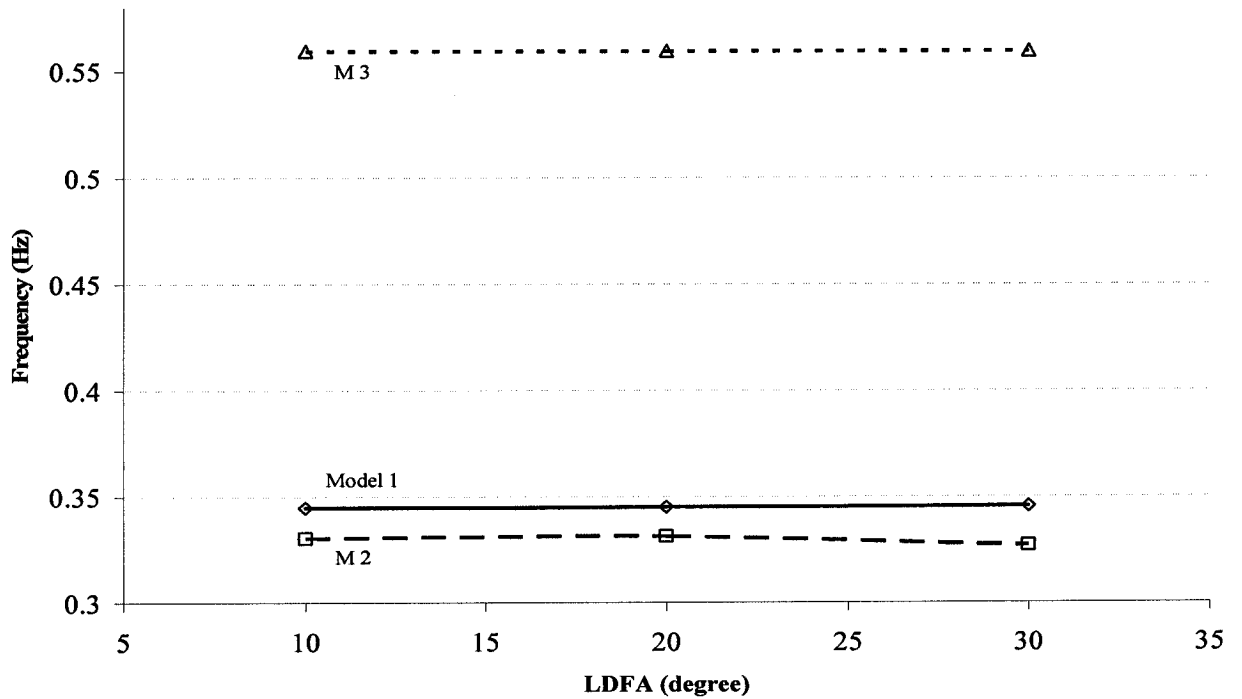


Figure 6.2.4.4.2 Laminas Dominant Fiber Alignment Effect on the 1st Torsional Natural Frequencies, Thb, CCR 0.125m- Second Chang River Bridge

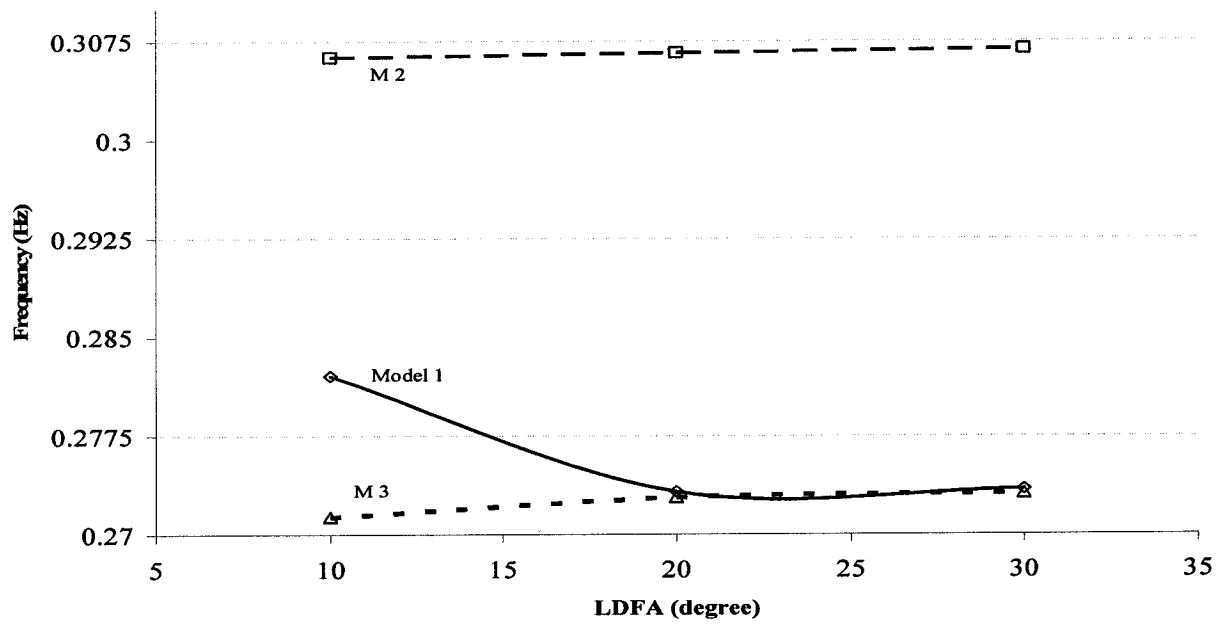


Figure 6.2.4.4.3 Laminas Dominant Fiber Alignment Effect on the 1st Torsional Natural Frequencies, Thb, CCR 0.150m- Tatar Bridge Bridge

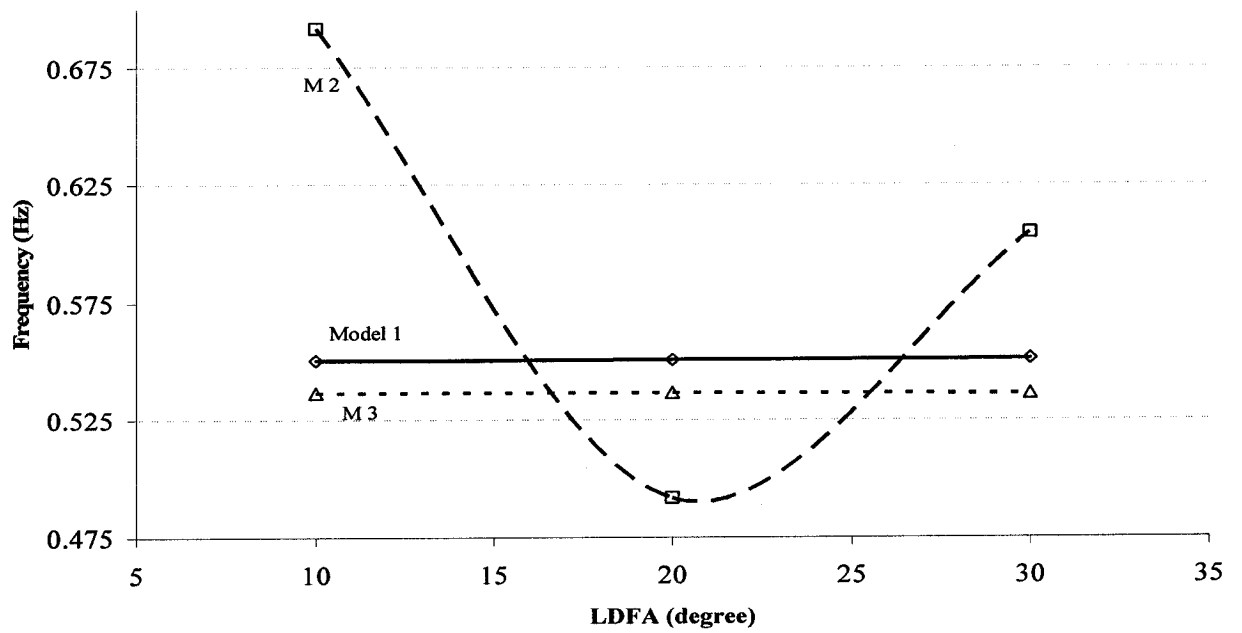


Figure 6.2.4.4.4 Laminas Dominant Fiber Alignment Effect on the 2nd Torsional Natural Frequencies, Thb, CCR 0.100m- Annacis Island Bridge

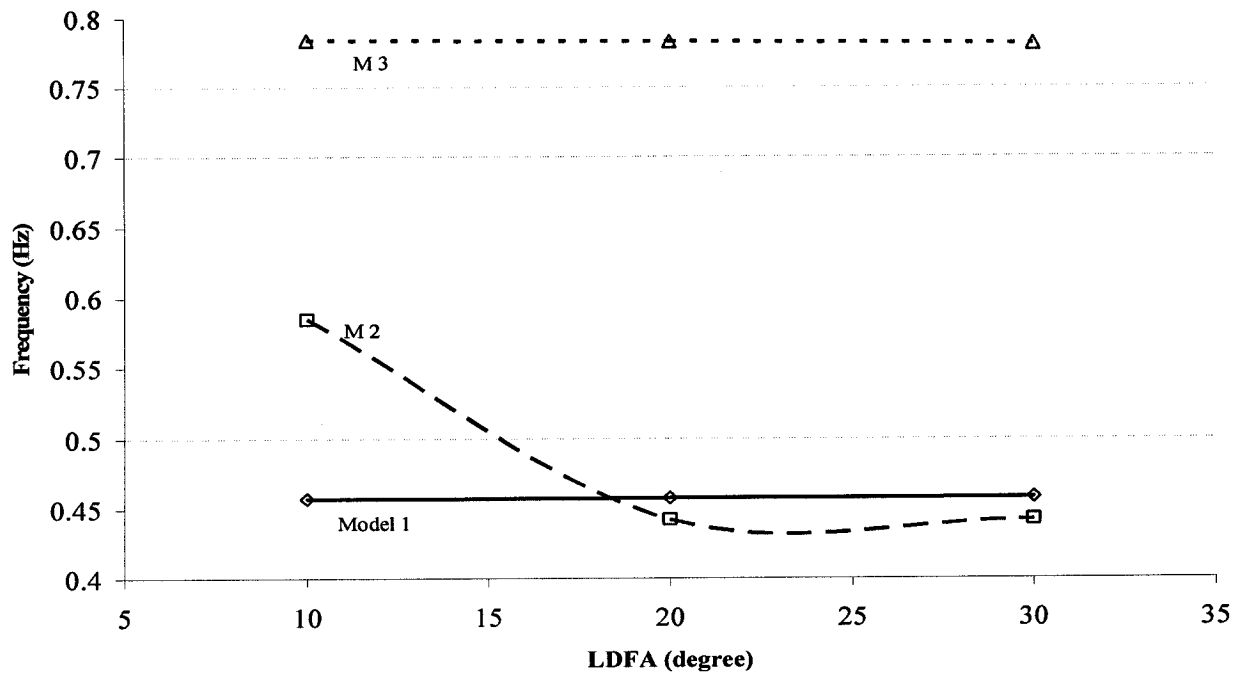


Figure 6.2.4.4.5 Laminas Dominant Fiber Alignment Effect on the 2nd Torsional Natural Frequencies, Thb, CCR 0.125m- Second Chang River Bridge

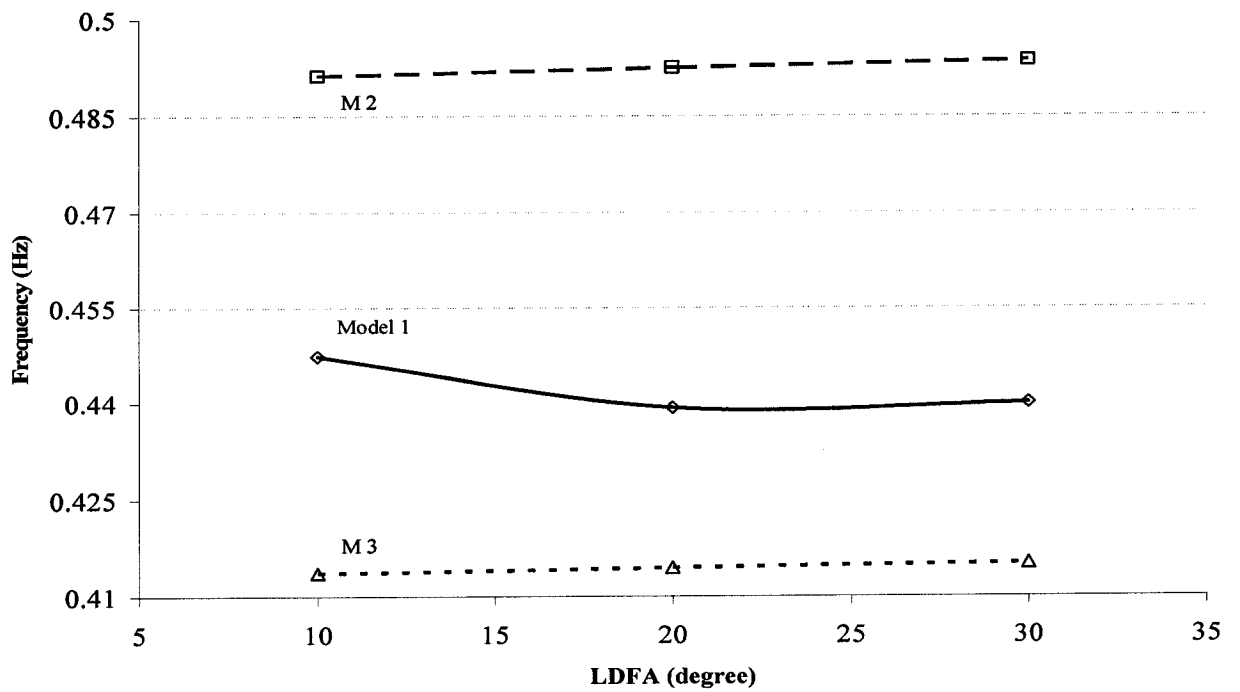


Figure 6.2.4.4.6 Laminas Dominant Fiber Alignment Effect on the 2nd Torsional Natural Frequencies, Thb, CCR 0.150m- Tatara Bridge



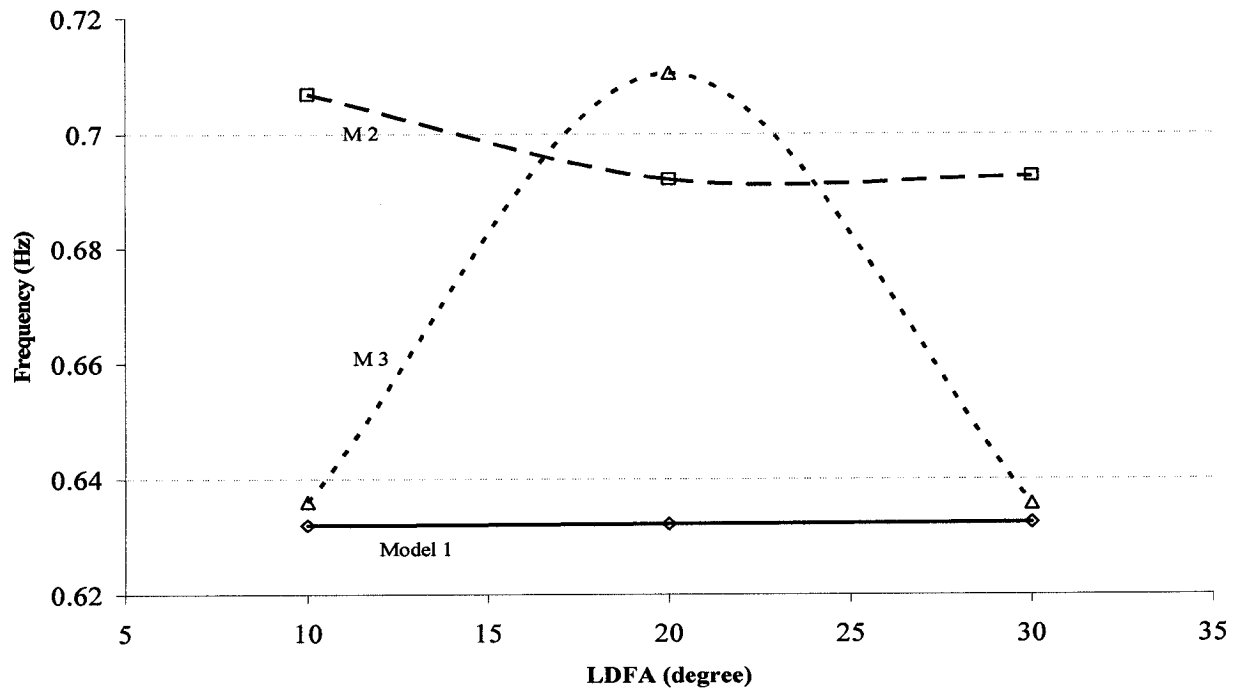


Figure 6.2.4.4.7 Laminas Dominant Fiber Alignment Effect on the 3rd Torsional Natural Frequencies, Thb, CCR 0.100m- Annacis Island Bridge

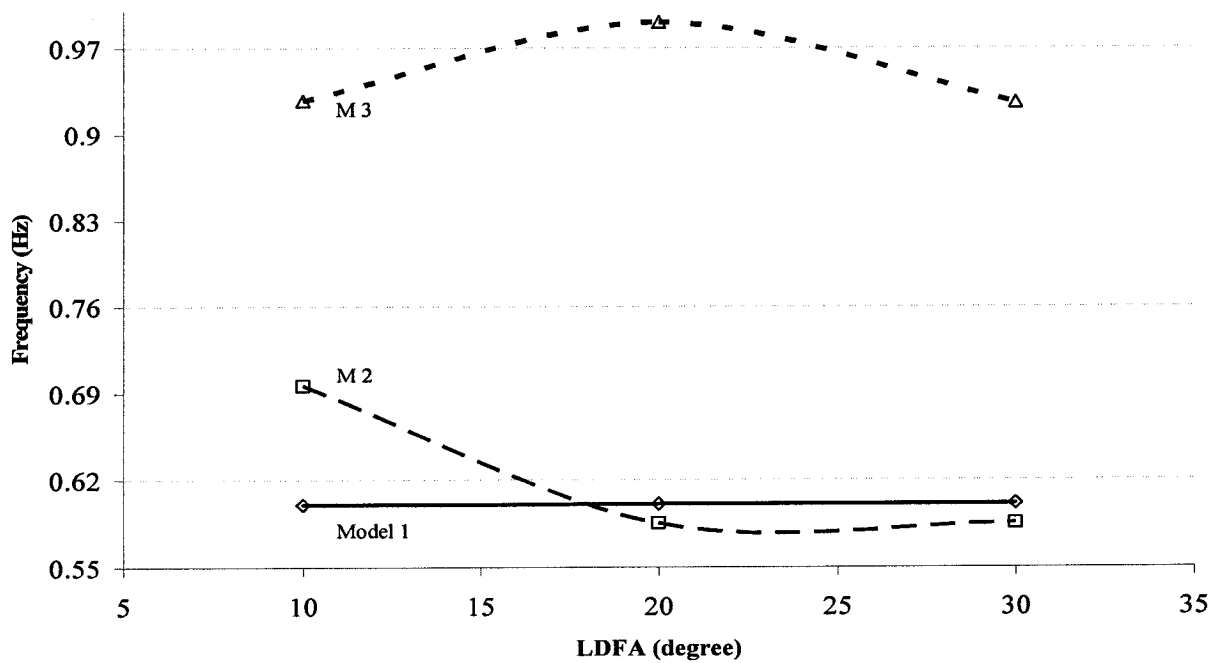


Figure 6.2.4.4.8 Laminas Dominant Fiber Alignment Effect on the 3rd Torsional Natural Frequencies, Thb, CCR 0.125m- Second Chang River Bridge

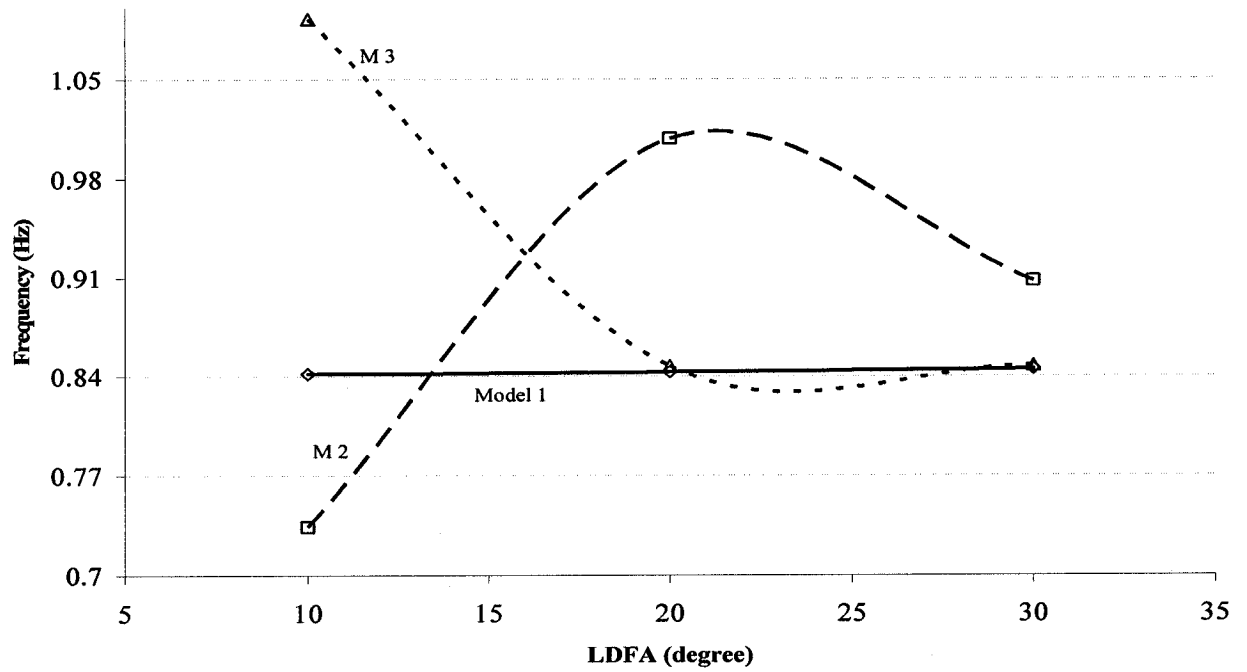


Figure 6.2.4.4.9 Laminas Dominant Fiber Alignment Effect on the 4th Torsional Natural Frequencies, Thb, CCR 0.100m- Annacis Island Bridge

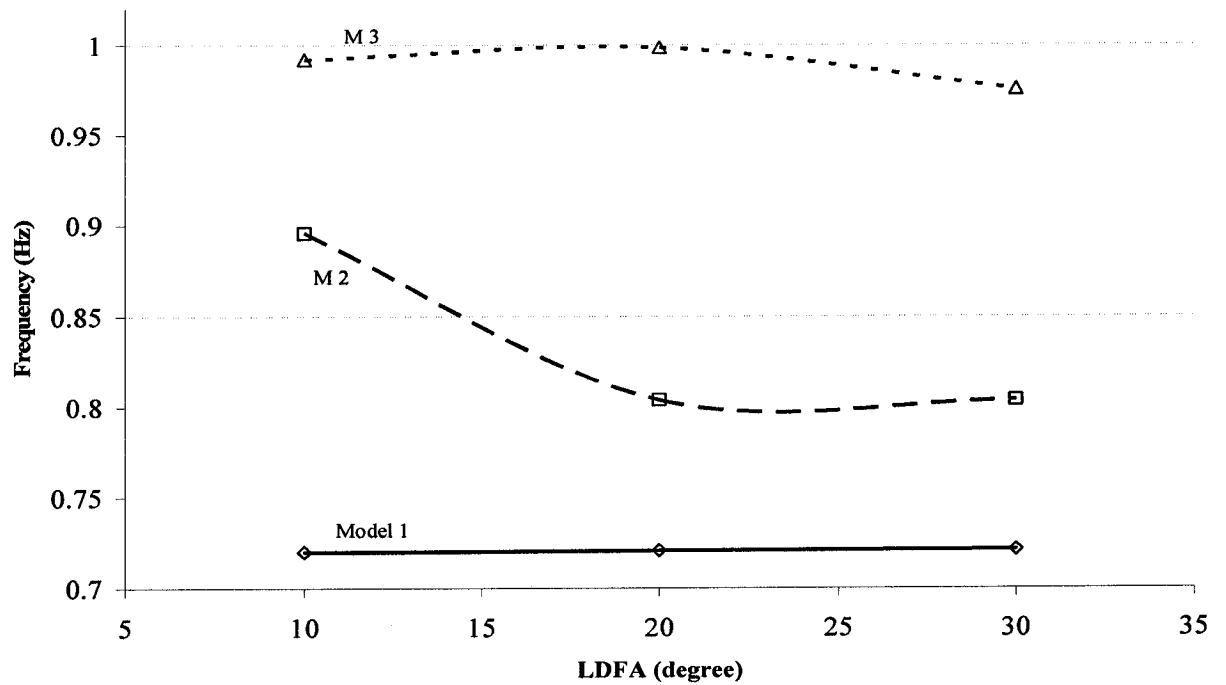


Figure 6.2.4.4.10 Laminas Dominant Fiber Alignment Effect on the 4th Torsional Natural Frequencies, Thb, CCR 0.125m- Second Chang River Bridge

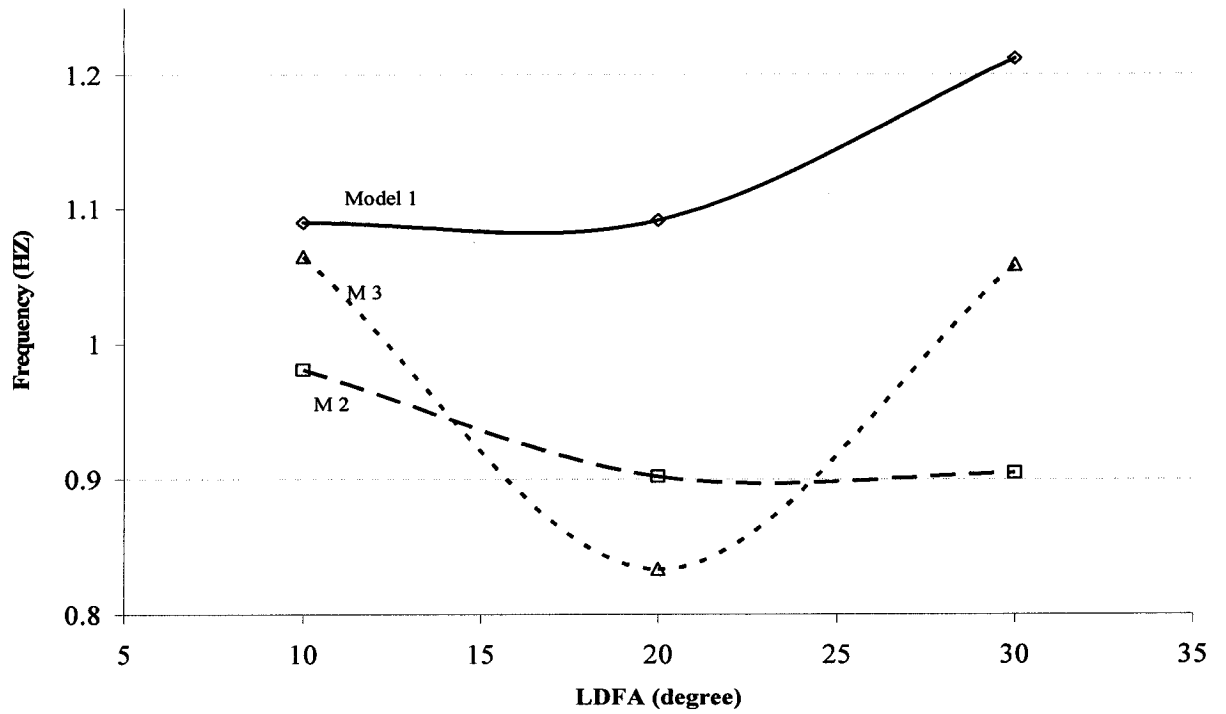


Figure 6.2.4.4.11 Laminas Dominant Fiber Alignment Effect on the 5th Torsional Natural Frequencies, Thb, CCR 0.100m- Annacis Island Bridge

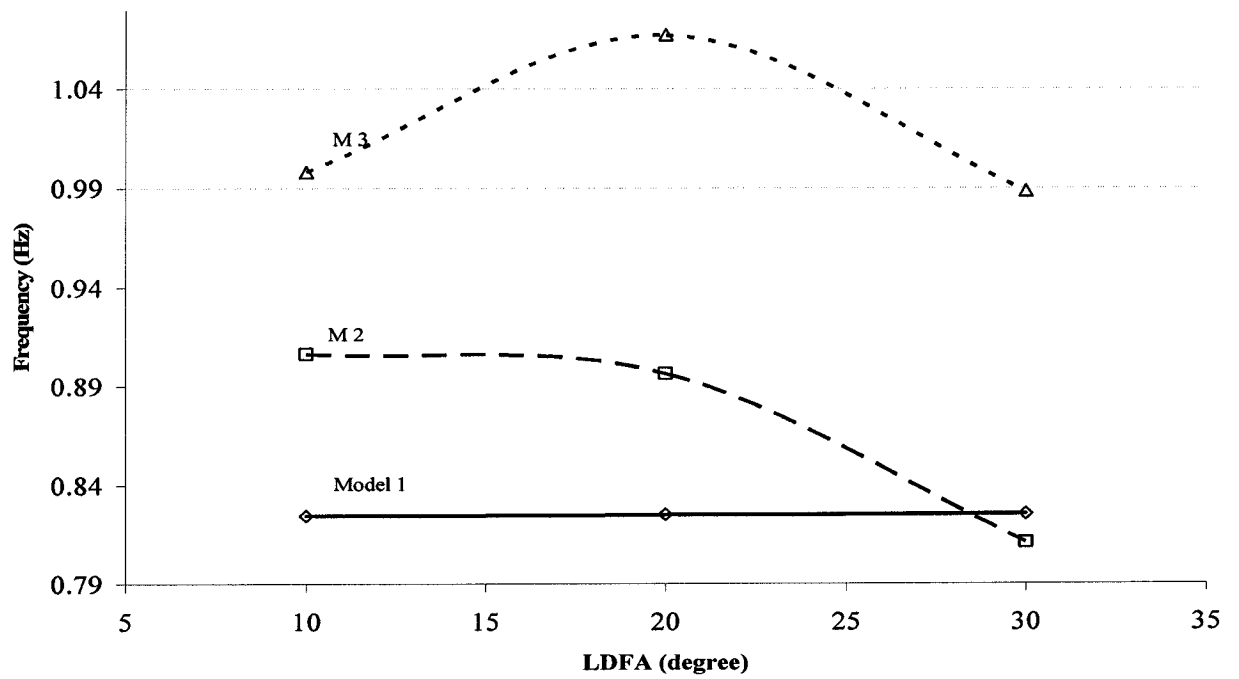


Figure 6.2.4.4.12 Laminas Dominant Fiber Alignment Effect on the 5th Torsional Natural Frequencies, Thb, CCR 0.125m- Second Chang River Bridge

#### 6.2.4.5 Effect of Laminate Dominant Fiber Alignment on the Horizontal Natural Frequencies:

The Horizontal Natural Frequencies of the proposed hybrid long span cable stayed bridges models and for all the examined central spans are almost constant or have a slight change of less than 4%. The horizontal natural frequencies can be related to the horizontal (or lateral) flexural stiffness of the deck, which is much higher than the vertical flexural stiffness for all the bridge models. The change in the horizontal flexural stiffness due to LDFA increase is small, and hence its effect on the HNF is slight. The increase of the flexural stiffness due to the increase of LDFA of the models with tubular laminates (Model III and IV) results in the slight increase of the HNF (Figures 6.2.8.2 and 6.2.8.4). On the other hand the decrease of the horizontal and the vertical flexural stiffnesses due to LDFA increase of the models with only flat laminates (Models I and II) leads to a decrease of HNF with the medium central span length. The shortest and the longest span lengths show almost no variation of HNF with the LDFA increase (Figures 6.2.8.1, 3 and 5).

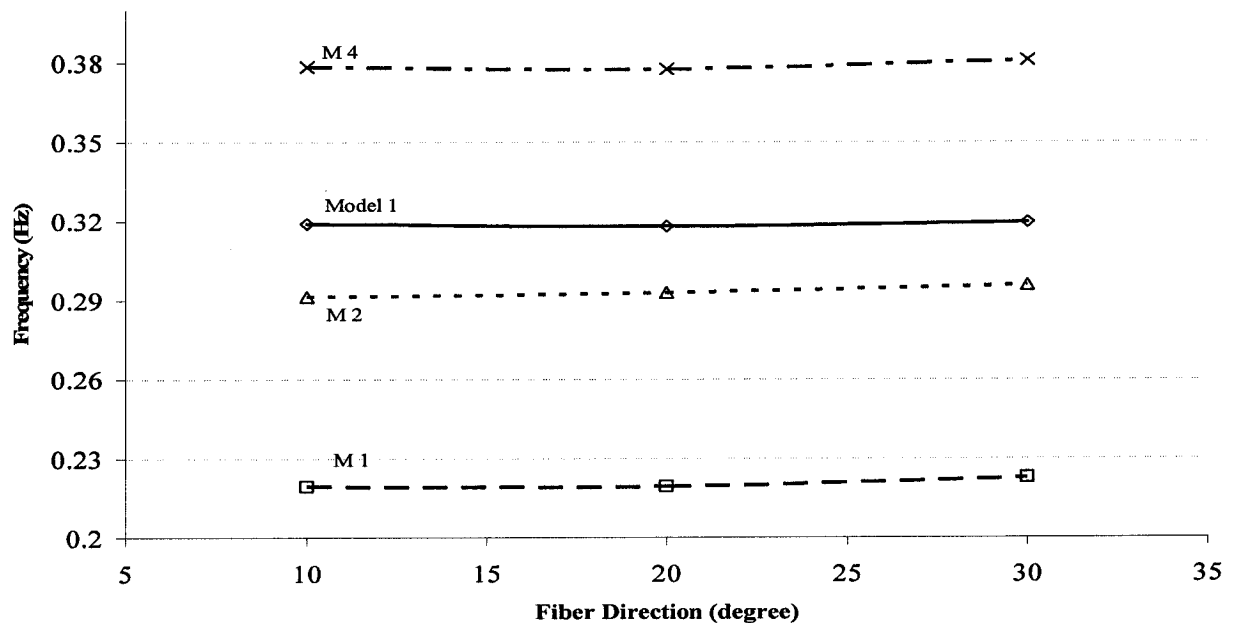


Figure 6.2.4.5.1 Laminas Dominant Fiber Alignment Effect on the 1st Horizontal Natural Frequencies, Thb, CCR 0.100m- Annacis Island Bridge

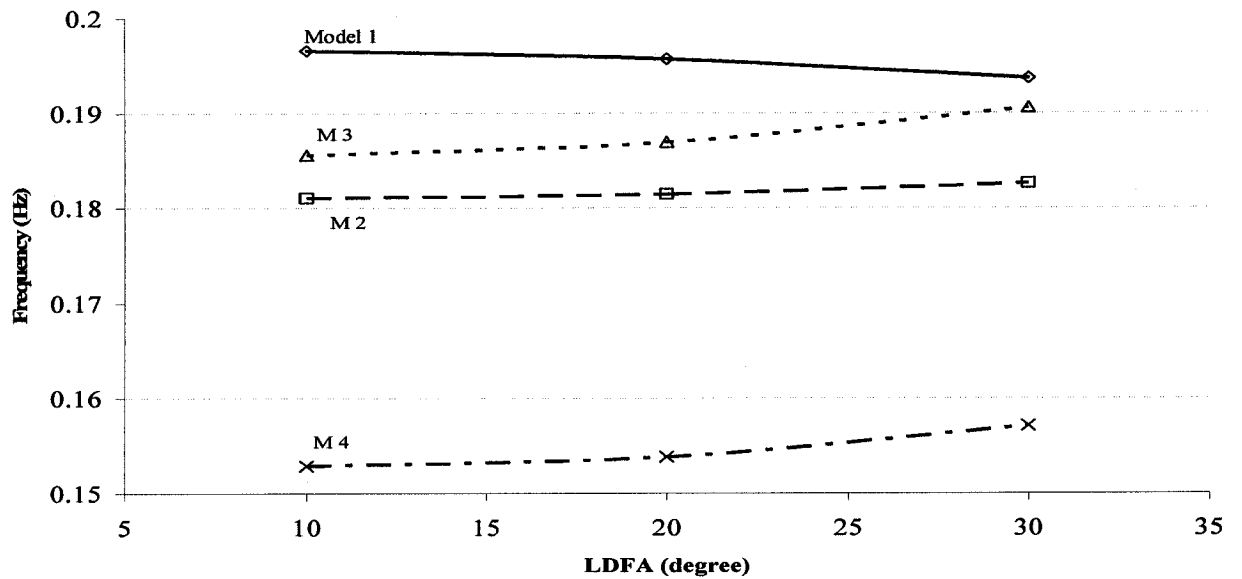


Figure 6.2.4.5.2 Laminas Dominant Fiber Alignment Effect on the 1st Horizontal Natural Frequencies, Thb, CCR 0.125m- Second Chang River Bridge

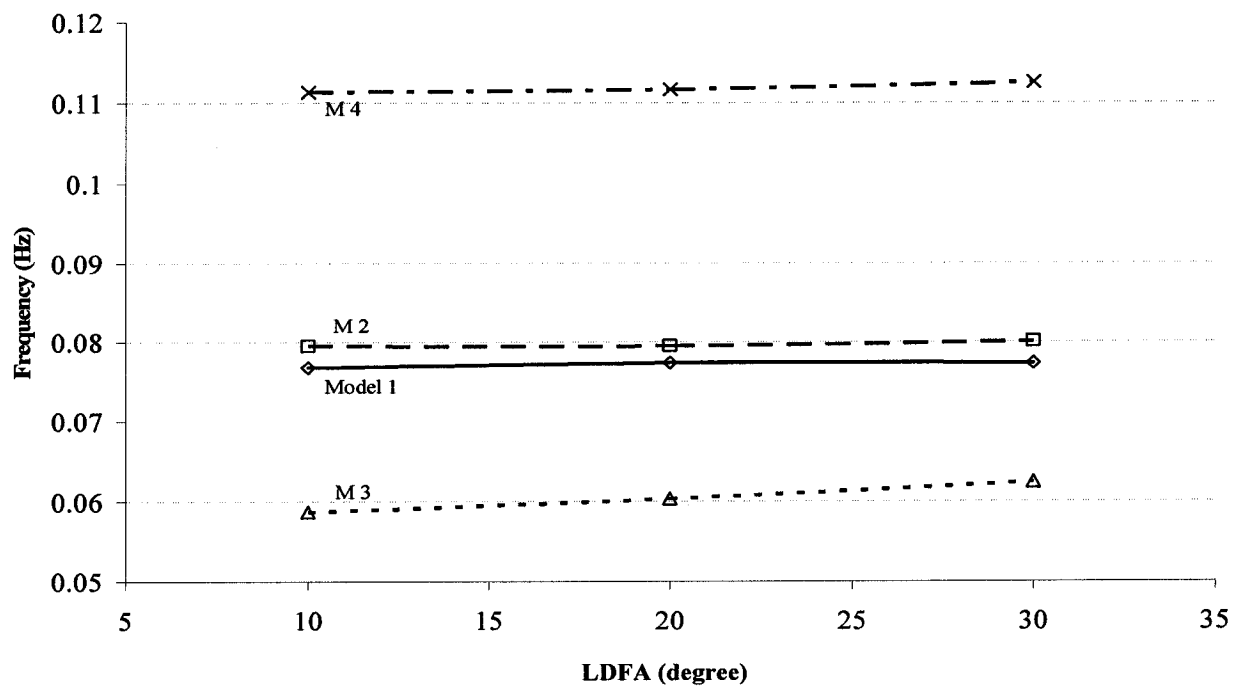


Figure 6.2.4.5.3 Laminas Dominant Fiber Alignment Effect on the 1st Horizontal Natural Frequencies, Thb, CCR 0.150m- Tatara Bridge

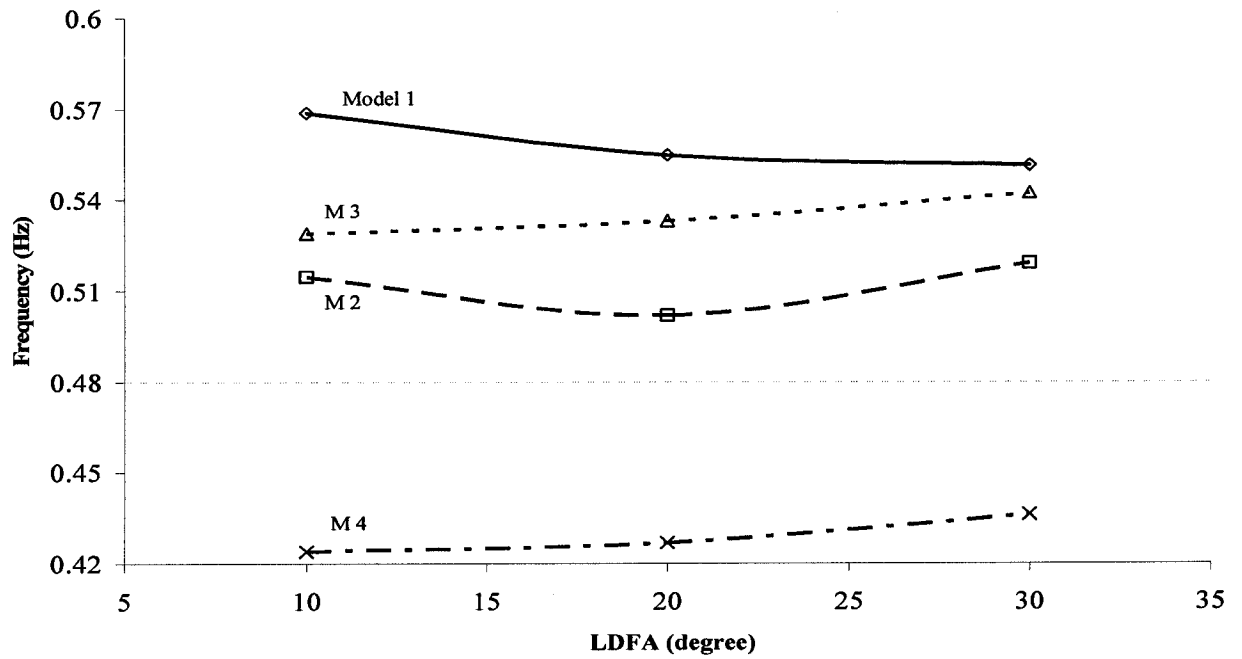


Figure 6.2.4.5.4 Laminas Dominant Fiber Alignment Effect on the 2nd Horizontal Natural Frequencies, Thb, CCR 0.125m- Second Chang River Bridge

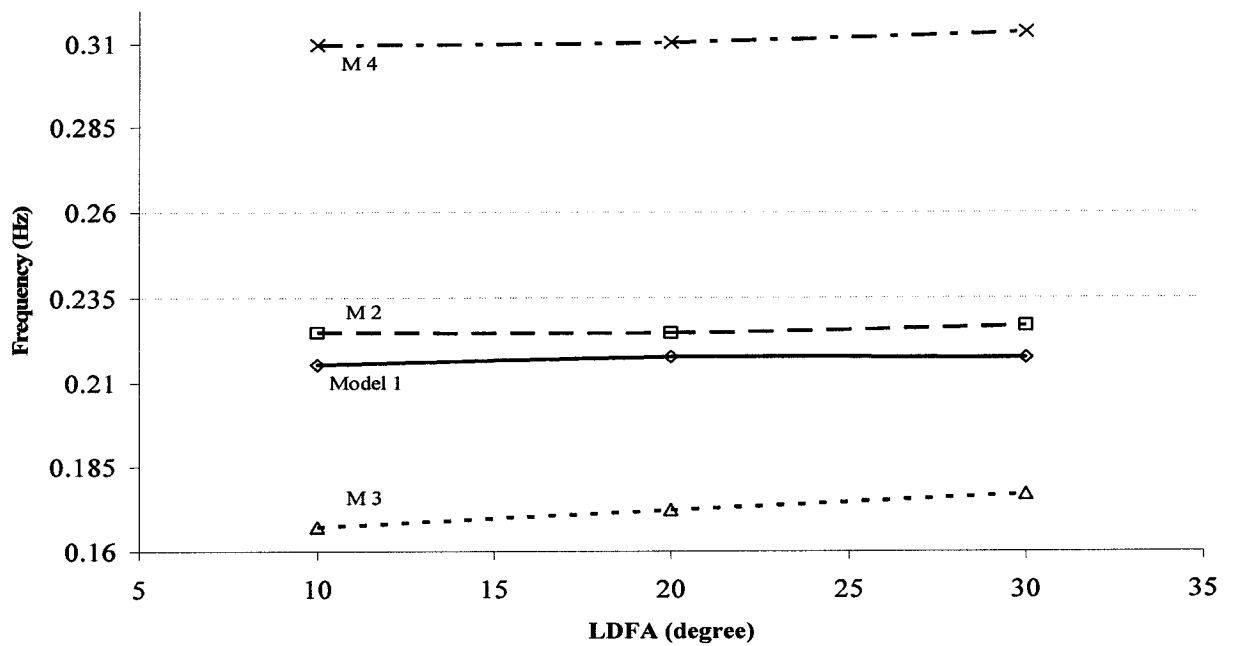


Figure 6.2.4.5.5 Laminas Dominant Fiber Alignment Effect on the 2nd Horizontal Natural Frequencies, Thb, CCR 0.150m- Tatara Bridge

## **6.2.5 Effect of Laminate Thickness:**

### **6.2.5.1 Effect of Laminate Thickness on the Maximum Deck Top Surface**

#### **Displacements:**

Considering the most critical static loading case, Live Load Case A, which represents full traffic lane loads applied on the bridge central span only and the side bridge spans are loaded by the superimposed dead loads only. The increase of the laminate thickness appears to be a very effective parameter on the deck displacements. It has been found that all the displacements of the hybrid long span cable-stayed bridge deck are reduced with the increase of the laminate thickness. However, the rates of reduction of the displacements are different from model to other. While Model II seems to be the most sensitive, Model IV is the one with the minimum rate of change. In most of the cases, the rate of decrease in the displacements is further reduced as the laminate thickness increased. From the static point of view, this would lead to an economical laminate thickness for every one of the four proposed models, as increasing the laminate thickness beyond a certain range does not result in any significant reduction in the deck maximum deformations.

It has been observed that the lateral displacement  $U$  (Figures 6.2.5.1.1– 6.2.5.1.3) is reduced in general as the laminate thickness (LTh) increases. Due to the apparent anisotropy of laminated parts of the deck, and the resulting bend-twist coupling, the lateral displacement shows the tendency of the deck to move in an unsymmetrical pattern to one side in the unsupported or elastically supported spans. This is analogous to the lateral buckling of thin-walled structures. As the laminate thickness increases, the lateral moments of inertia are greatly increased and cause the observed reduction in the maximum lateral displacement of all models. On the other hand, according to the deck cross-sectional details, every model has a different level and a different variation pattern for the horizontal (or lateral) moment of inertia. This results in different variation patterns for different models. Model II has a lower lateral moment of inertia, and with the longest bridge central span, its inertia does not affect its maximum lateral displacement until the laminate thickness becomes high. Model IV laminates do not form a box girder, and the intermediate stiffening laminate is more responsive to the stiffness increase, giving a

relatively higher reduction in its maximum U displacement as its width is high (the Second Chang River Bridge).

The vertical displacements V of different models (Figures 6.2.5.1.4 and 6.2.5.1.5) show a reduction in their maximum values for all the bridge span lengths, as the laminate thickness (LTh) increases. For the deck deflections, the deck “vertical” flexural stiffness, and the central span dead load are the major factors affecting its maximum values. Model III has almost the same vertical moment of inertia of Model I but has slightly less mass. As a result, Model III shows a slightly higher rate of reduction in its maximum deflection than Model I for all span lengths as the laminate thickness increases. Model II gives very high rate of decrease in its maximum deflection for the Tatara Bridge, or the very long central span. The increase of its deck self dead load due to the increase of the laminate thickness is lower than other models. Model IV has the minimum laminated parts in the cross section. The increase in the deck stiffness and the deck self weight in the central span as the laminate thickness increases are small, and result in the shown small variation of this model deflection.

As the laminate thickness increases, the longitudinal displacements W are reduced slightly in general, except for few cases where the decrease is relatively high (Figures 6.2.5.1.6 – 6.2.5.1.8). The high increase of the deck vertical flexural stiffness reduces the forces transferred through the cable stays, and increases the forces transferred through the deck itself. However the high stiffness to mass ratio of the composite material, makes the stiffness increase higher than the increase of the central span dead load, as the laminate thickness increases. On the other hand the axial deformability of the deck is highly reduced by its sectional area enhancement. Hence the rates of rise in through-deck-transferred loads are less than the rates of increase in the decks area and stiffness, which leads to reduce the longitudinal displacements. A balance is occurring in most models with the Second Chang River Bridge, where the central span to the side span ratio is the lowest, and the forces transferred through the deck is higher than in other cases. Model II shows a higher rate of decrease in its maximum W as its stiffness is lower than other models and the forces transferred through the deck are lower. With the longest spans, Model III gives higher reduction in its already high longitudinal displacement as it shows reasonably high stiffness to weight ratio.



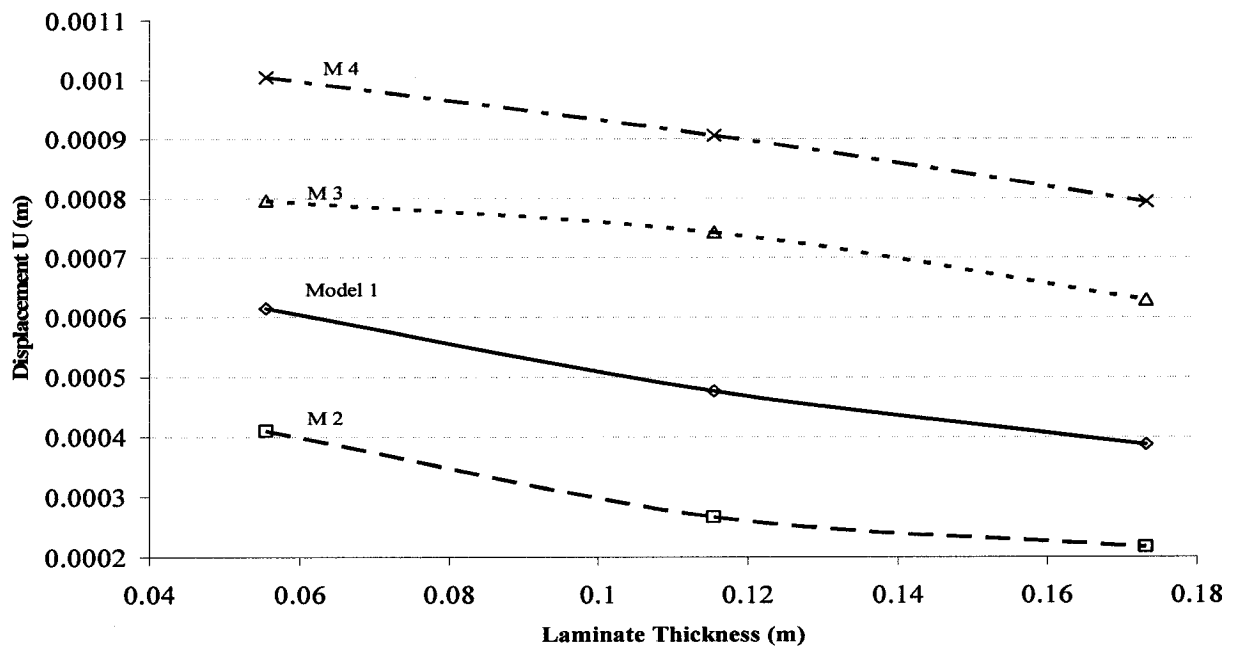


Figure 6.2.5.1.1 Laminate Thickness Effect on Lateral Displacement (U) – LDFA-a - CCR 0.100m - Annacis Island Bridge

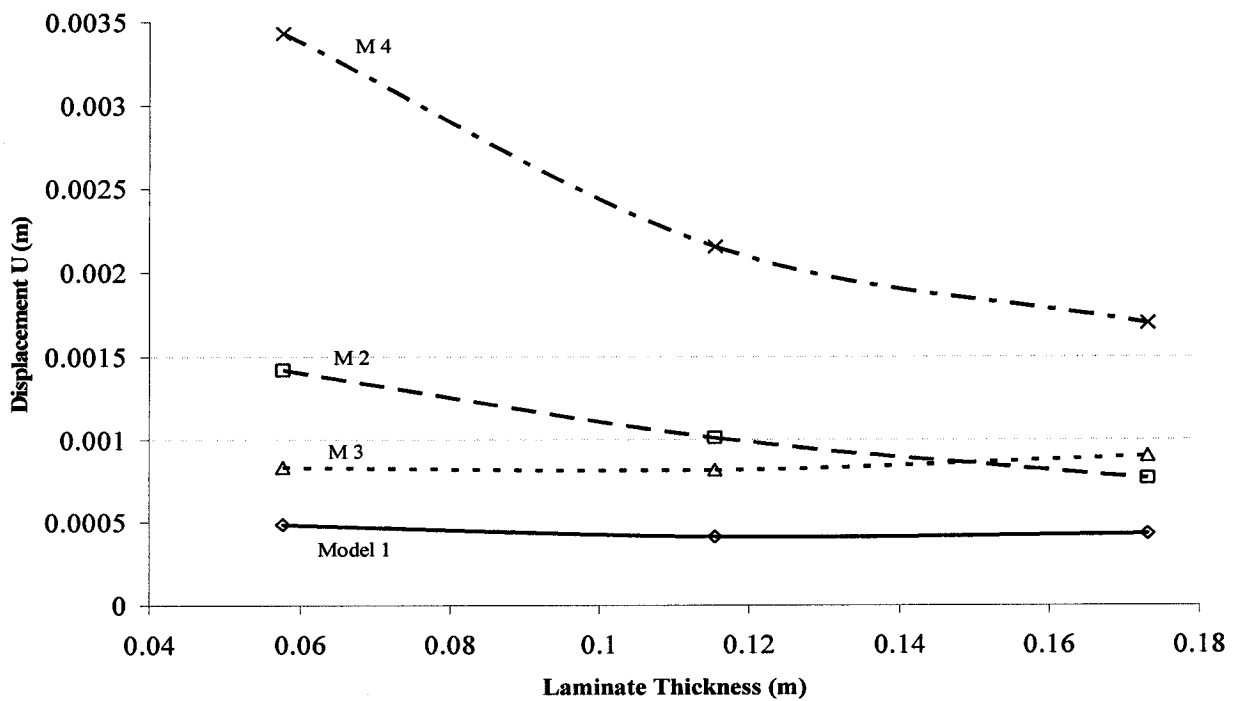


Figure 6.2.5.1.2 Laminate Thickness Effect on Lateral Displacement (U) – LDFA-a-LLa- CCR 0.125m – Second Chang River Bridge

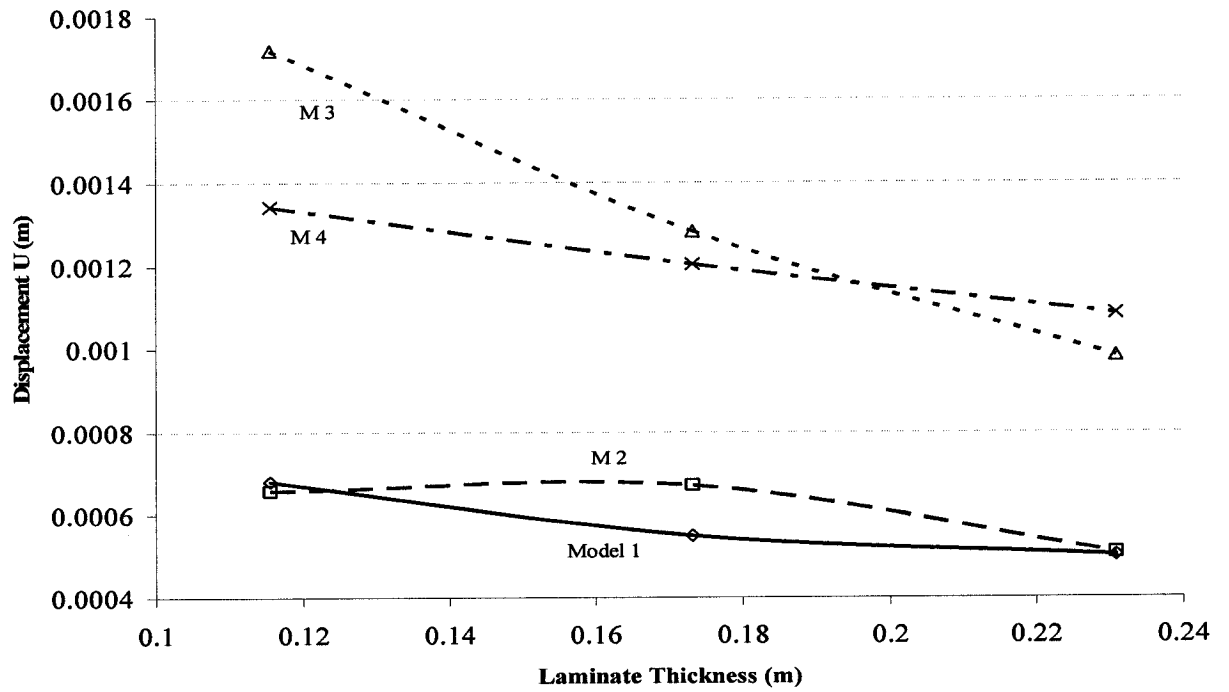


Figure 6.2.5.1.3 Laminate Thickness Effect on Lateral Displacement (U) – LDFA-a-LLa- CCR 0.150m – Tatara Bridge

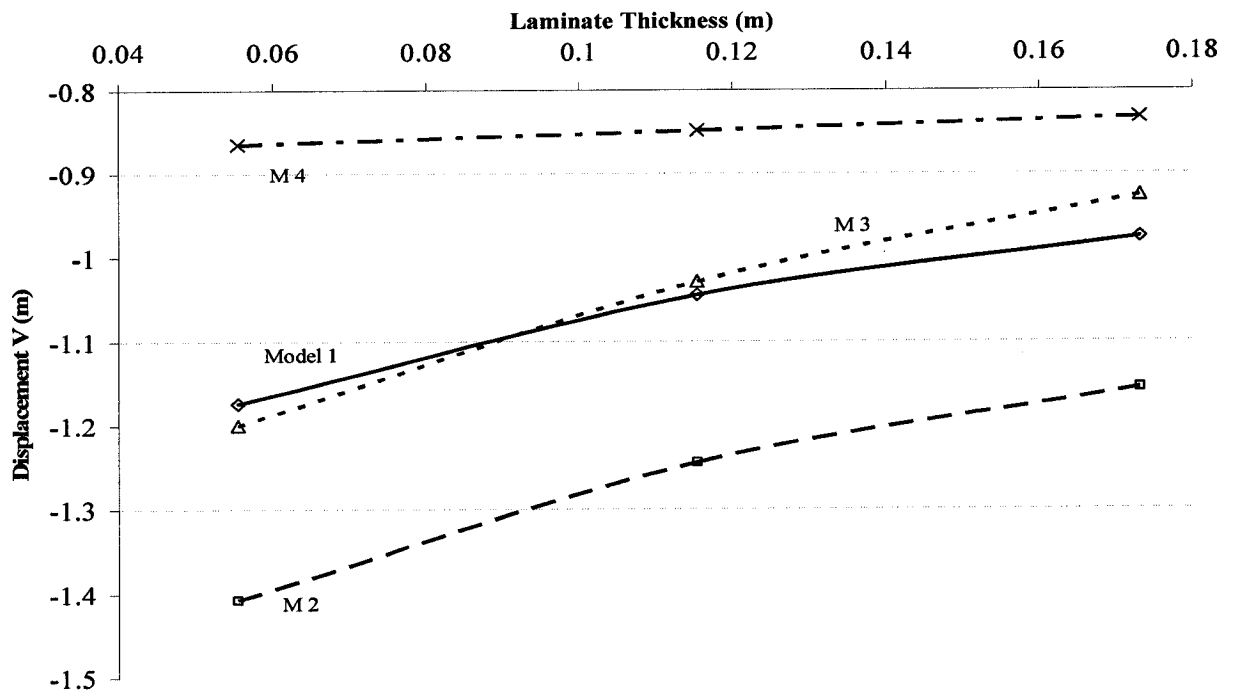


Figure 6.2.5.1.4 Laminate Thickness Effect on Lateral Displacement (V) – LDFA-a-CCR 0.100m - Annacis Island Bridge

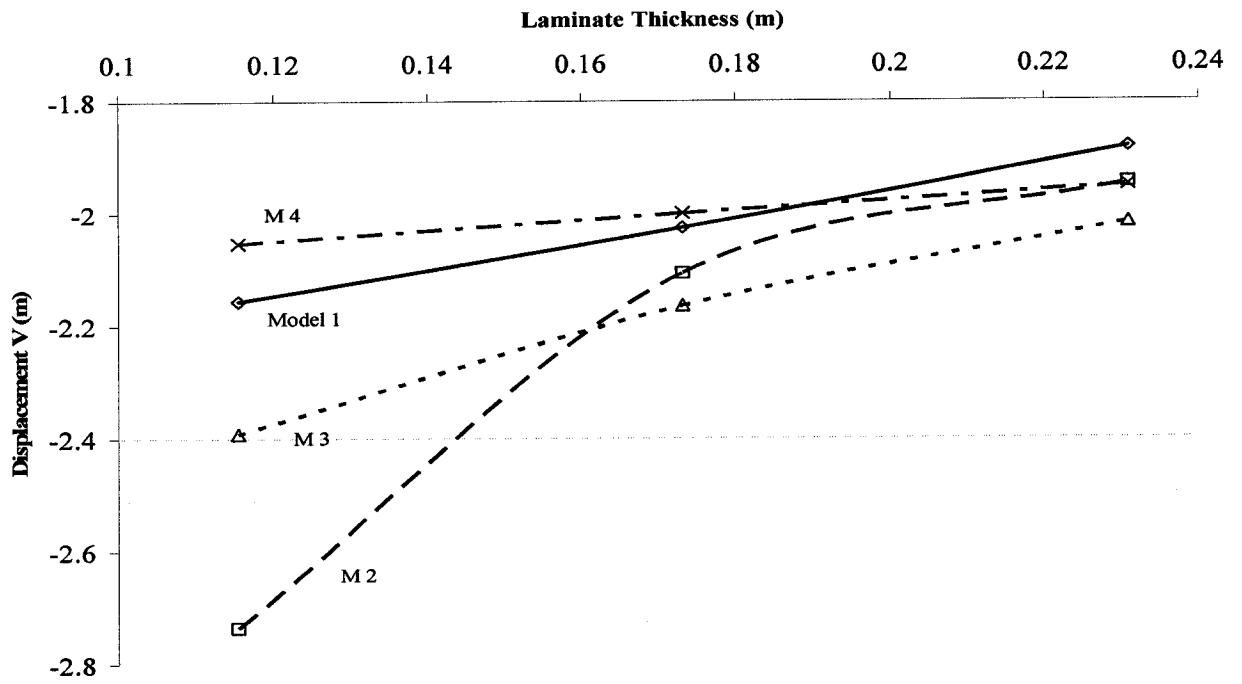


Figure 6.2.5.1.5 Laminate Thickness Effect on Vertical Displacement (V) – LDFA-a-LLa- CCR 0.150m – Tatara Bridge

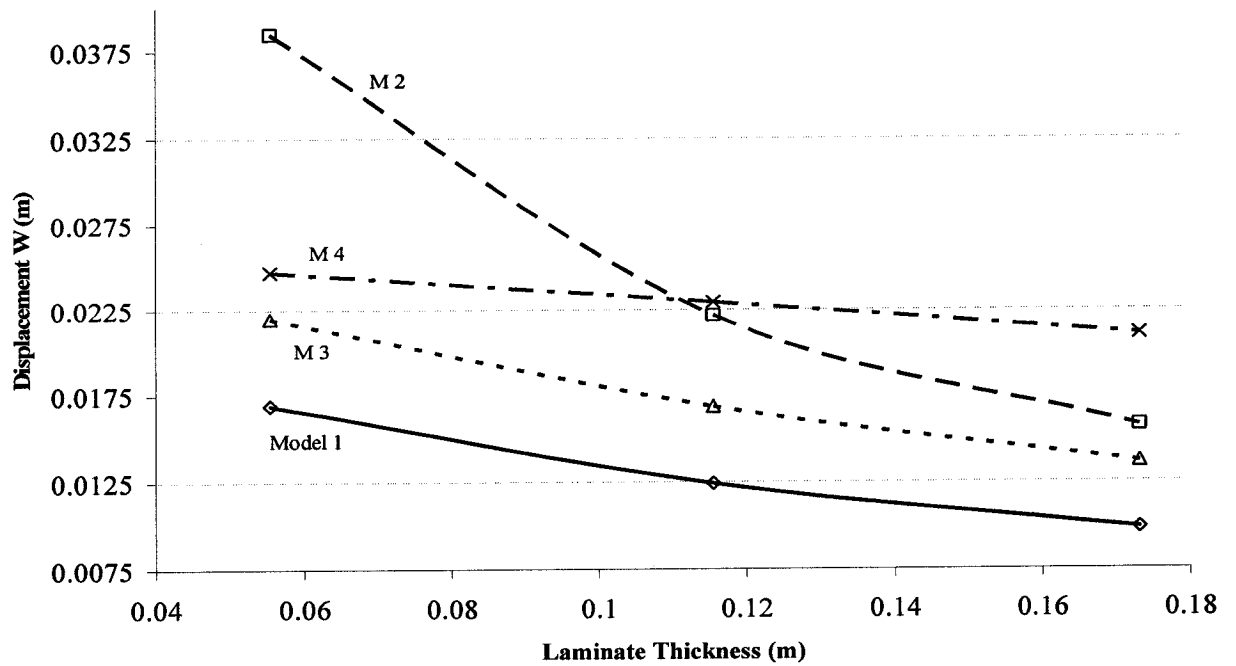


Figure 6.2.5.1.6 – Laminate Thickness Effect on Lateral Displacement (W) – LDFA-a-CCR 0.100m - Annacis Island Bridge

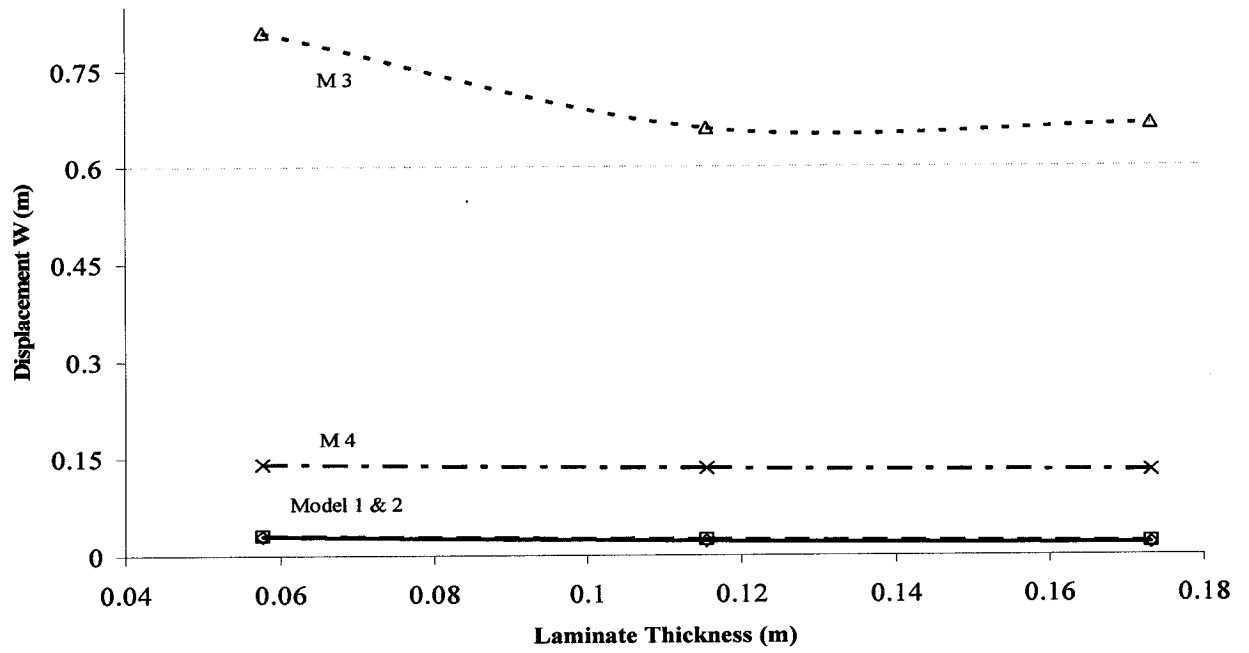


Figure 6.2.5.1.7 Laminate Thickness Effect on Longitudinal Displacement (W) – LDFA-a, LLa- CCR 0.125m – Second Chang River Bridge

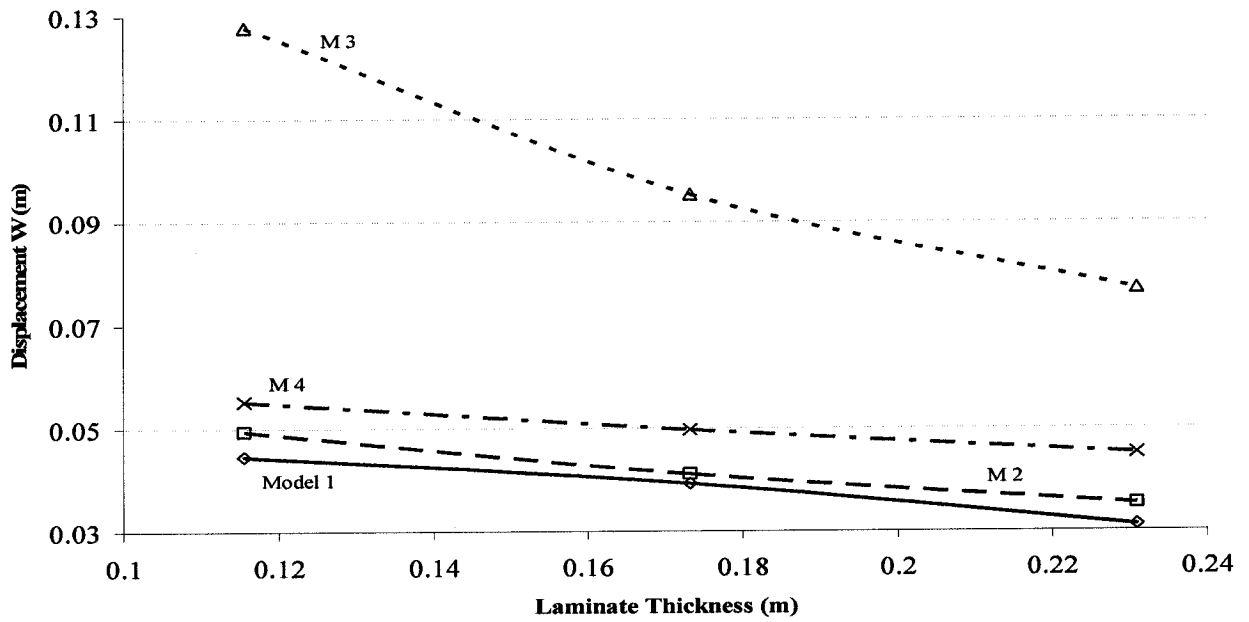


Figure 6.2.5.1.8 Laminate Thickness Effect on Longitudinal Displacement (W) – LDFA-a, LLa- CCR 0.150m – Tatara Bridge

#### **6.2.5.2 Effect of Laminate Thickness on the Maximum Tsai-Hill Failure Function of the Laminates, Principal Stresses of the Chopped FRP and Maximum Tensile Stresses of the Cable Stays:**

For a cable stayed bridge constructed from traditional materials, increasing the thicknesses of the box-girder-deck element, and hence the cross sectional area and the sectional moment of inertia, leads to decrease the stresses in the deck, provided that the bridge central span is short and the applied forces are constant [e.g. see the parametric study of Walther et al 1999]. Increasing the cross sectional area with medium and long bridge central spans would highly increase the deck self-loads, and hence, increases the axial compression stresses in the deck cross section at the pylon zones. For the proposed bridge models, the more complicated stress states of the laminated parts are investigated through the Tsai- Hill Failure Function THFF. It is basically combines the ratios between the different stresses to their related material strengths. However, the results indicate that the leading stresses in the deck are still the axial stresses, which are related mainly to the cable stayed bridge system arrangement and the applied loads.

The major difference between traditional bridge materials and the advanced composite materials is that the stiffness and strength to the weight ratios of the composite material are high. Hence, increasing the deck laminates thickness leads to improve the deck sectional properties, and hence the deck stiffnesses, with much lower additions to the deck self loads. The THFF results can then be clarified by the balance between: (i) the reduction of the stresses due to the increase of laminate thickness  $L_{Th}$  and hence the cross sectional area, (ii) the increase of the self loads of the deck due to the increase of  $L_{Th}$ , which leads to increase the stresses in the pylon zones as the bridge central span increases, (iii) the increase in the deck stiffness that leads to redistribute the stresses in the deck and the stay cables.

It has been found that the Tsai Hill Failure Functions (THFF) of all laminates are generally reduced as the Laminate Thickness ( $L_{Th}$ ) increased for most of the examined cases with some exceptions (Figures 6.2.5.2.1 to 7 show samples of the typical results). The rate of change is decreased as the laminate thickness increased, indicating to an economical laminate thickness as in the past section. The two major exceptions are; (i) the vertical laminates of Model II, which are basically critical, give fluctuated THFF

values for medium spans (Figures 6.2.5.2.6), and (ii) the bottom laminate of model III, more specifically the rounded parts of the bottom laminate, with the shortest span length span (Figure 6.2.5.2.2). In both cases, the failure function decreased nonlinearly and very rapidly as the laminate thickness increased to the medium examined level, then their function values rose back again at very high rates.

The general reduction of THFF with the increase of the laminate thickness is explained by the reduction of the stresses due to the increase in the sectional area. The apparent reduced rate of change in the general THFF-LTh relation as the bridge central span increases can be explained by the increase of the deck self-weight that increases the deck stresses in the pylon zones. The vertical laminates of Model II, for the Second Chang River Bridge, show a sudden reduction in the THFF as LTh increases to its medium level. This indicates two major changes; (i) the disappearance of the compression stress field due to the increase of the deck stiffness and the resulting redistribution of the stresses, (ii) the increase of the laminate sectional area, which basically reduces the stresses. With further increase of laminate thickness, the effect of the weight becomes more active and the critical balance of the vertical laminates of Model II section sides leads to switch the stresses back to compression. On the other hand, Model III's bottom laminate has similar critical regions where the axial stress field is variable and can easily switch from tension to compression and vice versa, according to the critical balance between the mentioned variables. This explains the occasional and sudden huge rise in THFF of the bottom laminate of Model III with the very high laminate thickness.

The extreme (maximum or minimum) principal stresses of the chopped FRP deck slab of Models I, II and III and the chopped FRP main part of Model IV are decreased slightly by the increase of the laminate thickness. Figure 6.2.5.2.8 shows sample results of this decrease. This material has a low modulus of elasticity. The increase of the laminate thickness, and the resulting redistribution of stresses in the system, lead to change the stress distribution in this part of the section. The increasing deck stiffness reduces the high strain zones in the top surface of the deck, and this is transferred through the low modulus of the material, to a slight reduction of the extreme stresses.

Similarly, the maximum cable tension is reduced due to the redistribution of the stay tensile forces (Figure 6.2.5.2.9). The increase of the deck stiffness leads to increase the tensile stresses in more cables close to the bridge center and ends. A high percentage of

the tensile force is now distributed over more long cables, far from the pylons, however the cable tension is not apparently changed in the short cables close to the pylon. This redistribution leads to reduce the maximum level but stresses more long cables. The reduction is less with Model IV as the laminated part of the section is small in comparison with other models.

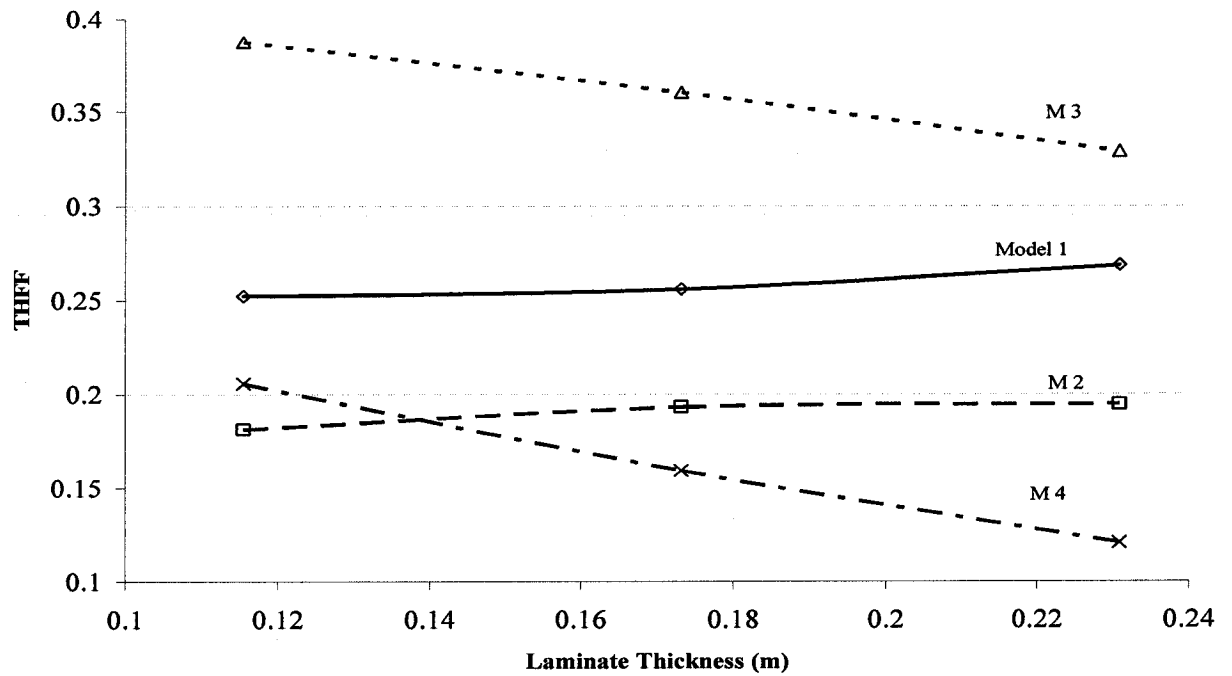


Figure 6.2.5.2.1 Laminate Thickness Effect on THFF – Laminate B- LDFA-a, LLa, CCR  
0.150m - Tatara Bridge

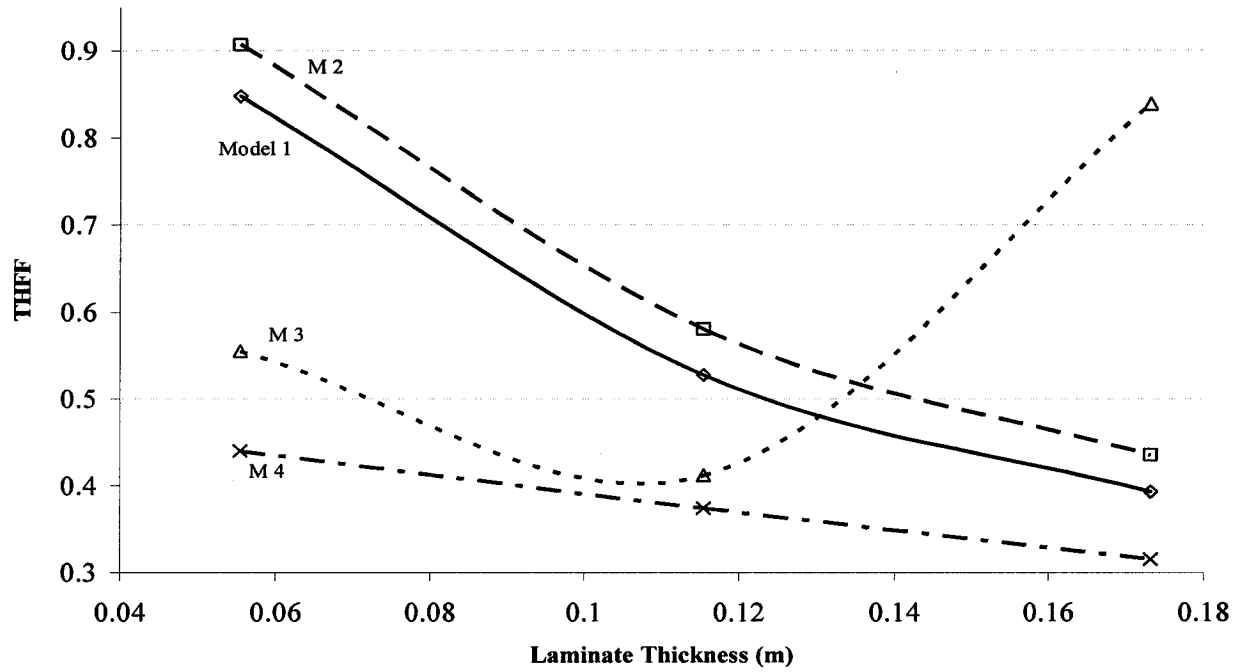


Figure 6.2.5.2.2 Laminate Thickness Effect on THFF – Laminate C – LDFA-a, CCR  
0.100m -Annacis Island Bridge

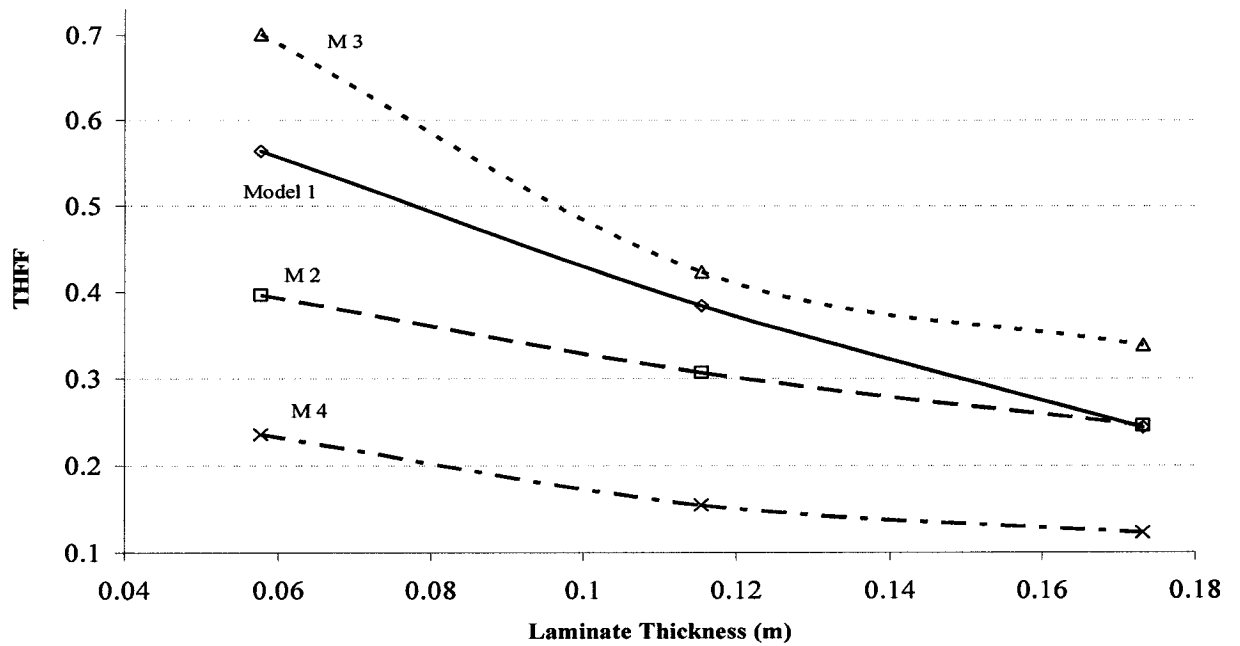


Figure 6.2.5.2.3 Laminate Thickness Effect on THFF – Laminate C- LDFA-a, LLa, CCR  
0.125m - Second Chang River Bridge



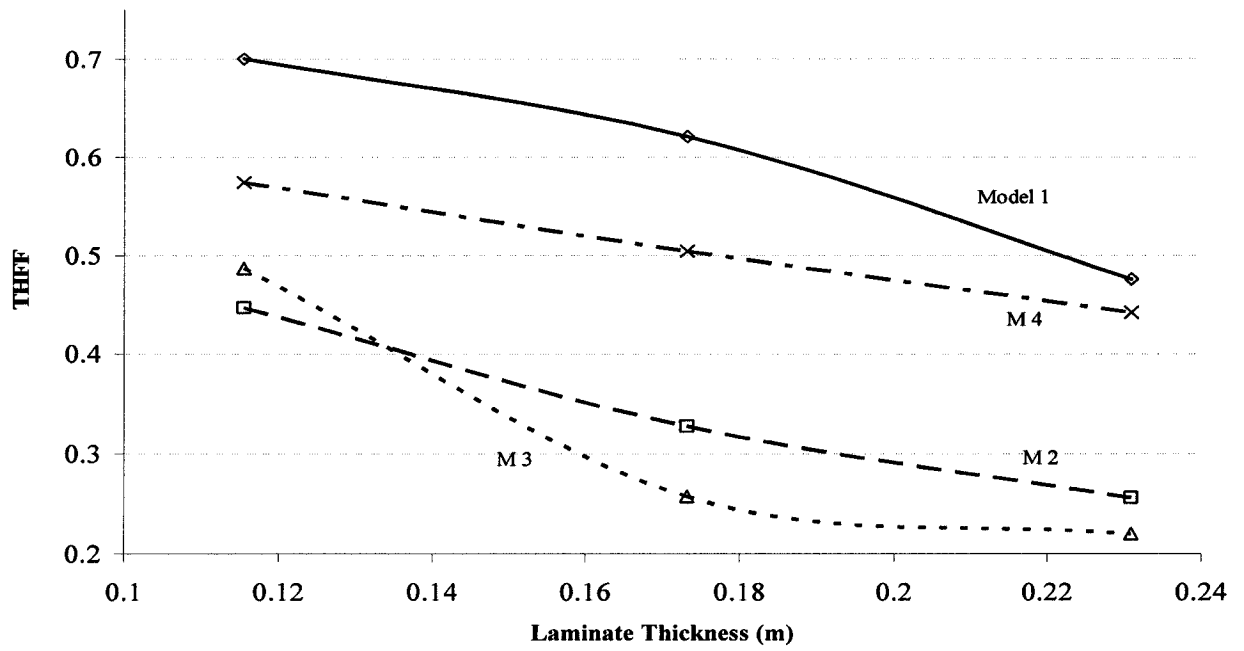


Figure 6.2.5.2.4 Laminate Thickness Effect on THFF, Laminate C, LDFA-a, LLa, CCR  
0.150m - Tatara Bridge

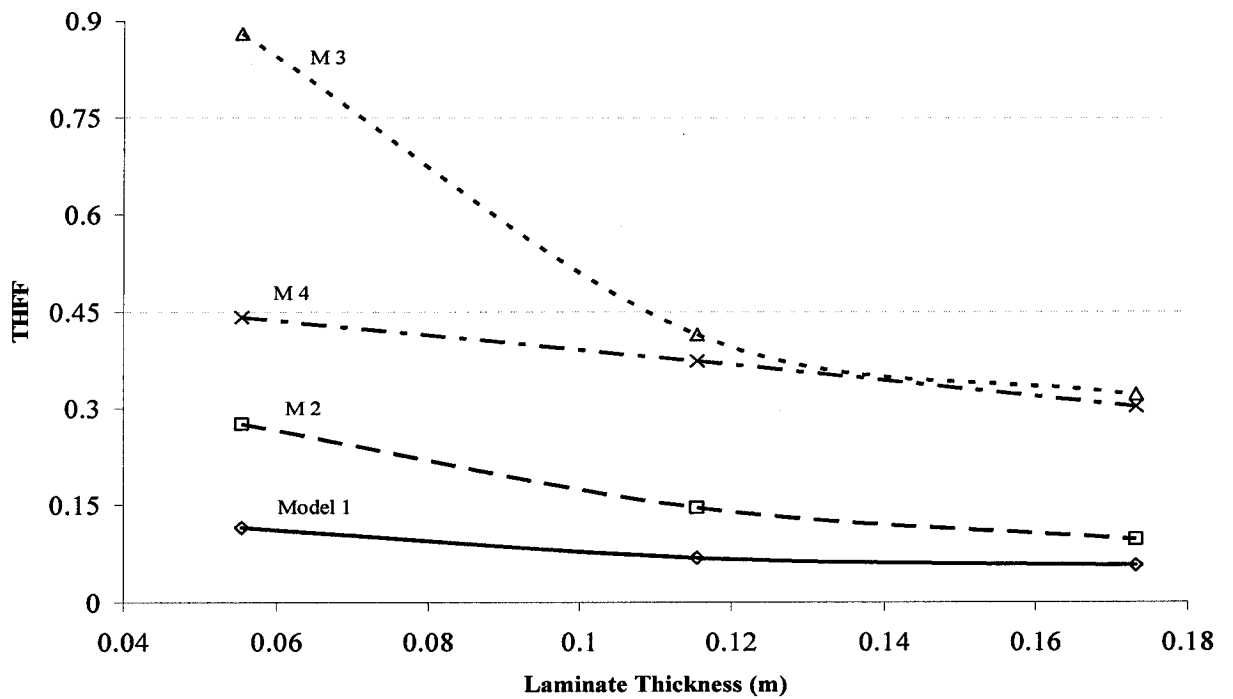


Figure 6.2.5.2.5 Laminate Thickness Effect on THFF, Laminate D, LDFA-a, CCR  
0.100m - Annacis Island Bridge

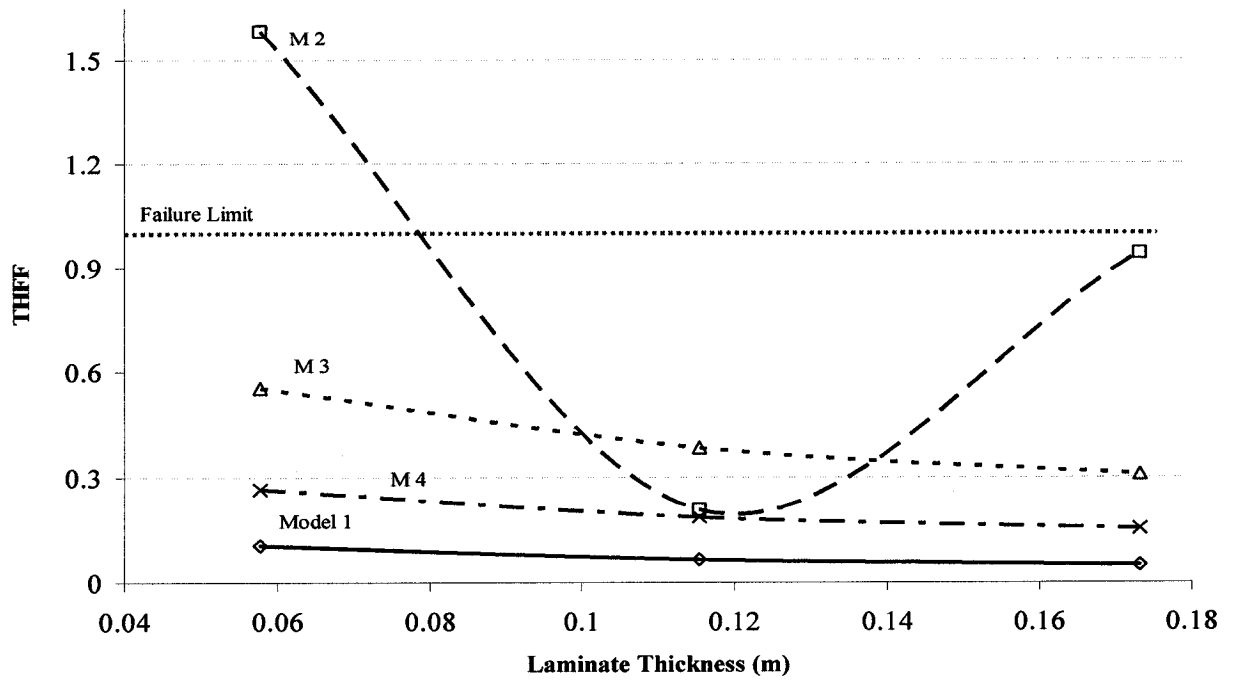


Figure 6.2.5.2.6 Laminate Thickness Effect on THFF – Laminate D, LDFA-a, LLa, CCR  
0.125m - Second Chang River Bridge

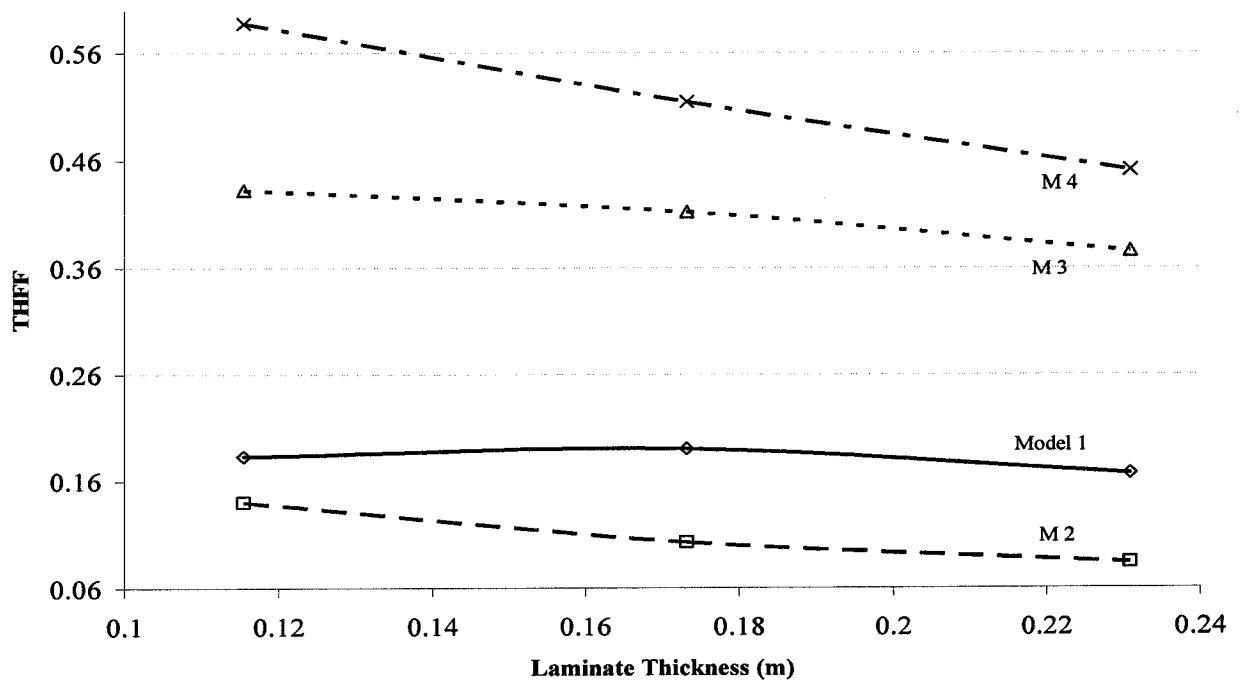


Figure 6.2.5.2.7 Laminate Thickness Effect on THFF – Laminate D, LDFA-a, LLa, CCR  
0.150m - Tatara Bridge Bridge

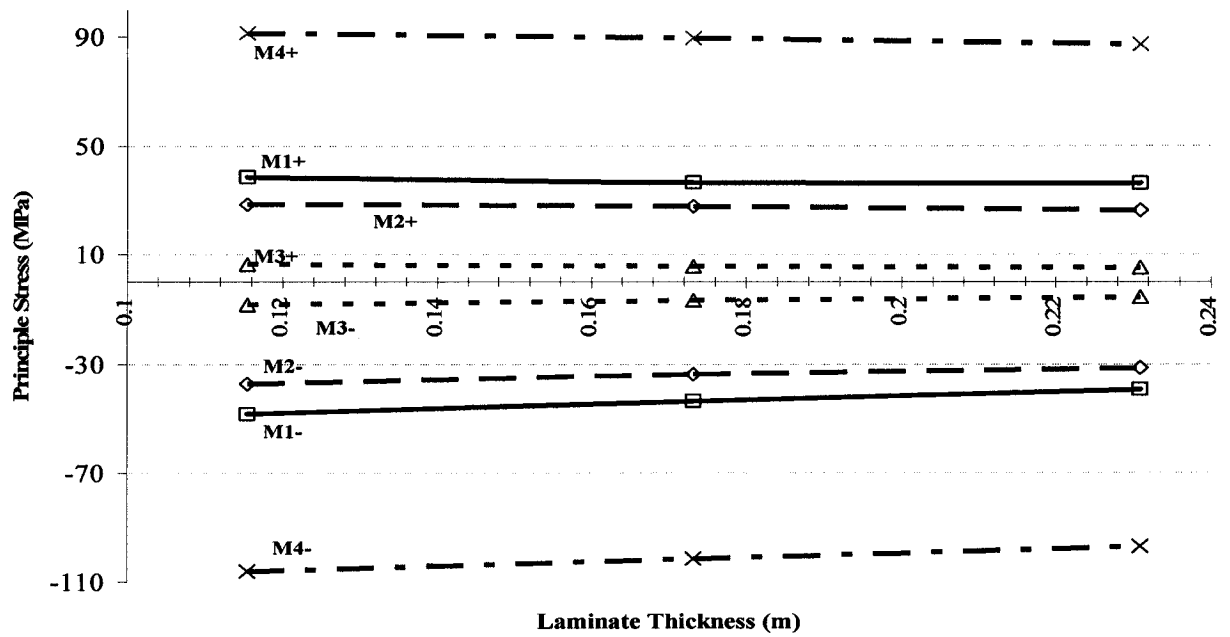


Figure 6.2.5.2.8 Laminate Thickness Effect on Principal Stresses at the Chopped FRP Deck Slab for Models I, II and III and the deck section for Model IV- LDFA-a, CCR 0.150m - Tatara Bridge

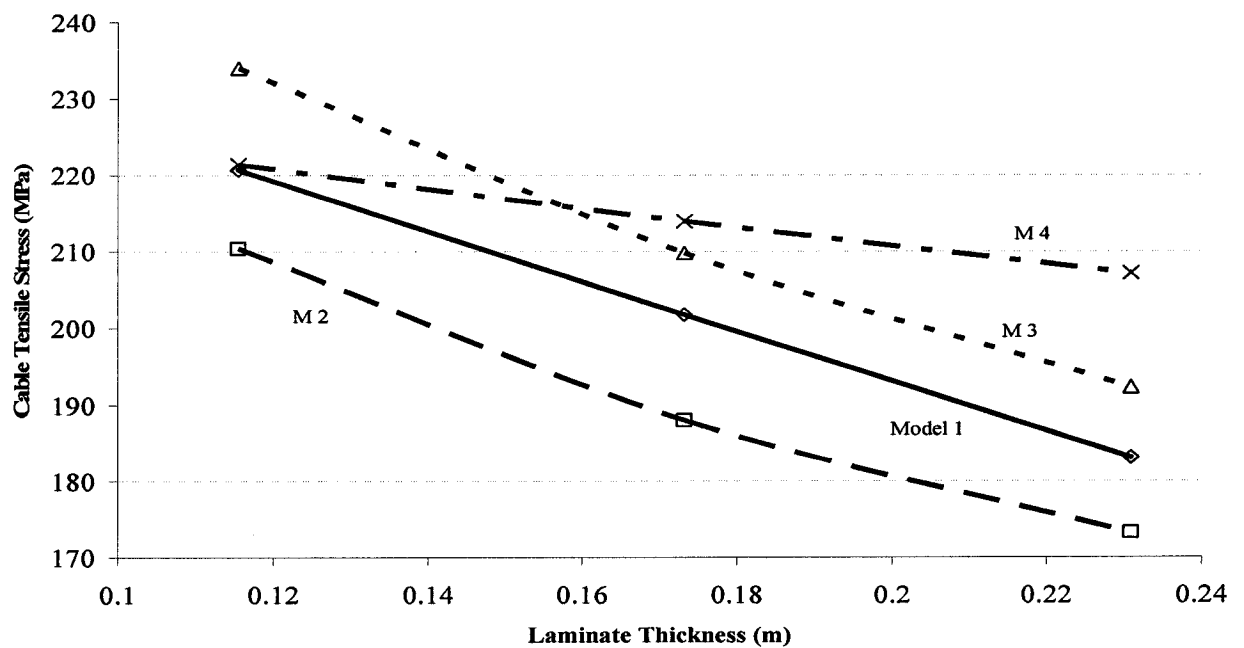


Figure 6.2.5.2.9 Laminate Thickness Effect on Maximum Cable Tension for different models, LDFA-a, CCR 0.150m - Tatara Bridge

### **6.2.5.3 Effect of Laminate Thickness on the Vertical Natural Frequencies:**

Generally it is apparent that the increase in the laminate thickness reduces the vertical natural frequencies (VNF) with some exceptions where the natural frequencies may increased then decrease (e.g third VNF of Model III of the Annacis Island Bridge in Figure 6.2.5.3.5) or decrease then rise again (e.g. fifth VNF of Model II of the Tatara Bridge in Figure 6.2.5.3.12) and for more rare cases it could slightly rise (fifth VNF of Model I of the Annacis Island Bridge in Figure 6.2.5.3.10). Whatever those exceptional behaviors are, the end frequencies related to the high laminate thicknesses are mostly less than the start frequencies related to the low laminate thicknesses. The fluctuation in the frequency could be as high as 40% but the difference between the frequencies for the lowest and highest examined laminate thickness ranges between 3% and 20%. Figures 6.2.5.3.1 to 6.2.5.3.12 show sample results of the laminate thickness effect on the VNF. Results similar to Figure 6.2.5.3.2 or 6.2.5.3.3, not shown here, where a successive reduction of VNF is observed.

Similar to the traditional structure, the balance between the vertical flexural stiffness of the deck and its mass contributions to the natural frequencies clearly explains the relation between the vertical natural frequencies and the laminate thickness. Usually, advanced composites have high stiffness to weight ratio in comparison with the traditional materials and the increase of the laminate thickness is expected to increase the natural frequencies. This is apparent with the short span bridge as shown in Figure 6.2.1.6; however, the rate of increase in the frequencies is reduced as the laminate thickness increases. Nevertheless, the material used for the lamina (E-Glass fiber and Vinylester Matrix GFVM) has the lowest stiffness-to-weight ratio among other advanced composite combinations (such as Carbon fibers, with either Glass Epoxy or Kevlar® Epoxy-Matrix ...etc). In addition, the examined Hybrid Long Span Cable Stayed System has extremely long central span ranged between 450m and 900m. With such huge spans and with the increase of the laminate thickness, the effect of the mass increase is surplus the increase of the system vertical flexural stiffness. Indeed, the huge length of the bridge system highly reduces the deck stiffness and the stiffness increase becomes less effective on the vertical flexural natural frequencies than the mass increase.

Hence the general reduction of VNF of most of the cases can be related to the effect of the increase in the deck mass at a higher rate than the increase of the vertical flexural stiffness, which is more apparent with the medium and long central spans. Indeed, increasing the laminate thickness increases the deck vertical flexural stiffness, increases greatly the deck lateral and torsional stiffnesses, and increases the mass at relatively higher rates with the medium and the longest central spans. The only exception for the first VNF is that of Model II where the effect of increasing the stiffness is surplus to the effect of the mass (Figure 6.2.5.3.1). This case is understandable in the light of the critical relation between the stiffness and the mass for Model II. For higher natural frequencies, (fourth and fifth) Model II shows higher sensitivity to the balance between the stiffness and the mass, while it is shown increasing then decreasing with medium span lengths, and it is shown decreasing then increasing with the very long central span lengths (Figures 6.2.5.3.7, 8 and 12). Model III's stiffness and mass balance is also sensitive but with the low and medium bridge central span lengths, rather than the longest central span where the mass becomes very high. Third, fourth and fifth VNF of Model III of Annacis Island rises then decreases (Figures 6.2.5.3.5, 7 and 10), however the fifth VNF of the second Chang River Bridge slightly decreases then raises.

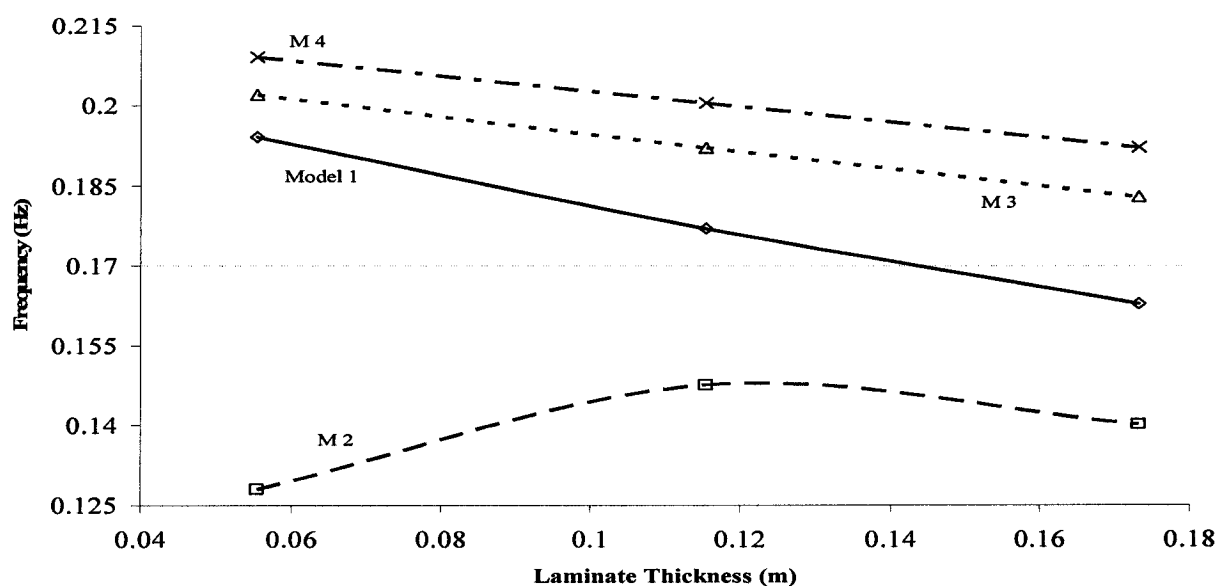


Figure 6.2.5.3.1 Laminate Thickness Effect on the 1st Vertical Natural Frequencies—LDFA-a, CCR 0.100m - Annacis Island Bridge

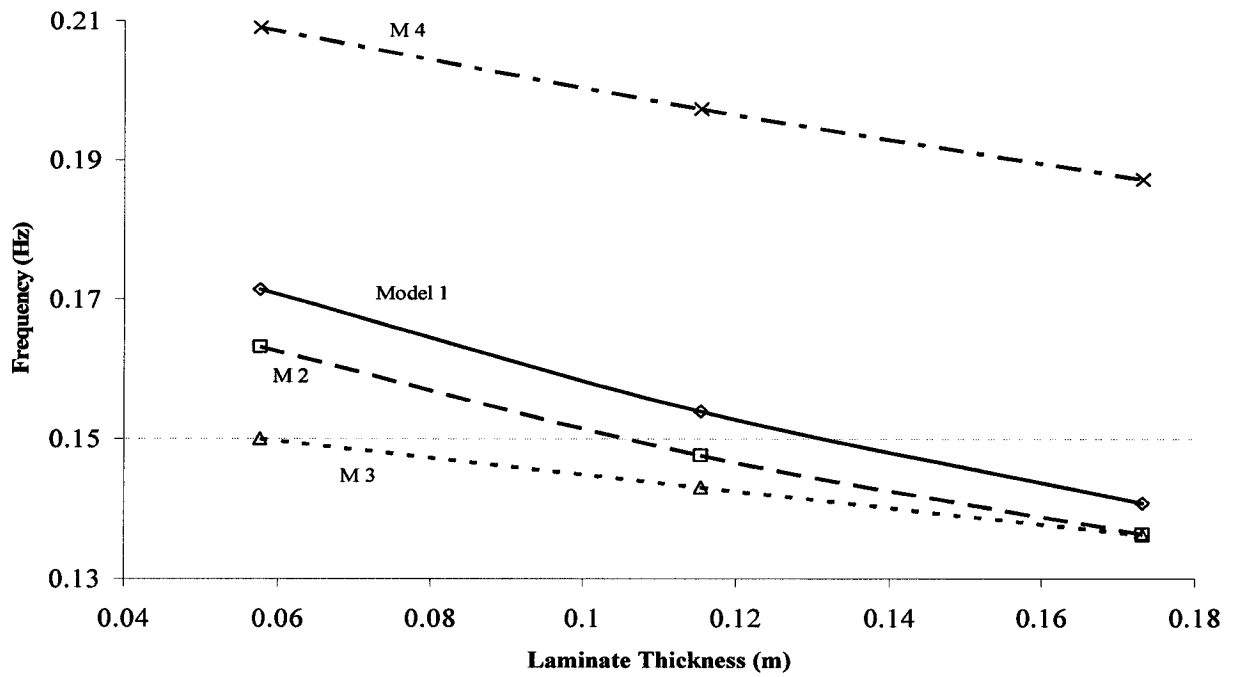


Figure 6.2.5.3.2 Laminate Thickness Effect on the 1st Vertical Natural Frequencies—LDFA-a, CCR 0.125m - Second Chang River Bridge

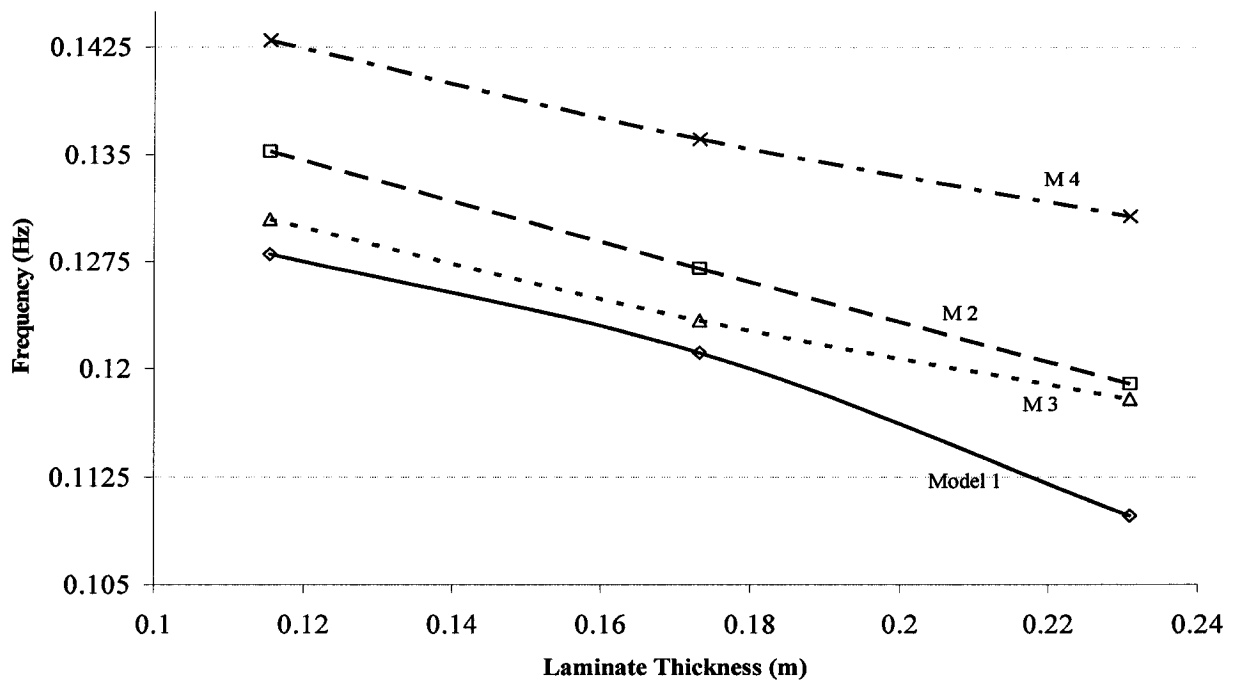


Figure 6.2.5.3.3 Laminate Thickness Effect on the 1st Vertical Natural Frequencies—LDFA-a, CCR 0.150m- Tatara Bridge

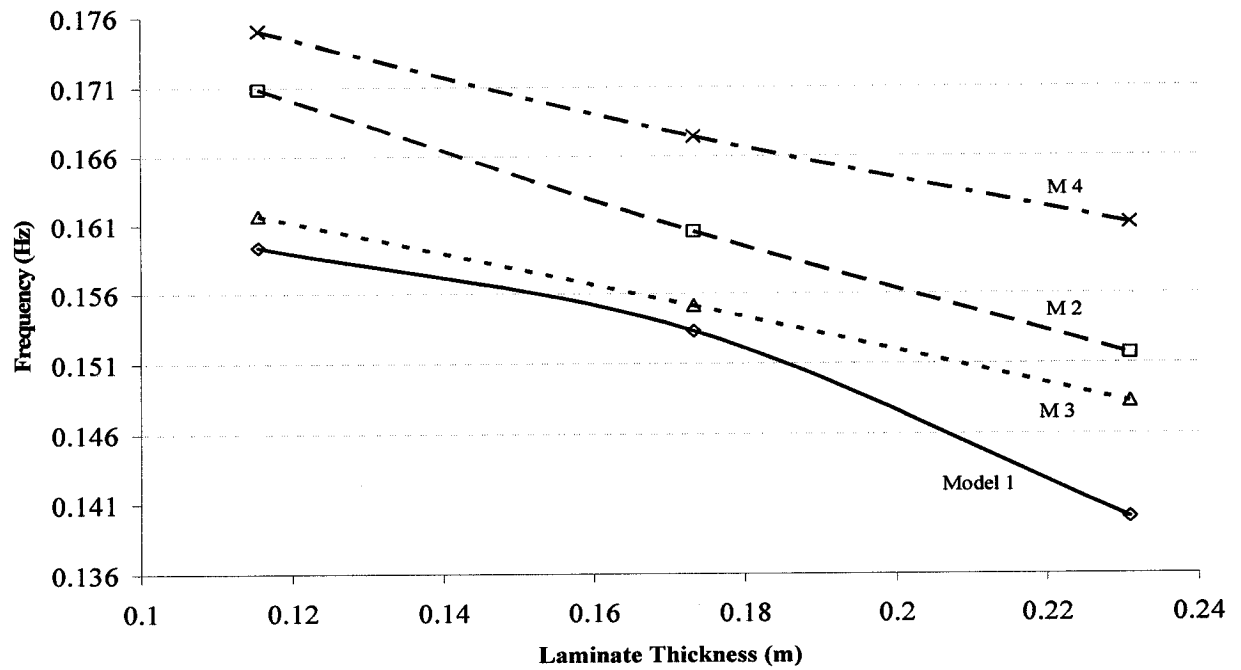


Figure 6.2.5.3.4 Laminate Thickness Effect on the 2nd Vertical Natural Frequencies- LDFA-a, CCR 0.150m- Tataru Bridge

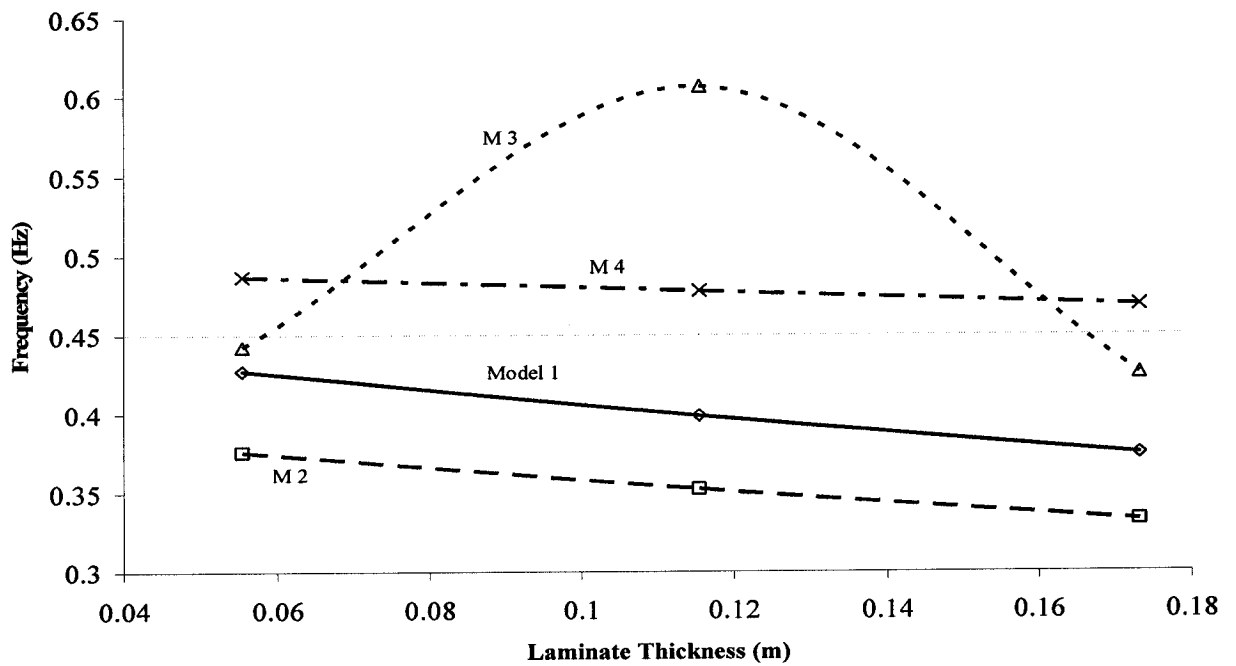


Figure 6.2.5.3.5 Laminate Thickness Effect on the 3<sup>rd</sup> Vertical Natural Frequencies- LDFA-a, CCR 100m- Annacis Island Bridge

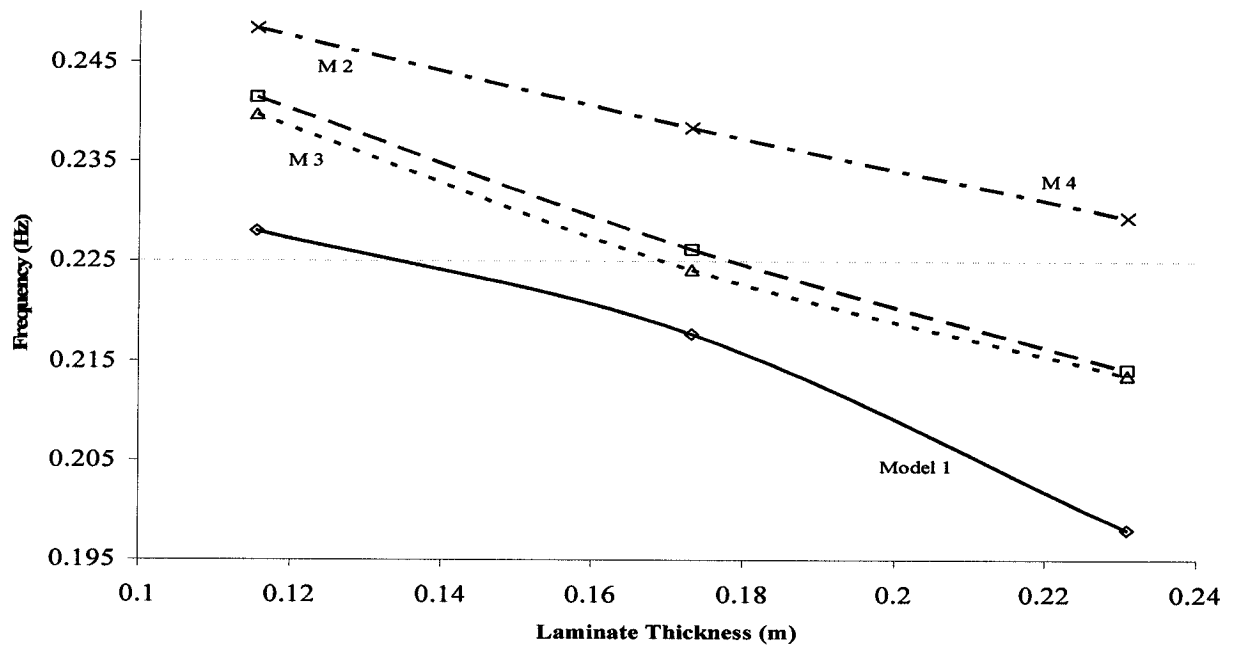


Figure 6.2.5.3.6 Laminate Thickness Effect on the 3<sup>rd</sup> Vertical Natural Frequencies- LDFA-a, CCR 0.150m- Tatara Bridge

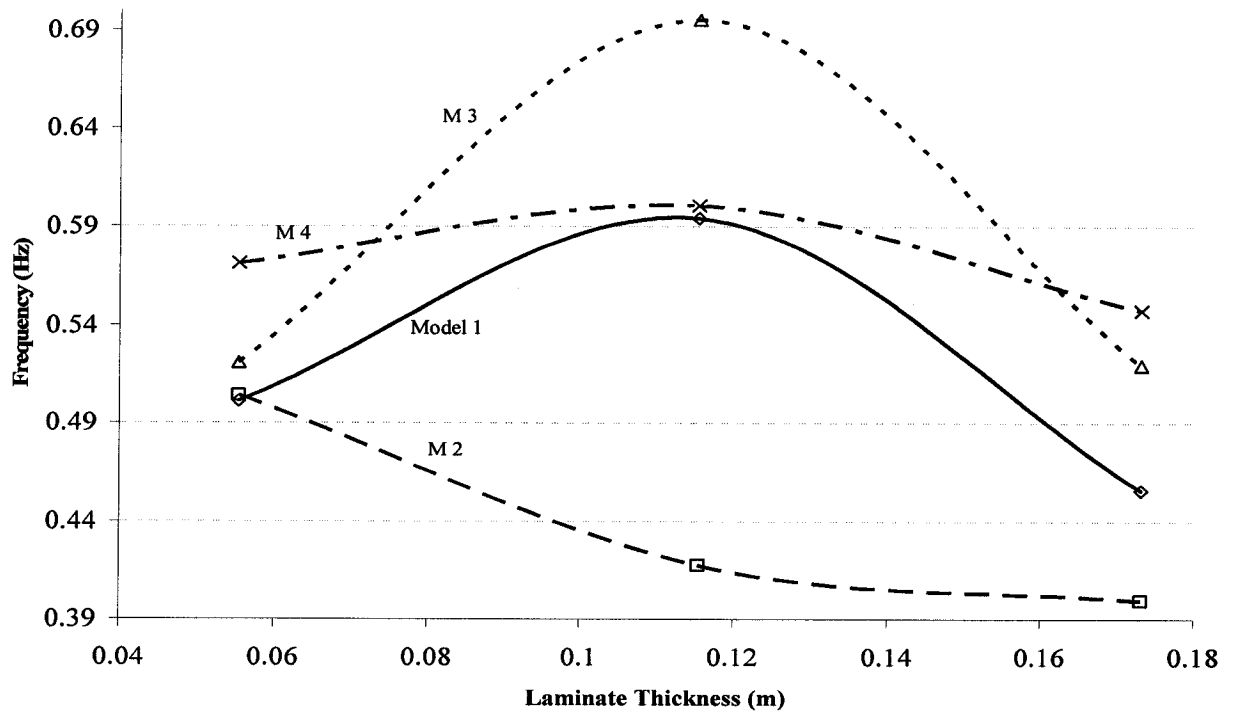


Figure 6.2.5.3.7 Laminate Thickness Effect on the 4<sup>th</sup> Vertical Natural Frequencies- LDFA-a, CCR 0.100m- Annacis Island Bridge



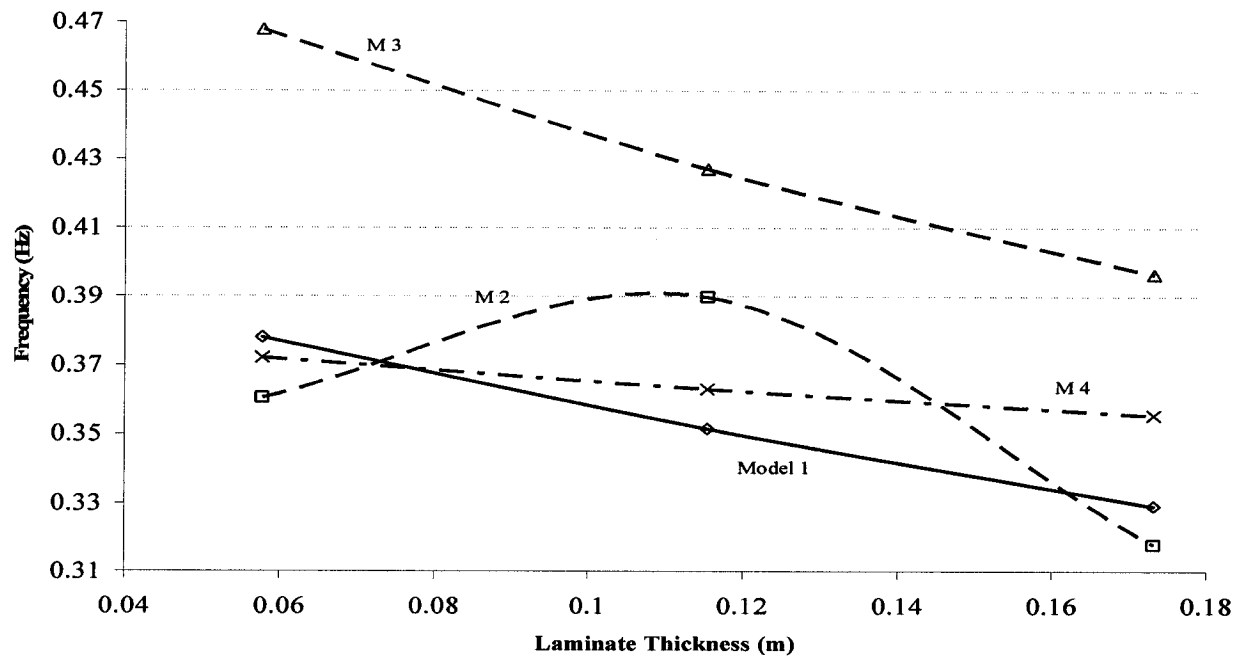


Figure 6.2.5.3.8 Laminate Thickness Effect on the 4<sup>th</sup> Vertical Natural Frequencies-LDFA-a, CCR 0.125m - Second Chang River Bridge

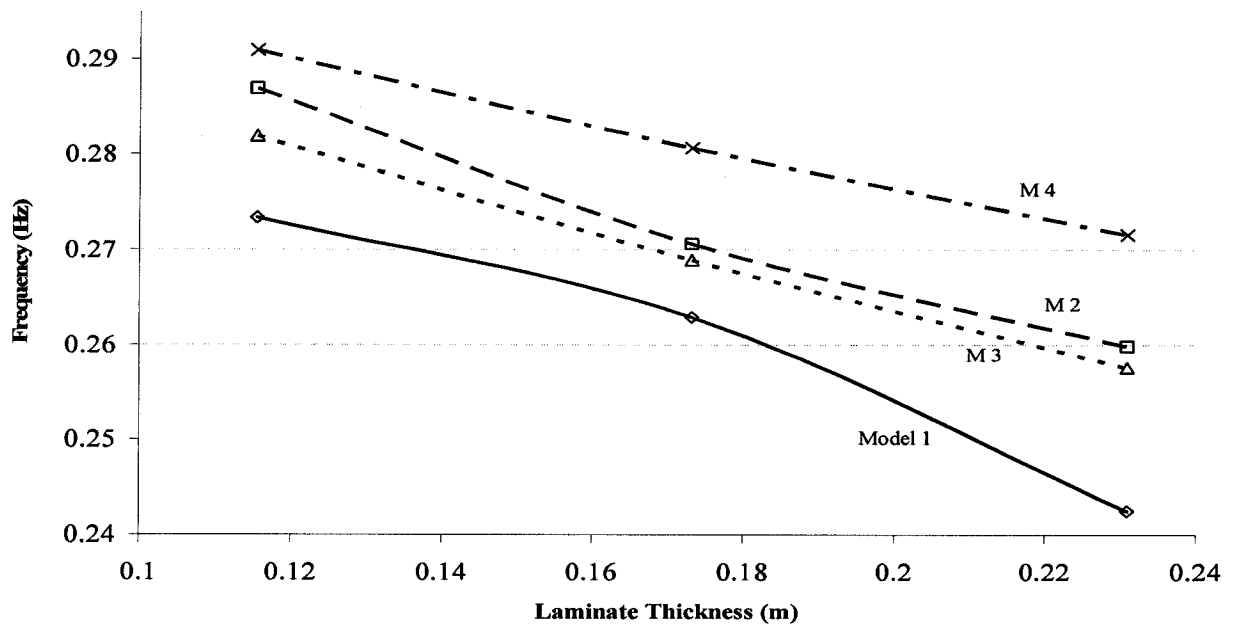


Figure 6.2.5.3.9 Laminate Thickness Effect on the 4<sup>th</sup> Vertical Natural Frequencies-LDFA-a, CCR 150m- Tatara Bridge

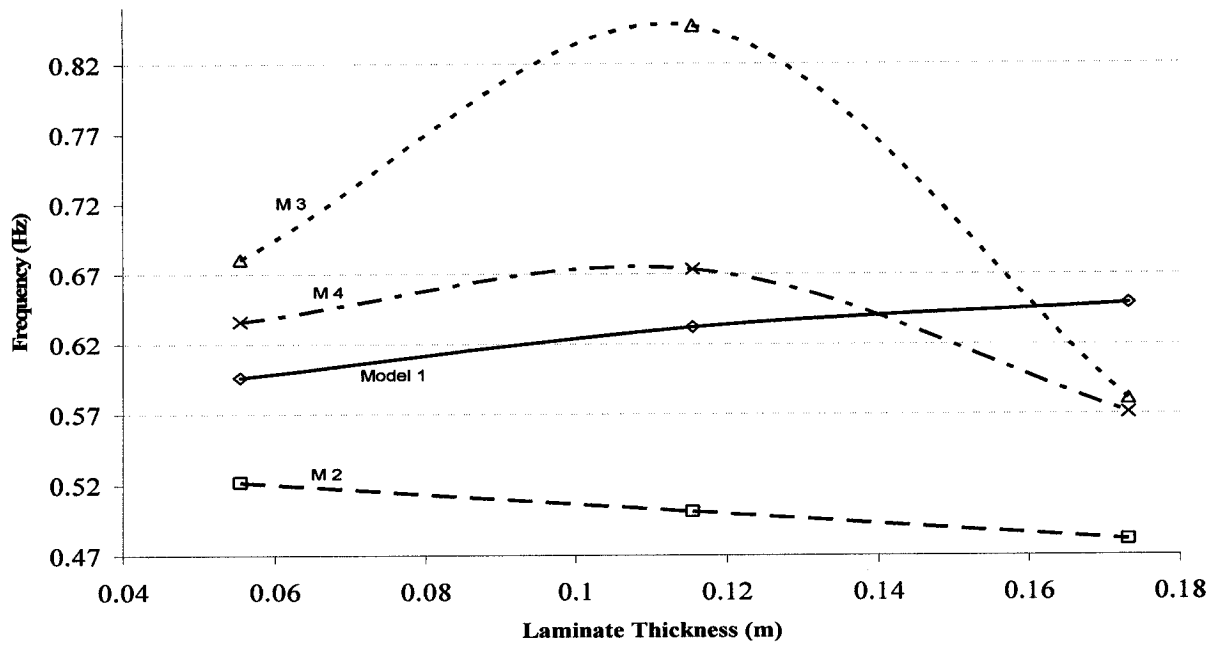


Figure 6.2.5.3.10 Laminate Thickness Effect on the 5<sup>th</sup> Vertical Natural Frequencies-LDFA-a, CCR 0.100m- Annacis Island Bridge

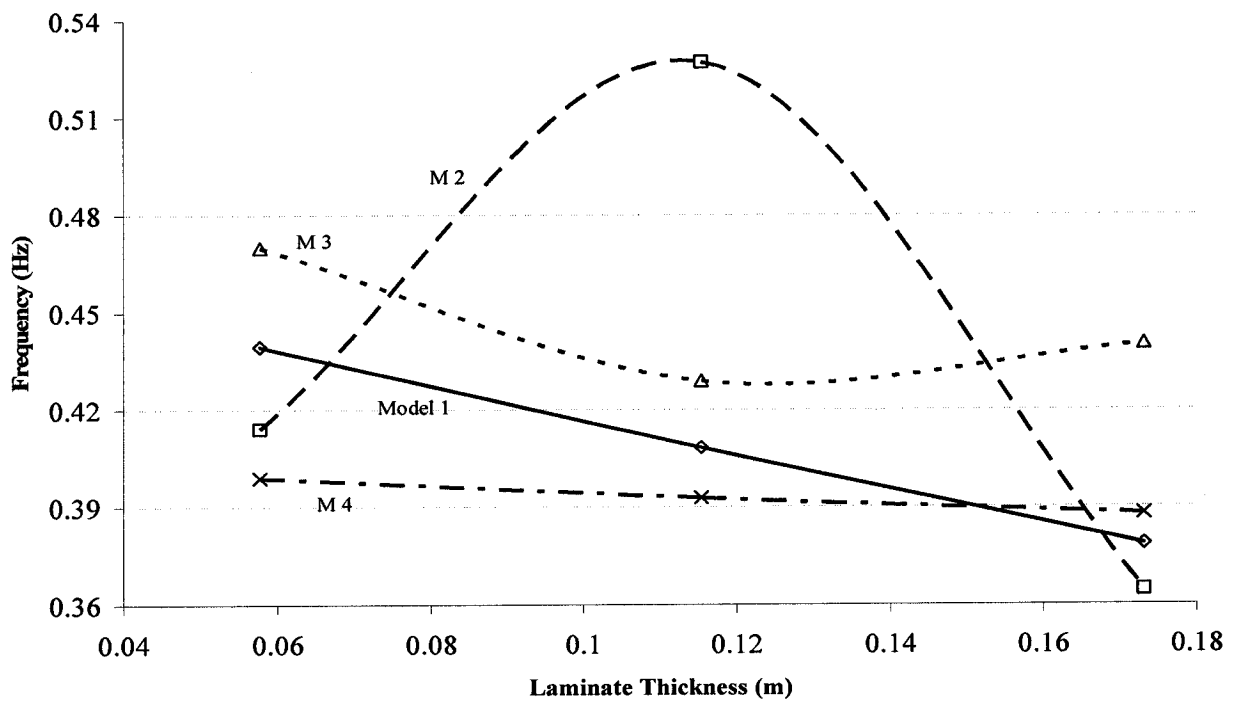


Figure 6.2.5.3.11 Laminate Thickness Effect on the 5<sup>th</sup> Vertical Natural Frequencies-LDFA-a, CCR 0.125m- Second Chang River Bridge

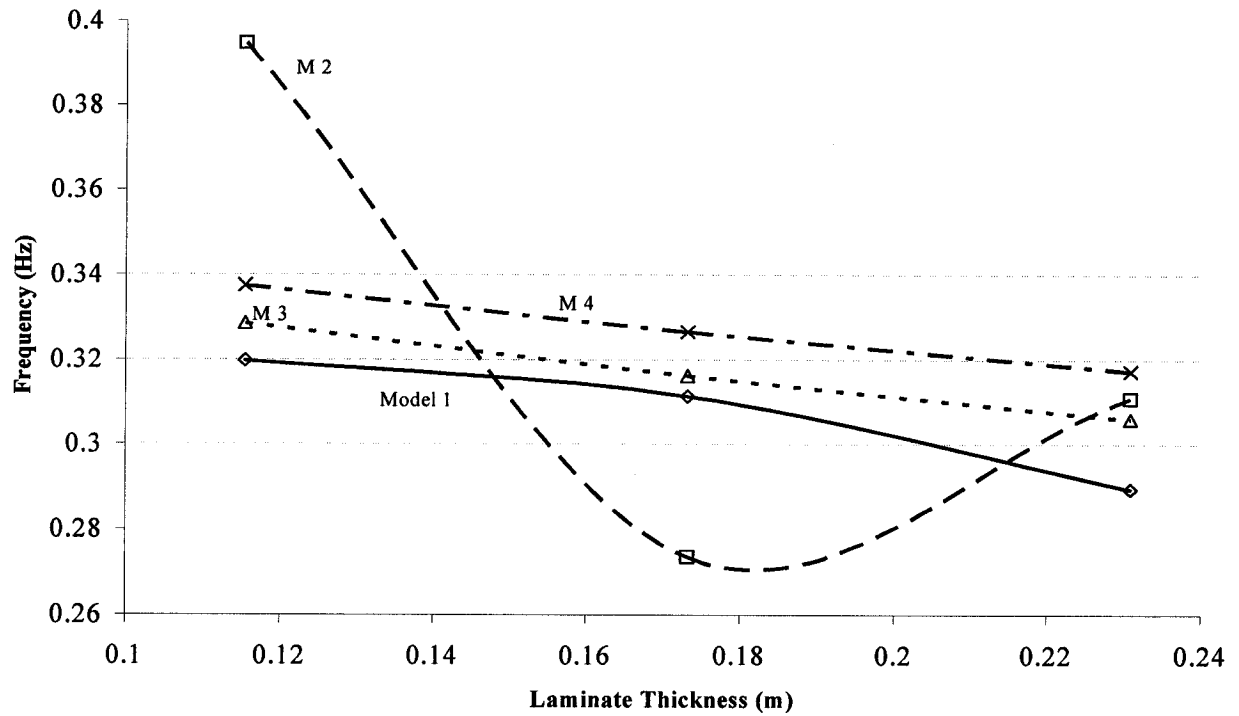


Figure 6.2.5.3.12 Laminate Thickness Effect on the 5<sup>th</sup> Vertical Natural Frequencies-LDFA-a, CCR 0.150m - Tatara Bridge

#### 6.2.5.4 Effect of Laminate Thickness on the Torsional Natural Frequencies:

A dominant increase in the Torsional Natural Frequencies (TNF) is related to the increase in the laminate thickness with some exceptions, where they may increase then decrease or decrease then rise back, and for more rare situations they could slightly drop. This is more apparent with the second TNF and higher frequencies. The increase with Model III could be as high as 75% for some torsional natural frequencies. It is also apparent that the change of TNF due to the increase of the laminate thickness is highly reduced with increase in the bridge central span length. Figures 6.2.5.4.1 to 15 show the variation of the first five torsional natural frequencies of different models and different bridges. The micro/macro structure of the materials is constant and the cable diameter is also constant during the laminate thickness increase.

Again the balance between the torsional stiffness of the deck and its mass explain the results here. Increasing the laminate thickness greatly increases the torsional stiffness but also it increases the deck mass. As the bridge central span increases, the effect of the

mass becomes significant and the balance between the stiffness and the mass contributions to the torsional natural frequencies becomes critical. The increase in the deck torsional stiffnesses of the three models, Model I, II and III (Model IV has very high TNF and it is not shown in the present section), due to the increase of the laminate thickness, exceeds on the increase in deck masses. The deck sectional geometrical shape plays a huge rule in the increase of the torsional stiffness, sectional area and deck mass levels with the increase of the laminate thickness. This explains the general increase in the torsional natural frequencies in most of the cases, which is more apparent with Model III (Figures 6.2.5.4.1 and 4 to 15). The Model III torsional stiffness is relatively high and it increases greatly with the increase of the laminate thickness. The deck mass of this model is also high and its contribution to the TNF becomes more effective as the bridge central span length increases. On the other hand, Model II basically has a low torsional stiffness to mass ratio. Consequently the results shows that the TNF of Model II is sensitive, and balance between the torsional stiffness and mass is critical and reverses the increase of the frequency to a decrease as the laminate thickness increases, with the shortest and medium bridge central span lengths (Figures 6.2.5.4.1, 4, 5, 7, 8, 10, 11, 13, 14). However with the longest bridge span length, the low mass of Model II becomes very effective in stabilizing the TNF, giving very slight increase or very slight decrease of TNF as the laminate thickness increases. Model I has less torsional stiffness than Model III and a slightly higher mass. Its torsional stiffness contribution to the TNF is balanced or slightly overcome by its mass contribution for the short and medium bridge span length, resulting in very low fluctuation of the TNF, and an increase in the high natural frequencies of the shortest span (Figures 6.2.5.4.1, 2, 4,5, 8, 10, 11, 13,14). With the longest span length, the Model I torsional stiffness contribution to the TNF slightly overcomes the contribution of the deck mass when the laminate thickness increases to the medium examined level. However with further increase of the laminate thickness, the relation is reversed and the mass contribution reduces the TNF (Figures 6.2.5.4.3, 6, 9, 12, 15).

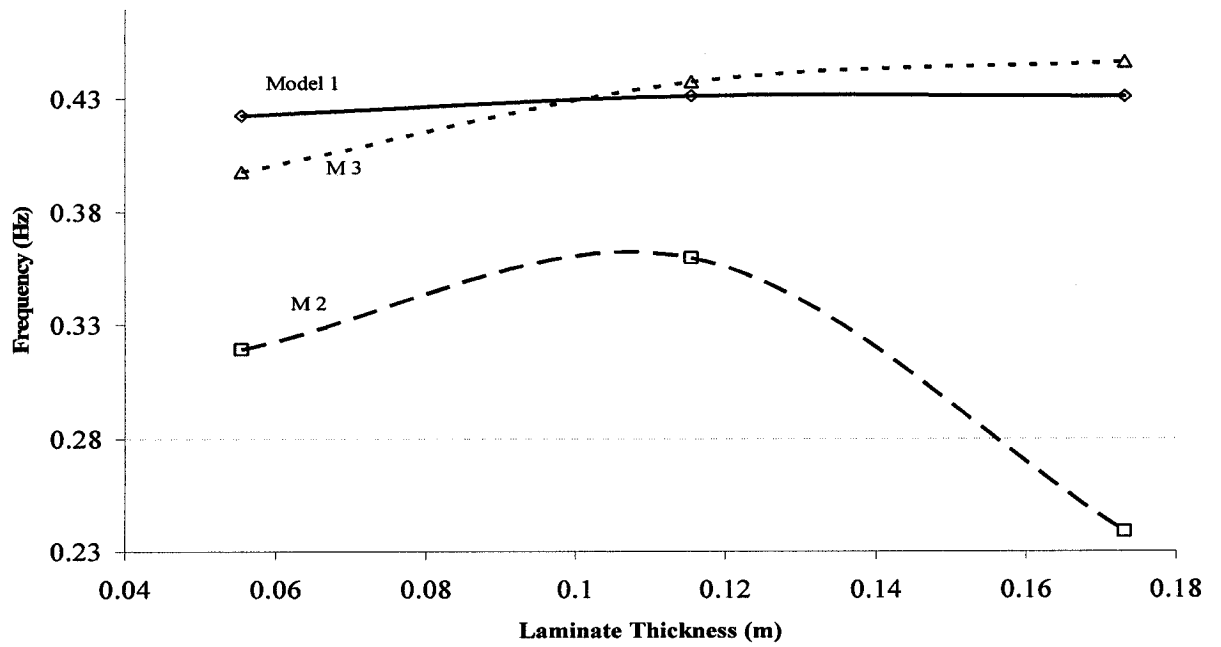


Figure 6.2.5.4.1 Laminated Thickness Effect on the 1st Torsional Natural Frequencies, LDFA-a, CCR 0.100m, Annacis Island Bridge

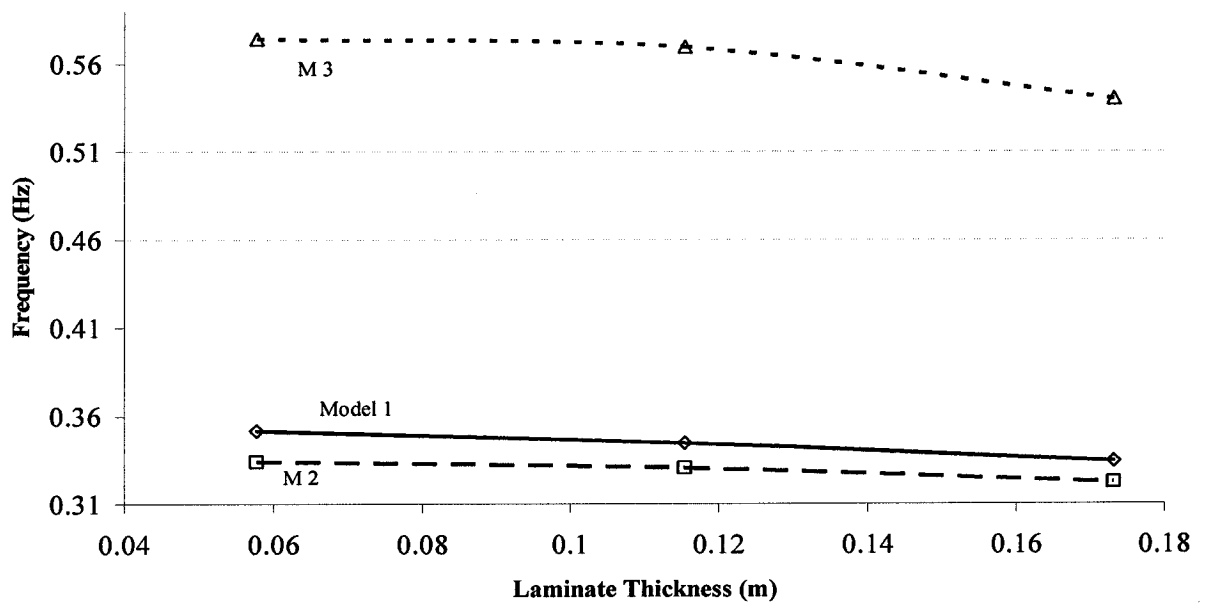


Figure 6.2.5.4.2 Laminated Thickness Effect on the 1st Torsional Natural Frequencies, LDFA-a, CCR 0.125m- Second Chang River Bridge

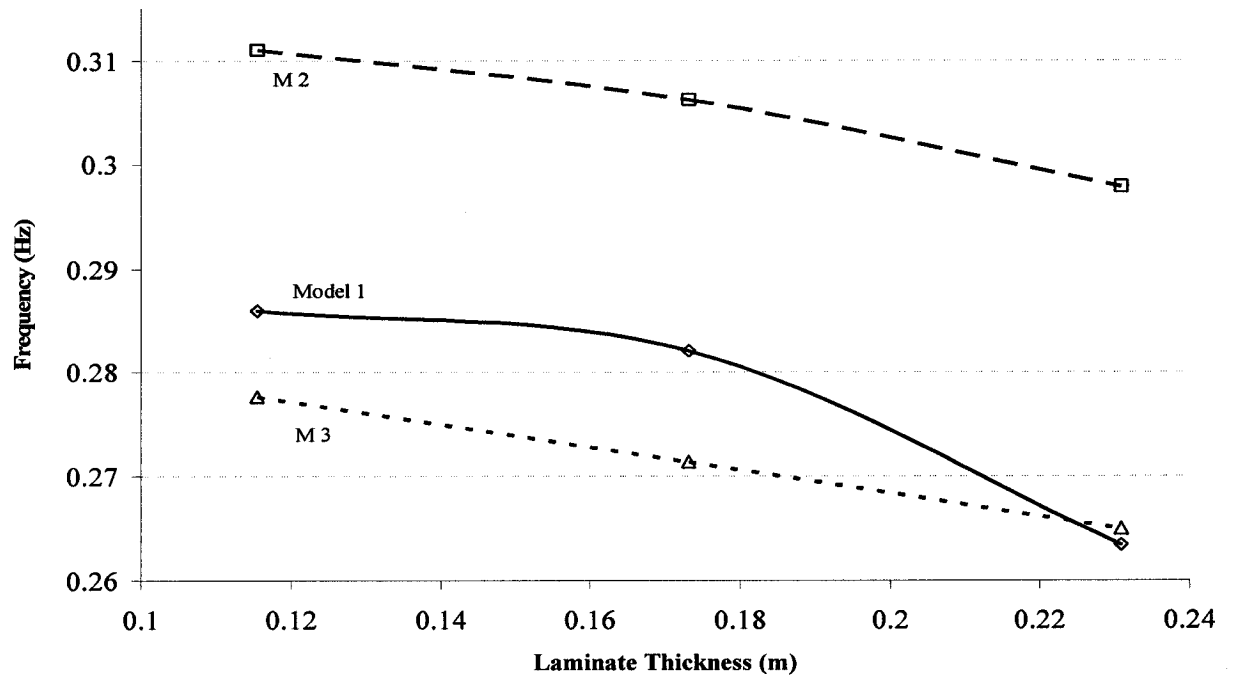


Figure 6.2.5.4.3 Laminate Thickness Effect on the 1st Torsional Natural Frequencies, LDFA-a, CCR 0.150m- Tatara Bridge

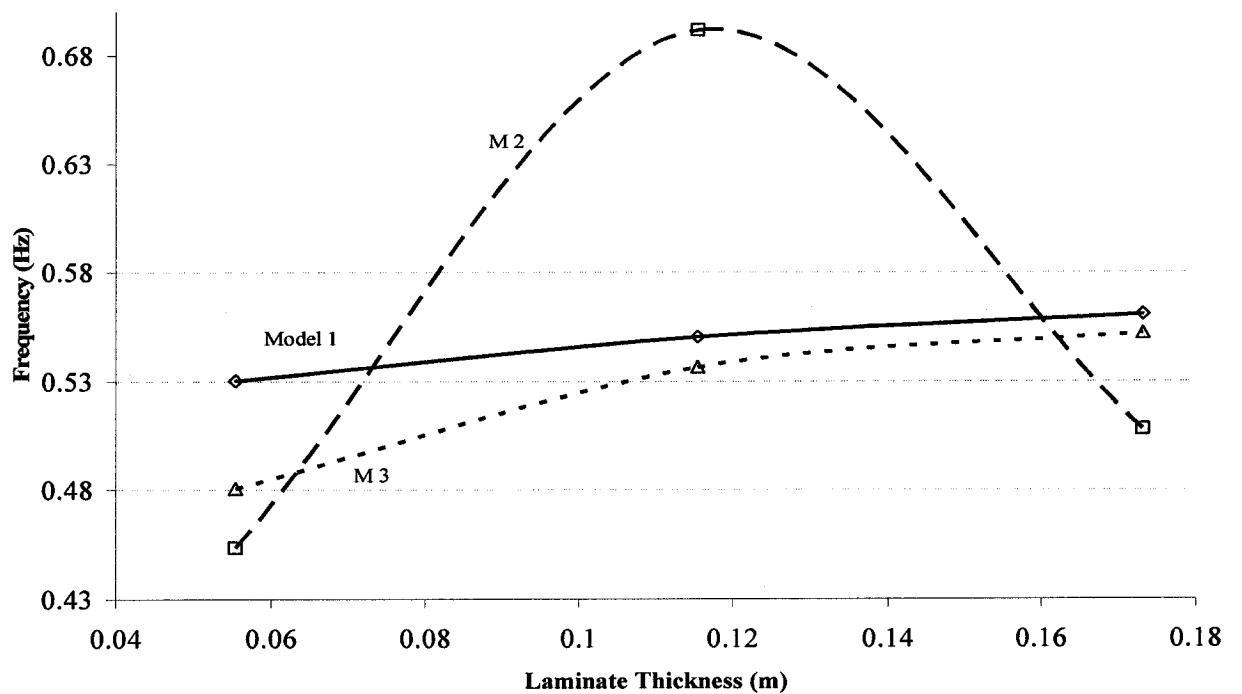


Figure 6.2.5.4.4 Laminate Thickness Effect on the 2<sup>nd</sup> Torsional Natural Frequencies, LDFA-a, CCR 0.100m- Annacis Island Bridge

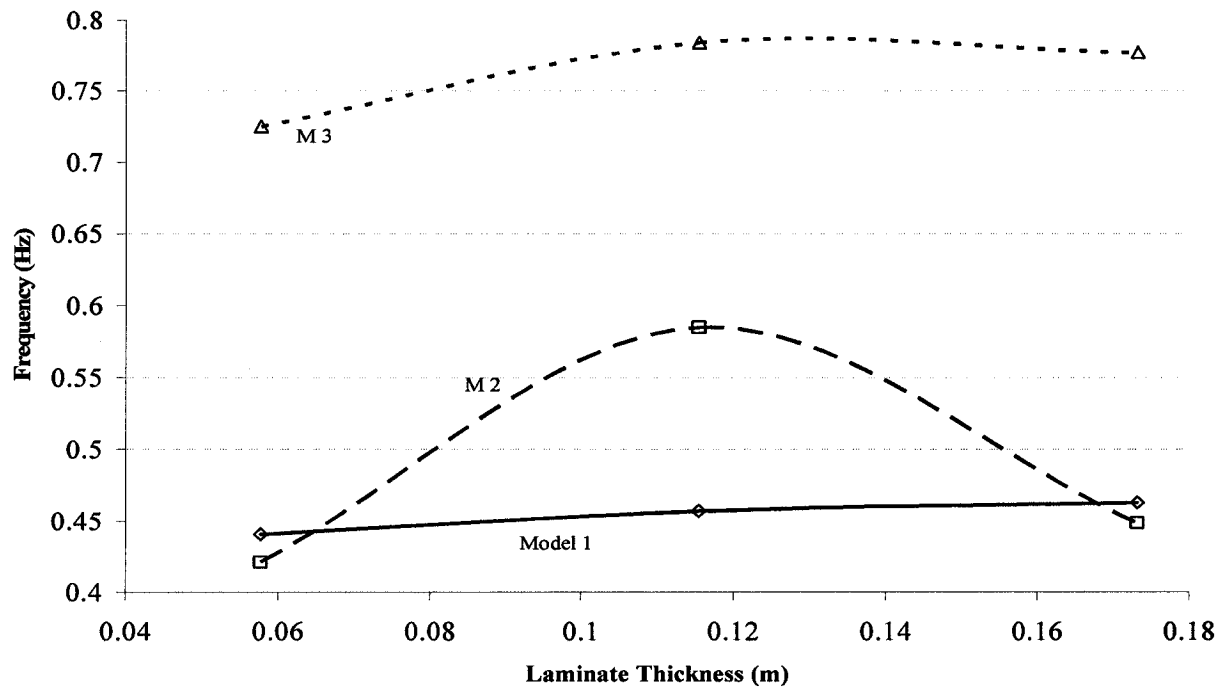


Figure 6.2.12.5 Laminate Thickness Effect on the 2<sup>nd</sup> Torsional Natural Frequencies, LDFA-a, CCR 0.125m- Second Chang River Bridge

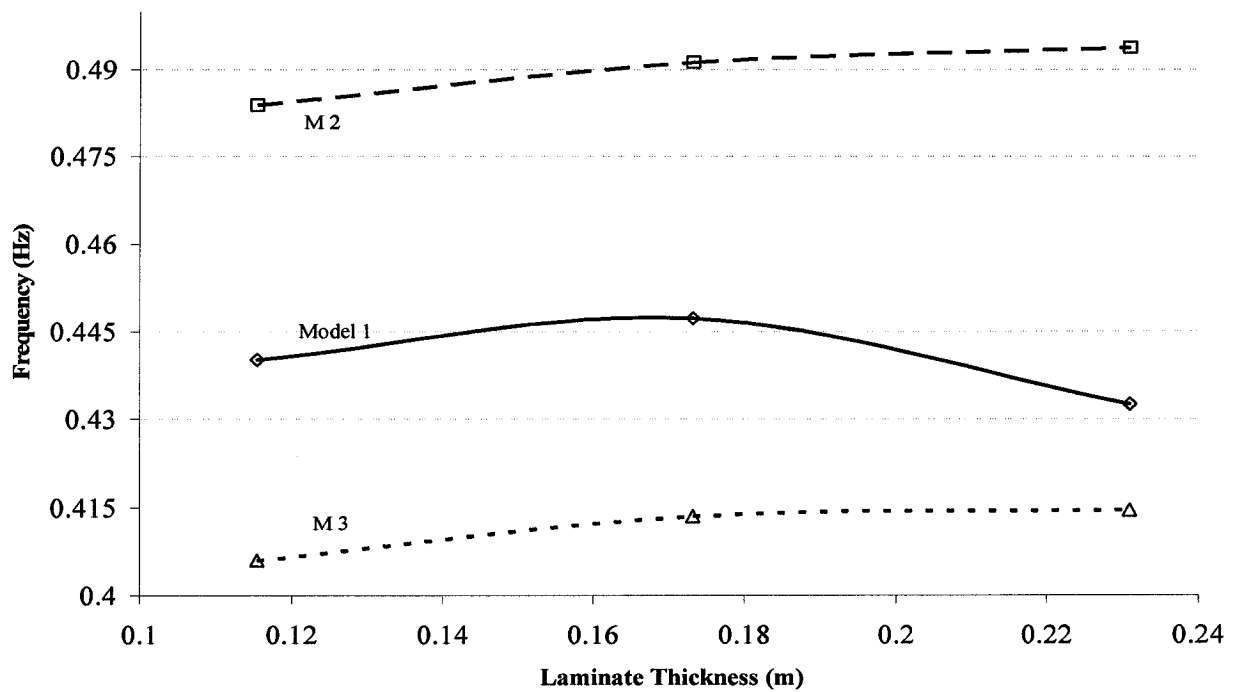


Figure 6.2.5.4.6 Laminate Thickness Effect on the 2<sup>nd</sup> Torsional Natural Frequencies, LDFA-a, CCR 0.150m- Tatara Bridge

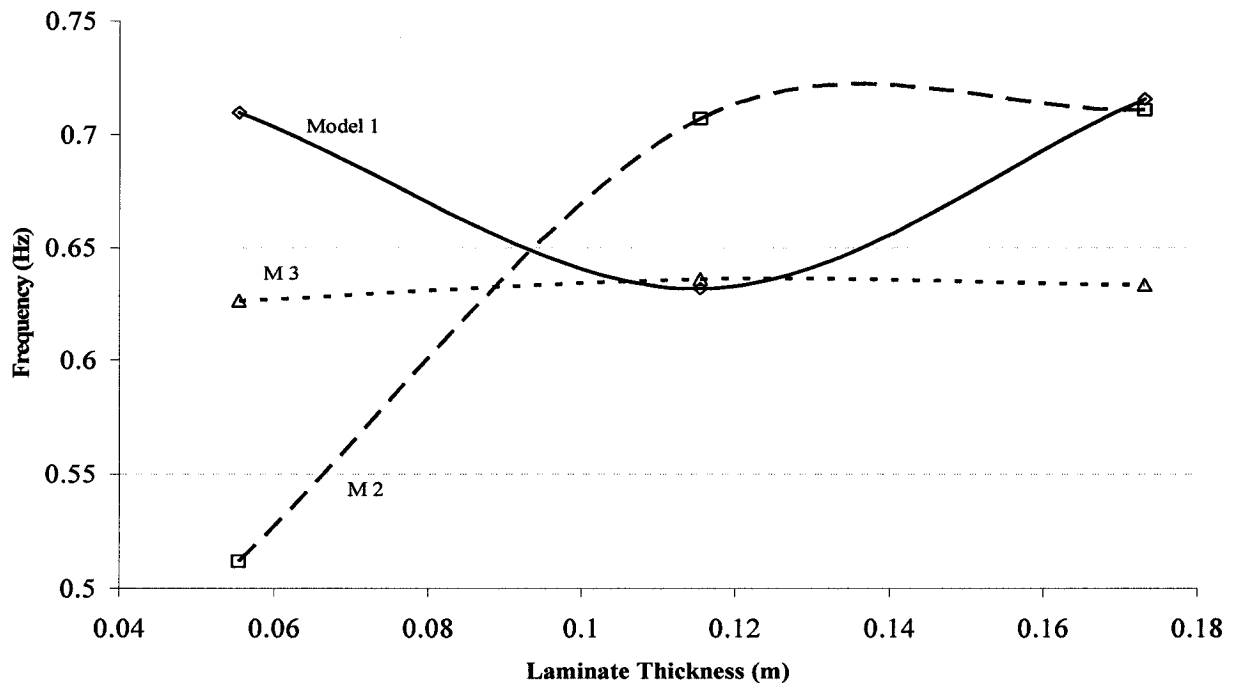


Figure 6.2.5.4.7 Laminate Thickness Effect on the 3<sup>rd</sup> Torsional Natural Frequencies, LDFA-a, CCR 0.100m- Annacis Island Bridge

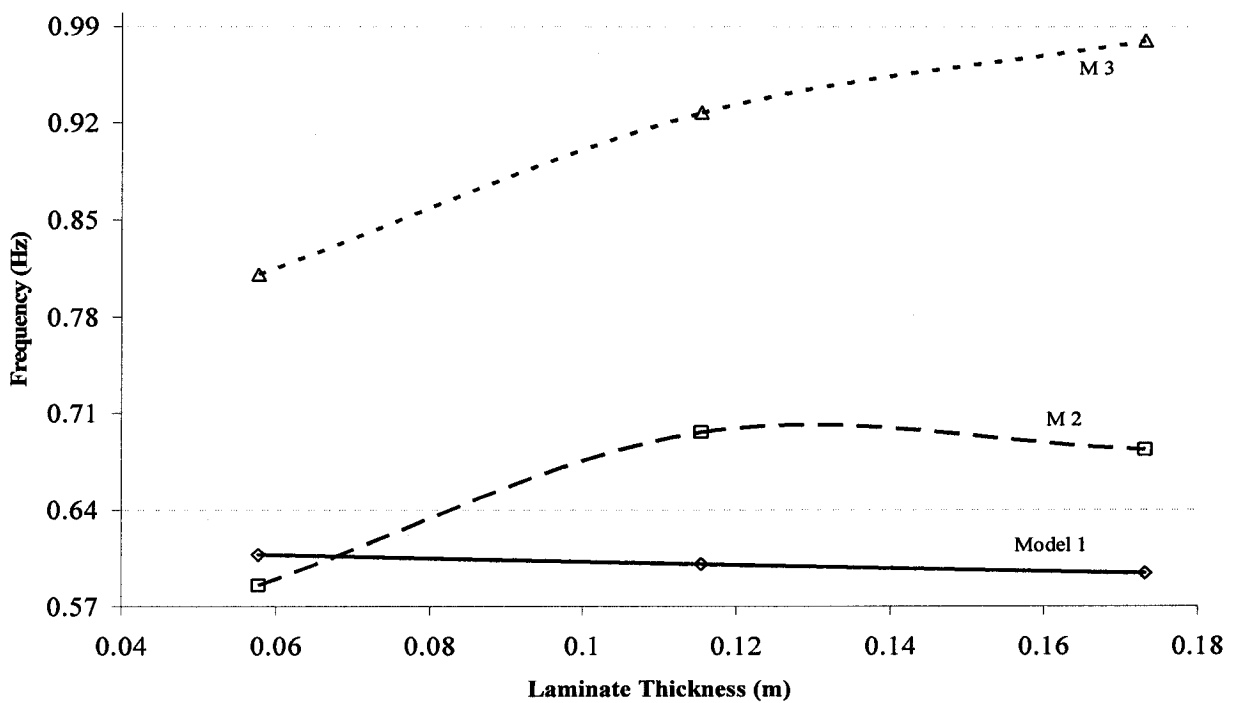


Figure 6.2.5.4.8 Laminate Thickness Effect on Natural Frequencies- 3<sup>rd</sup> Torsional – LDFA-a, CCR 0.125m- Second Chang River Bridge



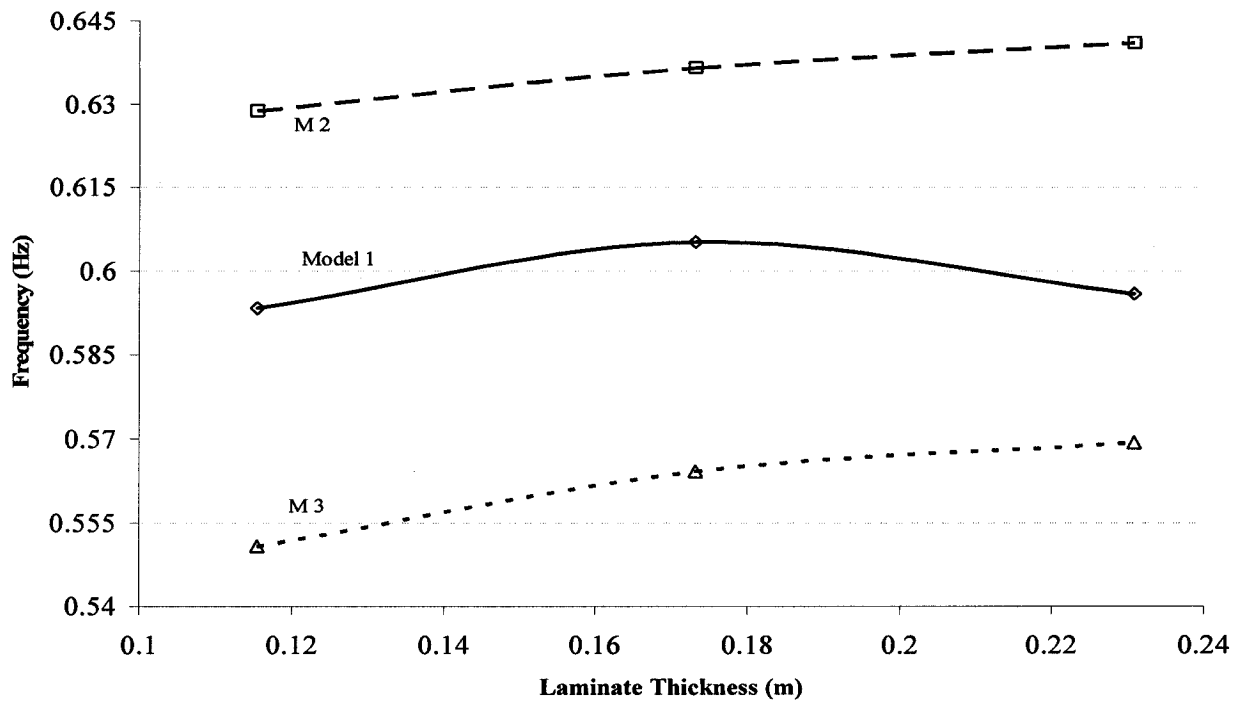


Figure 6.2.5.4.9 Laminate Thickness Effect on Natural Frequencies- 3<sup>rd</sup> Torsional – LDFA-a, CCR 0.150m- Tatara Bridge

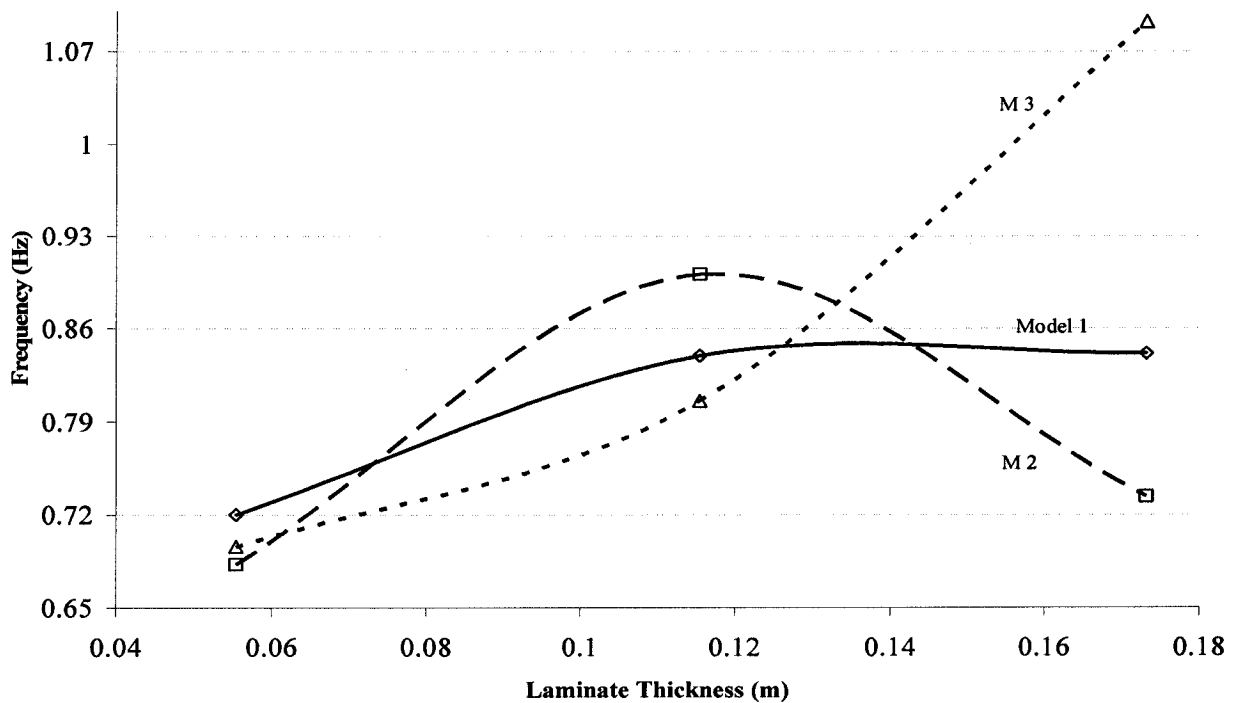


Figure 6.2.5.4.10 Laminate Thickness Effect on the 4<sup>th</sup> Torsional Natural Frequencies, LDFA-a, CCR 0.100m- Annacis Island Bridge

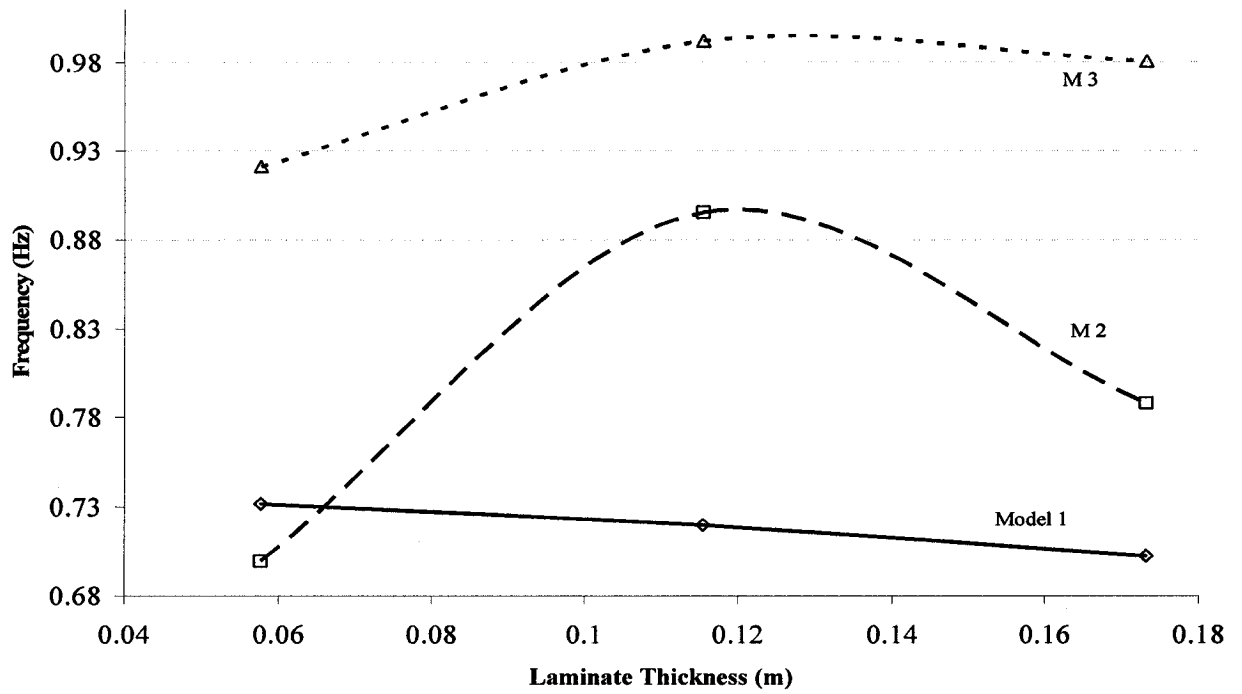


Figure 6.2.5.4.11 Laminate Thickness Effect on the 4<sup>th</sup> Torsional Natural Frequencies, LDFA-a, CCR 0.125m- Second Chang River Bridge

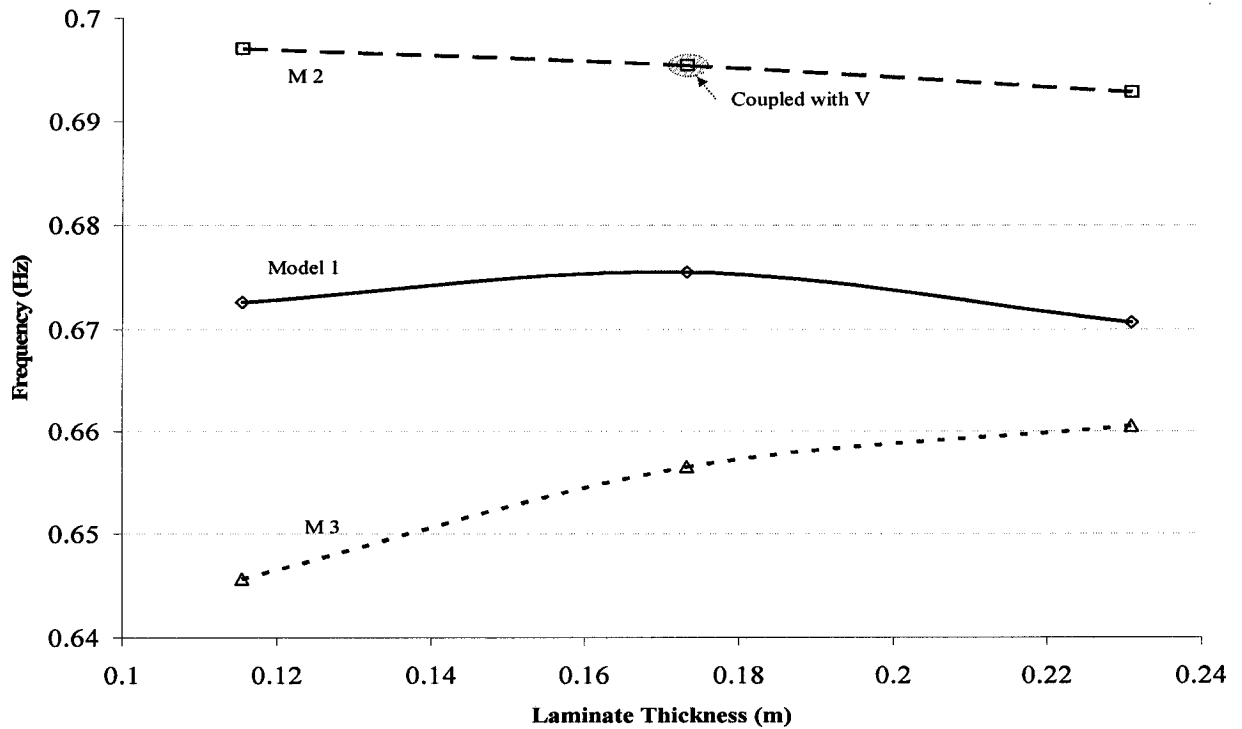


Figure 6.2.5.4.12 Laminate Thickness Effect on the 4<sup>th</sup> Torsional Natural Frequencies, LDFA-a, CCR 0.150m- Tatara Bridge

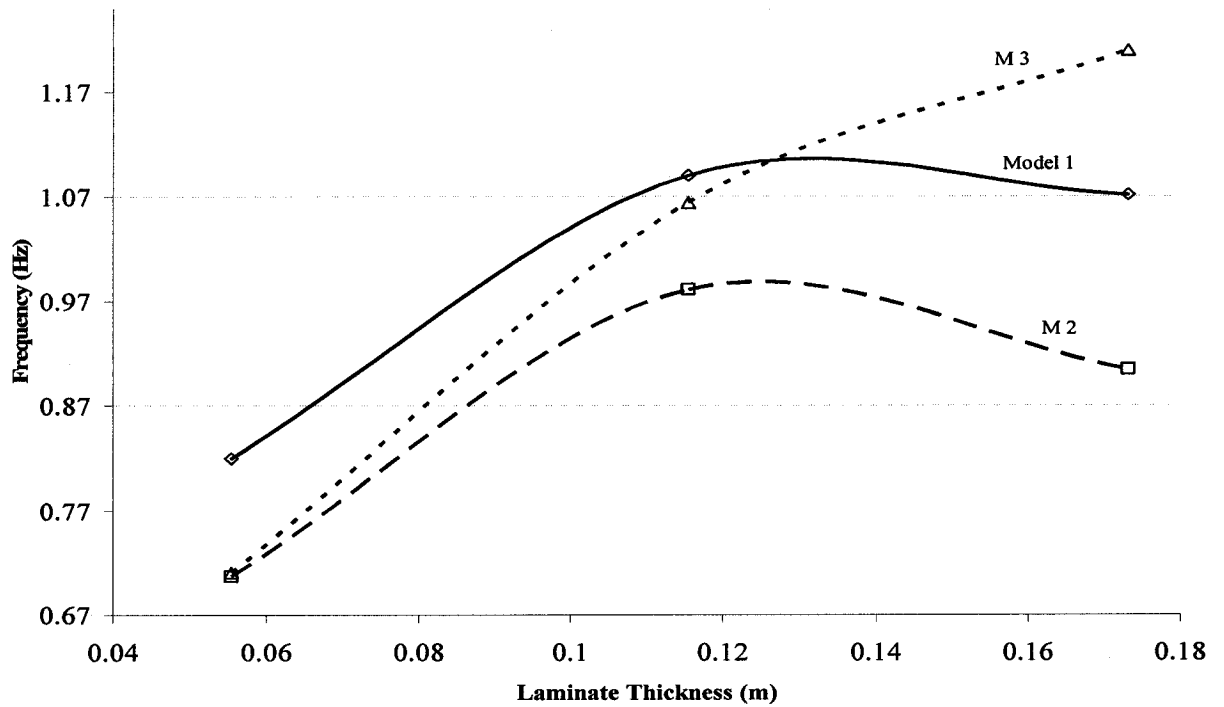


Figure 6.2.5.4.13 Laminate Thickness Effect on the 5<sup>th</sup> Torsional Natural Frequencies, LDFA-a, CCR 0.100m- Annacis Island Bridge

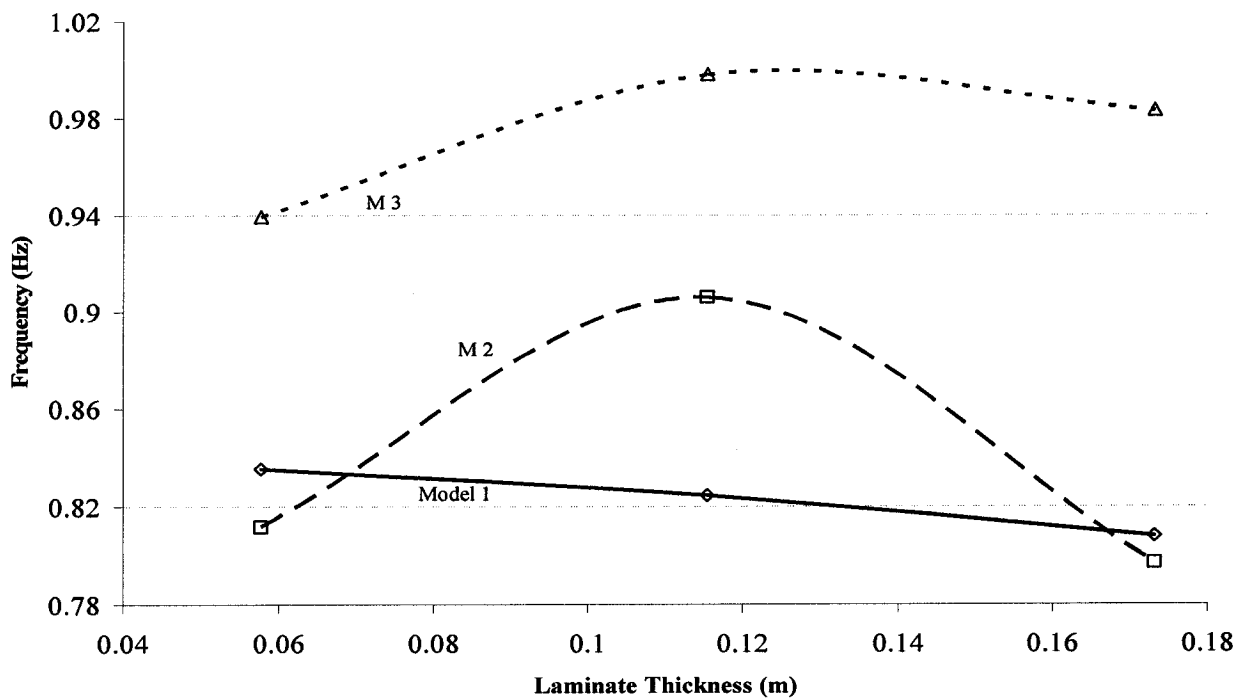


Figure 6.2.5.4.14 Laminate Thickness Effect on the 5<sup>th</sup> Torsional Natural Frequencies, LDFA-a, CCR 0.125m- Second Chang River Bridge

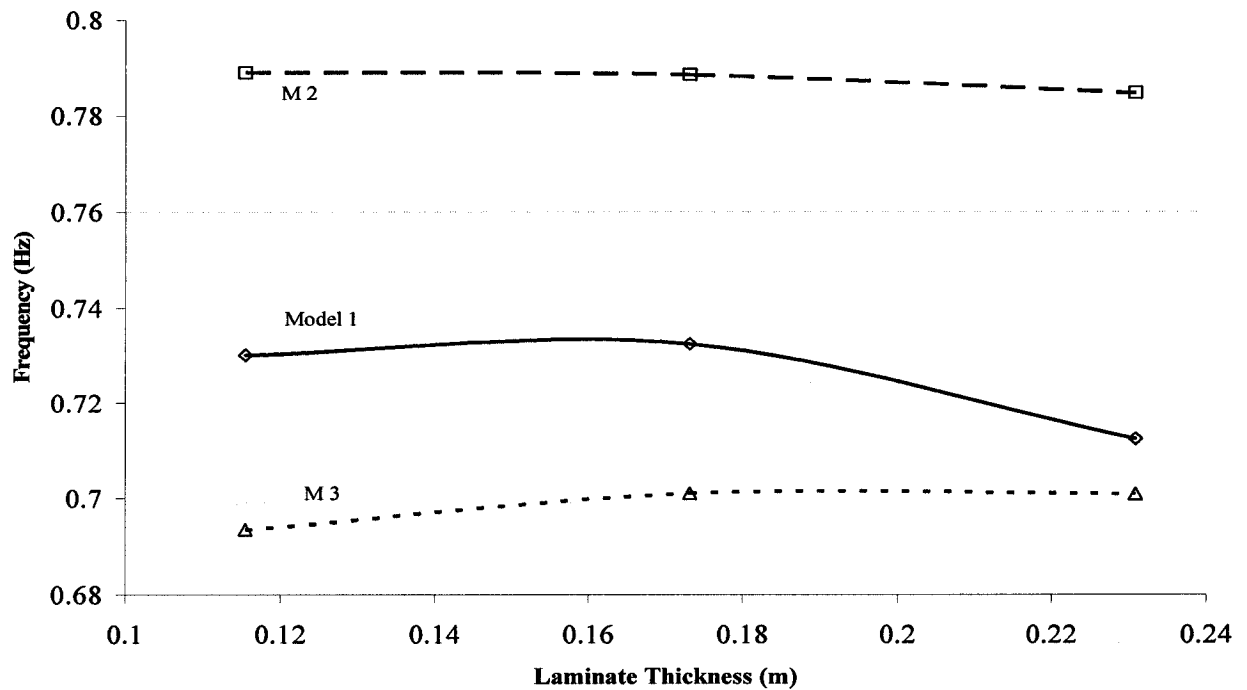


Figure 6.2.5.4.15 Laminate Thickness Effect on the 5<sup>th</sup> Torsional Natural Frequencies, LDFA-a, CCR 0.150- Tatara Bridge

### 6.2.5.5 Effect of Laminate Thickness on the Horizontal Natural Frequencies:

It has been found that the Horizontal Natural Frequencies (HNF) of Models I and III are very slightly affected by the increase of the laminate thickness, and Model II shows an increase with the short bridge central span length (Figures 6.2.5.5.1 to 3). However, the Horizontal Natural Frequencies (HNF) of Model IV appear to be sensitive to the increase of the laminate thickness (LTh) and generally show an increase in their magnitude with the Second Chang River Bridge. Increasing the laminate thickness could also release the coupling of the higher HNF (third and fourth) with the VNF of Model IV, for the medium central span length. However, Model IV- HNFs show very slight change with the shortest and longest span lengths (Figures 6.2.5.5.4 to 6).

As with the two other types of natural frequencies, the effect of the increase in the deck mass, and its balance with the increase in the deck horizontal flexural stiffness, due to the increase of the laminate thickness, explains the behavior of the HNF. However, the horizontal flexural stiffnesses of all models are high in comparison to the other deck stiffnesses and this clarifies the lesser effect of the laminate thickness increase on the

HNF in comparison with the other natural frequencies. Model I has the highest lateral flexural stiffness among the models, and the increase of the lateral flexural stiffness due to the laminate thickness increase is apparently ineffective. The increase of the mass of this model is relatively high, but because of its high stiffness the HNF is not affected with the shortest and the medium bridge central span lengths (Figures 6.2.5.5.1 and 2). With the longest span the effect of the mass becomes apparent when the laminate thickness is greatly reducing the HNF (Figures 6.2.5.5.3). Model II has the lowest lateral stiffness and lowest mass, and with the shortest span it becomes responsive to the laminate thickness increase (Figure 6.2.5.5.1). As the central span increases, the effect of the increasing mass overcomes the increase in the stiffness and reverses the increase in the HNF to a slight decrease (Figure 6.2.5.5.2 and 3). Model III mass is slightly less than Model I but its lateral moment of inertia is much less than that of Model I because of its cylindrical inner tubes. This is reflected in a decrease of the HNF in the shortest and medium bridge central span lengths (Figures 6.2.5.5.1 and 2). The lower mass effect results in increasing the HNF of this model for the longest bridge span length (Figure 6.2.5.5.2 and 3).

The increase of the horizontal stiffness contribution to the horizontal natural frequencies, due to the increase of the laminate thickness, explains the increase in Model IV- HNF. The increase in the mass is small for this model, as the majority of the deck material of its deck is formed from the chopped FRP. However, because of the two tubular laminates and the stiffening laminate position relative to the deck central plane in the horizontal direction (or Y-Z or 2-3 plane of the un-deformed bridge), gives high contributions to the horizontal stiffness of the deck. On the other hand, Model IV horizontal flexural stiffness is relatively low, and the Second Chang River Bridge has the widest deck as shown earlier. Hence, the increase of the laminate thickness increases the horizontal flexural stiffness effectively and results in the shown rise of the HNF (Figure 6.2.5.5.5). With the lesser deck width of the Annacis Island and the Tatara Bridge, the rise in the flexural stiffness in the lateral direction is reduced and becomes ineffective, giving almost constant HNF (Figures 6.2.5.5.4 and 6).

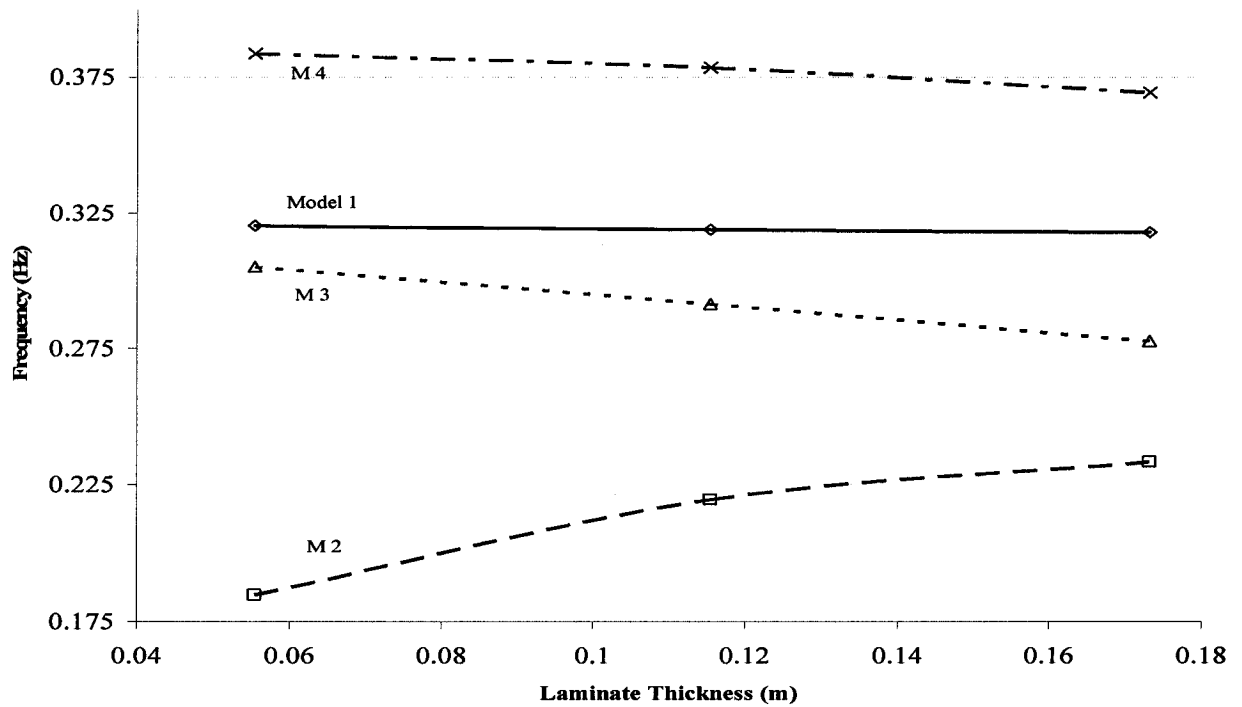


Figure 6.2.5.5.1 Laminate Thickness Effect on the 1st Horizontal Natural Frequencies, LDFA-a, CCR 0.100m- Annacis Island Bridge

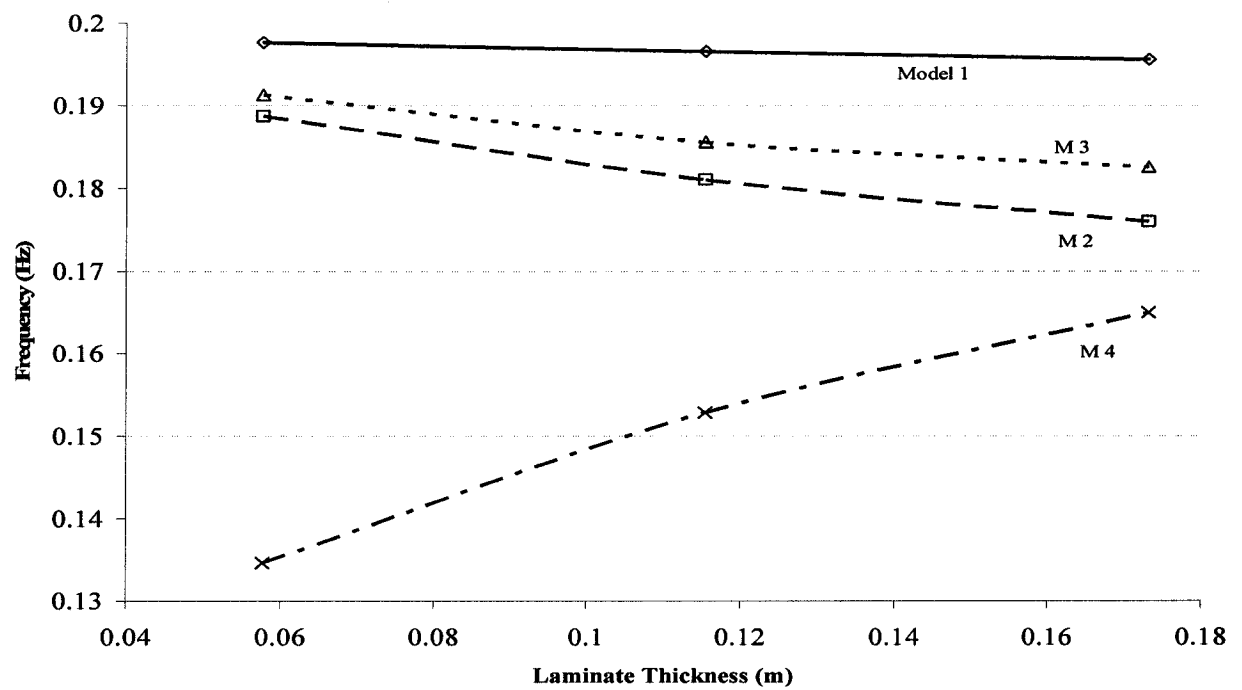


Figure 6.2.5.5.2 Laminate Thickness Effect on the 1st Horizontal Natural Frequencies, LDFA-a, CCR 0.125m- Second Chang River Bridge

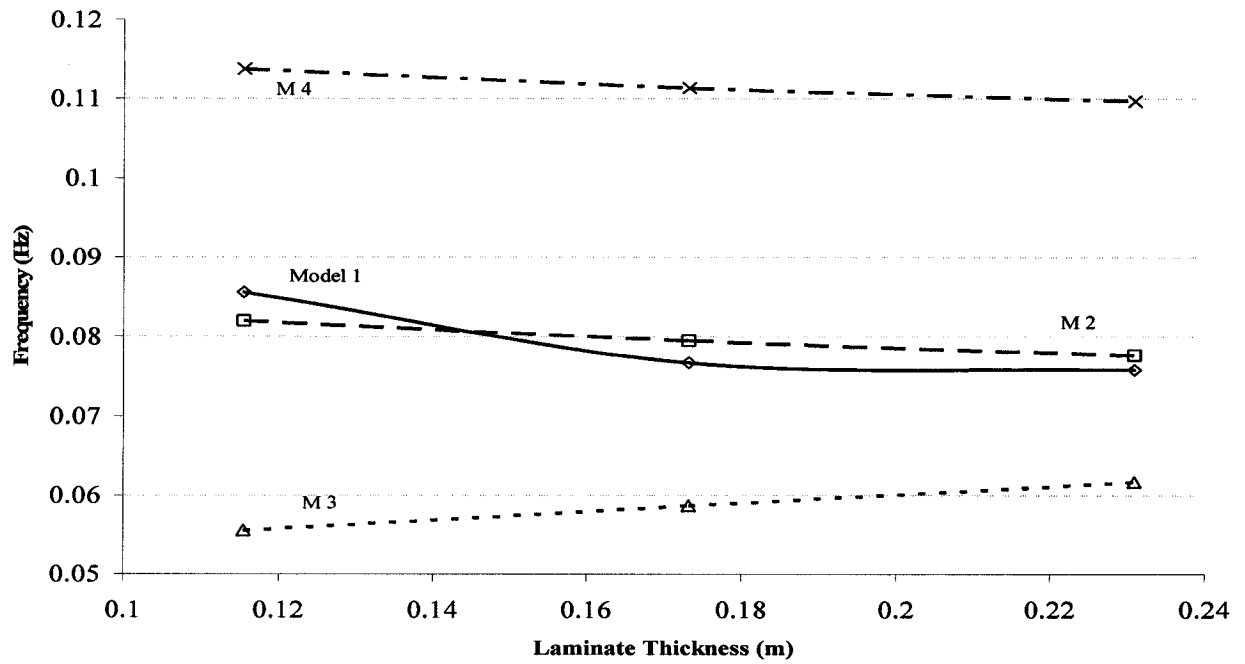


Figure 6.2.5.5.3 Laminate Thickness Effect on the 1st Horizontal Natural Frequencies, LDFA-a, CCR 0.150m- Tatara Bridge

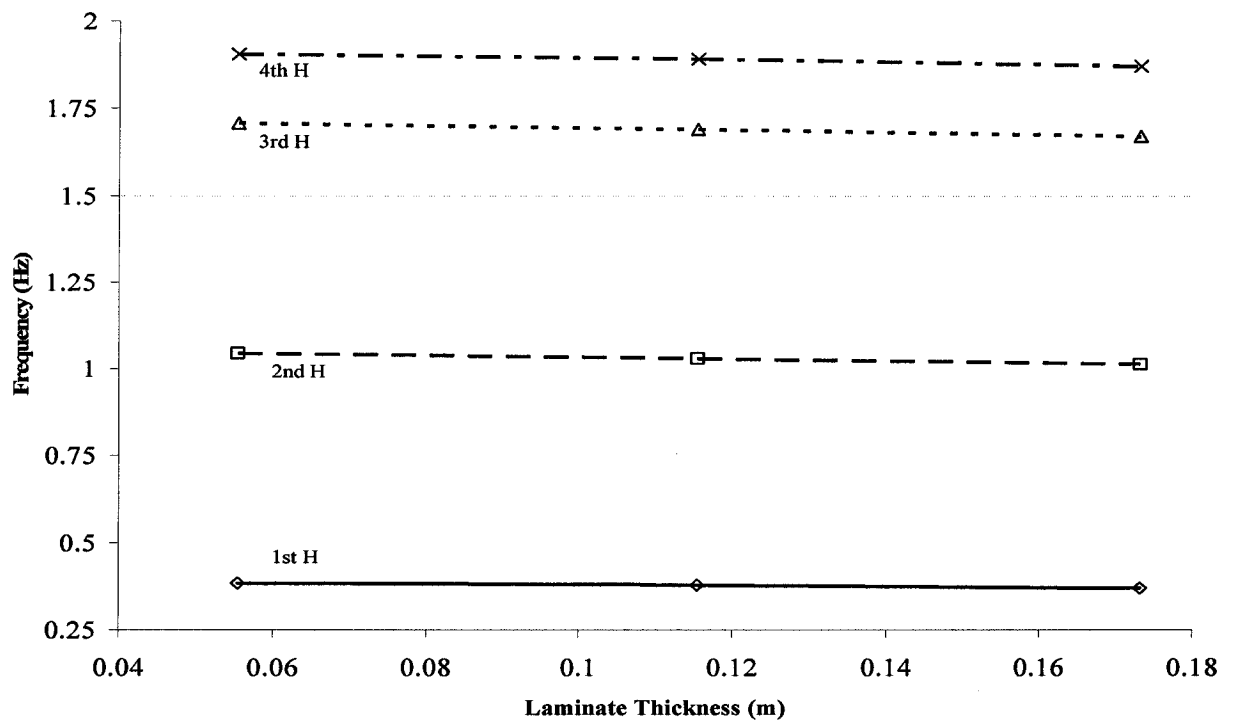


Figure 6.2.5.5.4 Laminate Thickness Effect on the Horizontal Natural Frequencies- Model IV – LDFA-a, CCR 0.100m- Annacis Island Bridge

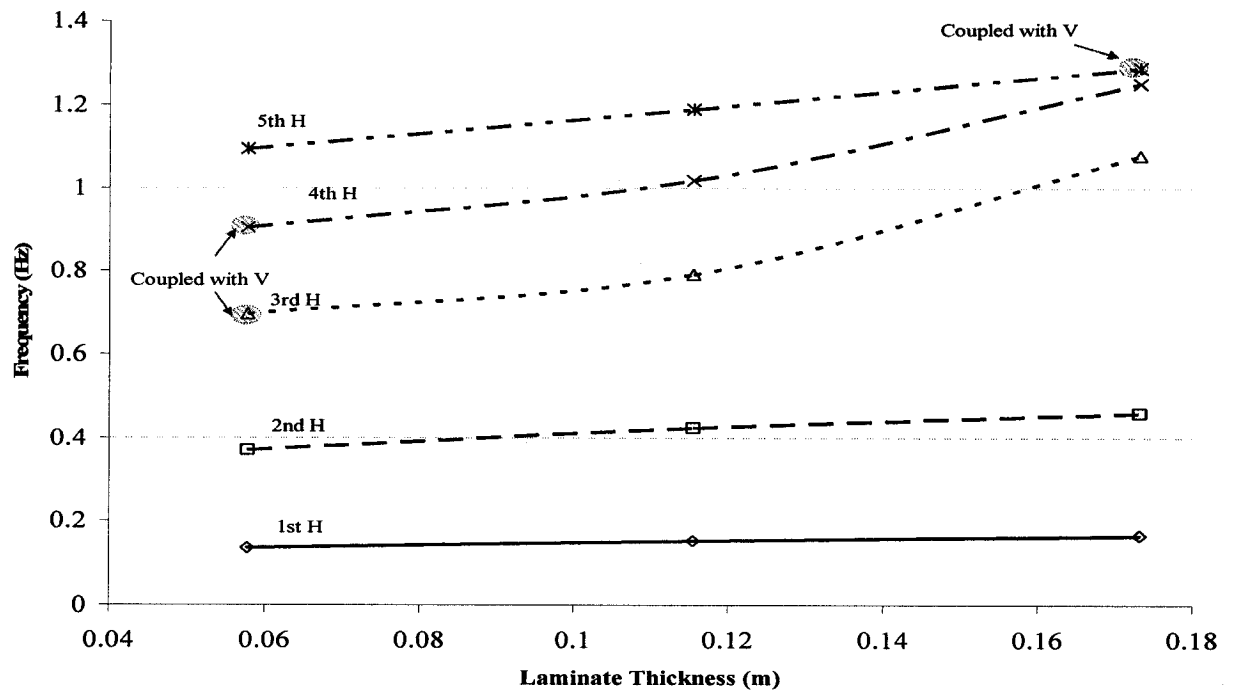


Figure 6.2.5.5.5 Laminate Thickness Effect on the Horizontal Natural Frequencies- Model IV – LDFA-a, CCR 0.125m- Second Chang River Bridge

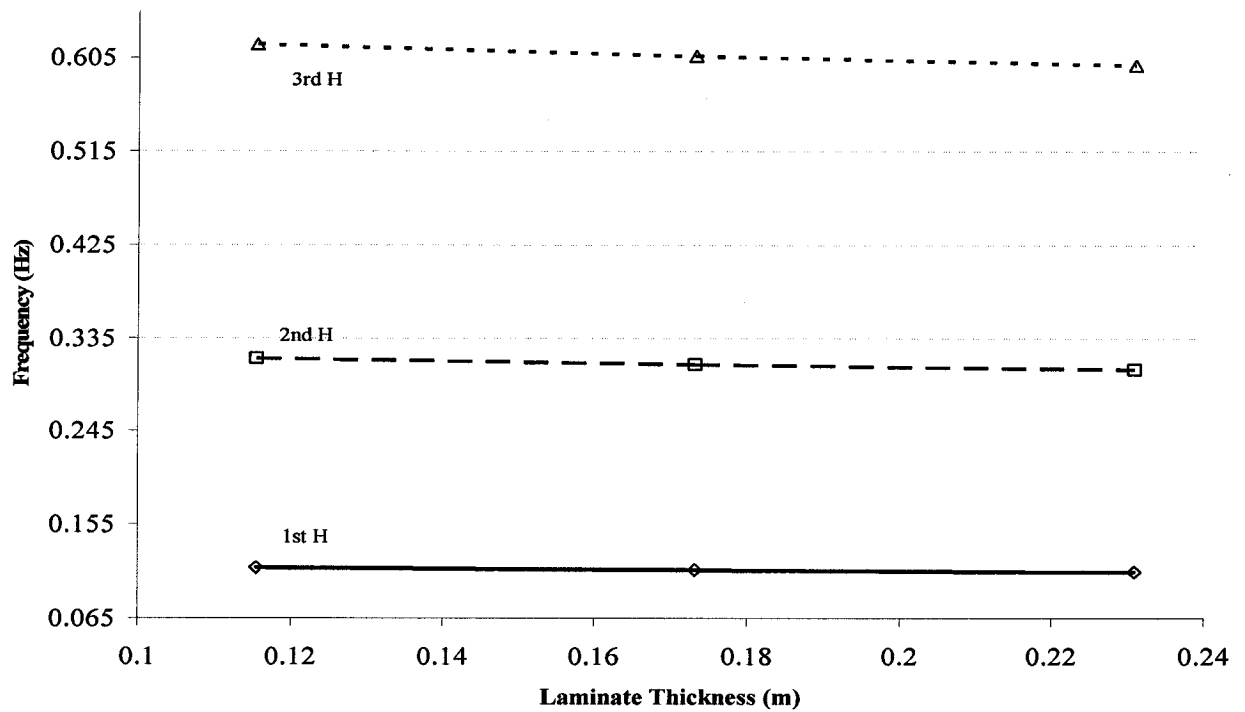


Figure 6.2.5.5.6 Laminate Thickness Effect on the Horizontal Natural Frequencies- Model IV – LDFA-a, CCR 150- Tatara Bridge



## **6.3 CFRP Cables:**

### **6.3.1 Effect of Cable Radius on the Maximum Deck Top Surface Displacements:**

It has been found that the increase in the CFRP Cable Radius (CCR) leads to significant decrease of the deck maximum vertical displacement (or maximum deflection) of all bridges and models. It also results in a reduction of the lateral displacement, with some exceptions. However the maximum longitudinal displacements either remain constant or slightly increase, with some exceptions.

The maximum absolute values of all the deck displacements are highly affected by the increase of the CCR. The effects can be summarized as follows: (i) absolute maximum displacements  $U$  are generally decreased, with two exceptions (Figures 6.3.1.1 to 6.3.1.3); Model II shows linear increase with the shortest span (Figures 6.3.1.1), and Model IV shows a very high increase then a slight decrease with the medium bridge central span length (Figures 6.3.1.2). (ii) For all bridge central span lengths, the maximum absolute values of the vertical displacement  $V$  show a huge reduction with successively reduced rates. The percentage of the total reduction is decreased as the bridge central-span is increased (Figures 6.3.1.4 to 6.3.1.6). (iii) The maximum absolute values of the longitudinal displacement  $W$  show either a constant level or slight increase, with some exceptions for the shortest and medium central spans (Figures 6.3.1.7 to 6.3.1.9). The Model IV longitudinal displacement decreases with the shortest span and increases very highly with the medium central span. Model III shows an increase with the shortest central span and extremely high increase with the medium span.

CFRP cables have very high stiffness to weight ratios and the losses in their Ernest modulus, or apparent modulus, due to the catenary action are very small in comparison to traditional steel cables (Chapter3, Section 3.3). In addition, their high ultimate and fatigue strength to weight ratios allow very high tensile stresses to be safely applied on the stay cables, which leads to further increase in their apparent modulus and hence their stiffness. Basically, stay cables are axial members and their major stiffnesses are axial, although they have some flexural and torsional stiffnesses. The cable stays serve as elastic supports to the central and side spans of the cable stayed bridge system. Consequently, they provide direct support to the deck in the vertical direction by the

vertical components of their axial tensile forces; however the horizontal components are transformed to an axial compression accumulated through the deck length and are maximum in the pylon zones. This explains the huge and dominant reduction of the maximum vertical displacements (or the maximum deflection) of the deck for all models, as the CFRP cables radius increases. The elastic restraint of the cables to the lateral, longitudinal, and the three rotational displacements of the deck at the anchorage points are small. The Models that have laminated box girders, Models I, II and III, show almost the same behavior. Model IV gives a higher rate of decrease in its maximum deflection with the second Chang River Bridge (Figures 6.3.1.5). The maximum deflection for this model is at the center of the “slab and grid” part in the bridge longitudinal direction. The flexures of all the deck parts are similar in the longitudinal bridge direction. The two “side boxes” are supported directly on the stay system and they basically have high moments of inertia in comparison with the “slab-grid” part. They behave as if they are huge longitudinal girders. As it has relatively low stiffness, the internal “slab-grid” part behaves as a beam connected to two elastic supports of equal stiffness (i.e. the two side “boxes”) in the lateral direction. Once the cable radius increases, the stiffness of the “elastic supports” are greatly increased, and their restraining action significantly improved, which results in a significant decrease of the lateral flexure of the intermediate part of the section and further reduces the maximum deflection. This is apparent with the Second Chang River Bridge as its deck width is a maximum.

The bend-twist coupling of the laminated material of the deck results in the unsymmetrical distribution of the lateral displacement. Depending on the deck stiffness, the distribution of the lateral displacement is changed from model to model, as the CFRP cable radius increases. For the “stiff” deck sections, the increase of the cable radius and the resulting restraint to the deck flexure-twisting action reduces the lateral displacement (Figures 6.3.1.1-6.3.1.3). For example, the slight difference in the vertical flexural stiffness between Models I and III is reflected in the form of a slight difference of their lateral displacement rates of decrease. The “stiff” decks of Models I and III are deformed laterally without “extra” local twisting in between the cable anchorage points. The Model II deck is much more flexible; specifically it has less torsional stiffness. As the elastic supporting action of the cables increases in the vertical flexural direction, Model II’s tendency to twist locally in the unsupported spans between the cable anchorage points is

also increased. The effectiveness of the stay cables elastic supporting action is high with the shortest bridge central spans for their shorter lengths; hence, Model II shows a higher increase in its lateral displacement  $U$ . On the other hand, Model IV has three laminated parts, the two side tubular laminates, and the stiffening laminate. Each tubular part is connected over most of its external surface to the big side part of the chopped FRP material. Therefore, the tubular laminates show very low lateral displacements in general. On the other hand, the wide stiffening laminate is attached to the intermediate, relatively flexible, part of the section. As the cable radius increases, the “slab-girder” is further restrained from the sides, and the bending stresses are increased. Consequently, when the width of the deck is high for the second Chang River Bridge, the bend-twist action of this laminate is highly increase and results in an unsymmetrical maximum zone of  $U$  close to the central part of the deck.

Increasing the cable radius reduces the maximum cable tensile stresses in the stay cables, (Figures 6.3.2.11 to 6.3.2.13), however, it increases the stay cables tensile forces as well (Figures 6.3.2.14 to 6.3.2.16). This results in a slight increase of the axial forces of the deck, changes the stretching pattern in the longitudinal direction and results in a slight increase of the longitudinal displacement, which is the general observed behavior. For the models with the cylindrical tubular laminates, the high bending-extension coupling action contributes in a high increase of  $W$ , particularly in the side-laminated tubes. As the restraint on the deck sides increases, the moment stresses rise and become very high with the wide deck of the Second Chang River. This explains the high rise in Model III and IV maximum  $W$  with this bridge.

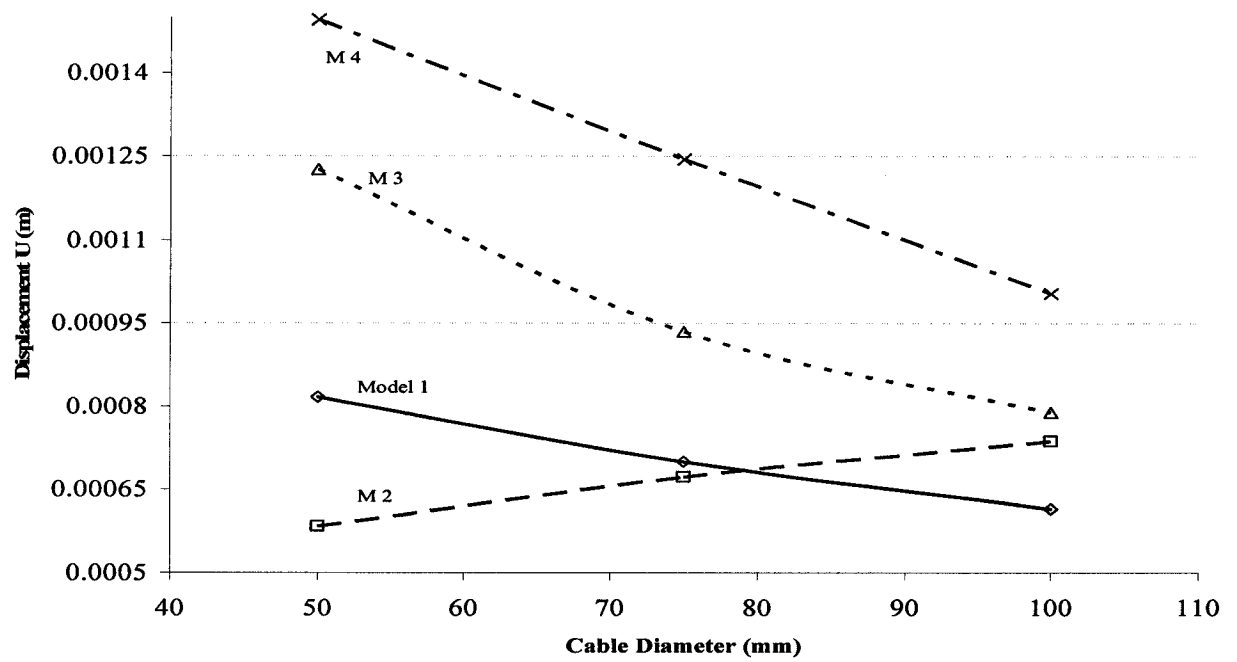


Figure 6.3.1.1 CFRP Cable Radius Effect on Horizontal Displacement (U)– LDFA-a, Tha ,LLA, Annacis Island Bridge

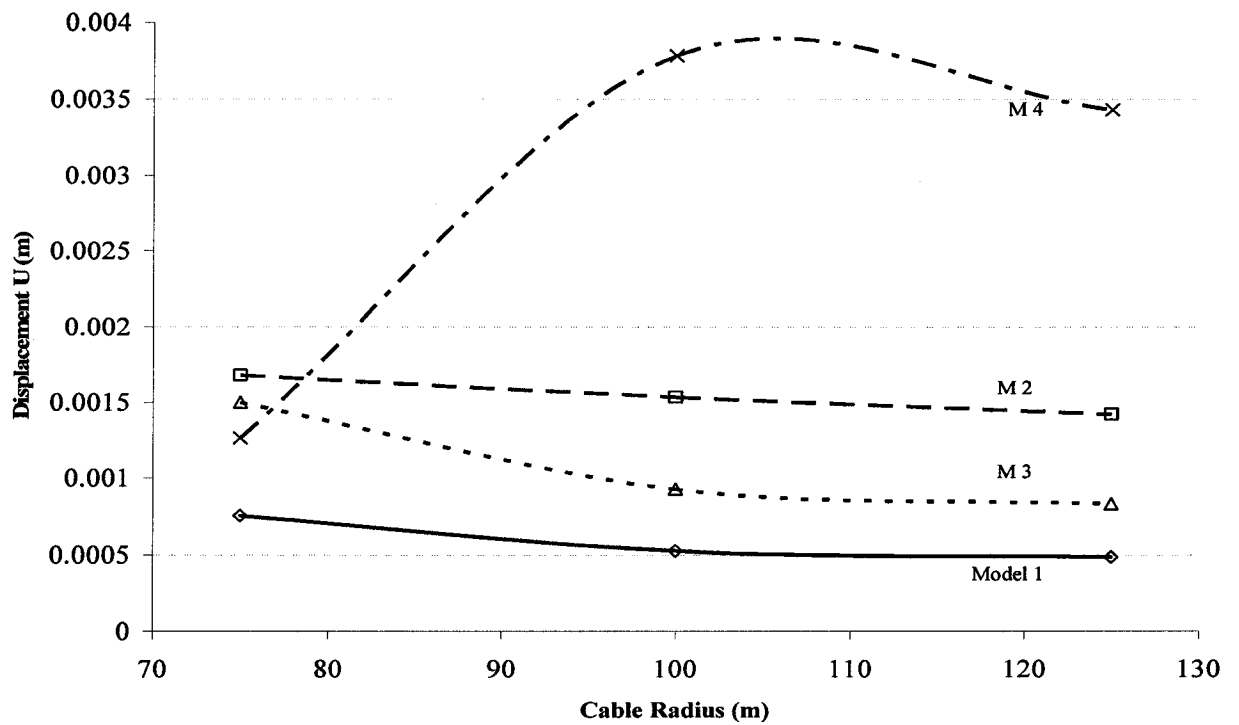


Figure 6.3.1.2 CFRP Cable Radius Effect on Lateral Displacement (U) – LDFA-a, Tha, LLA– Second Chang River Bridge

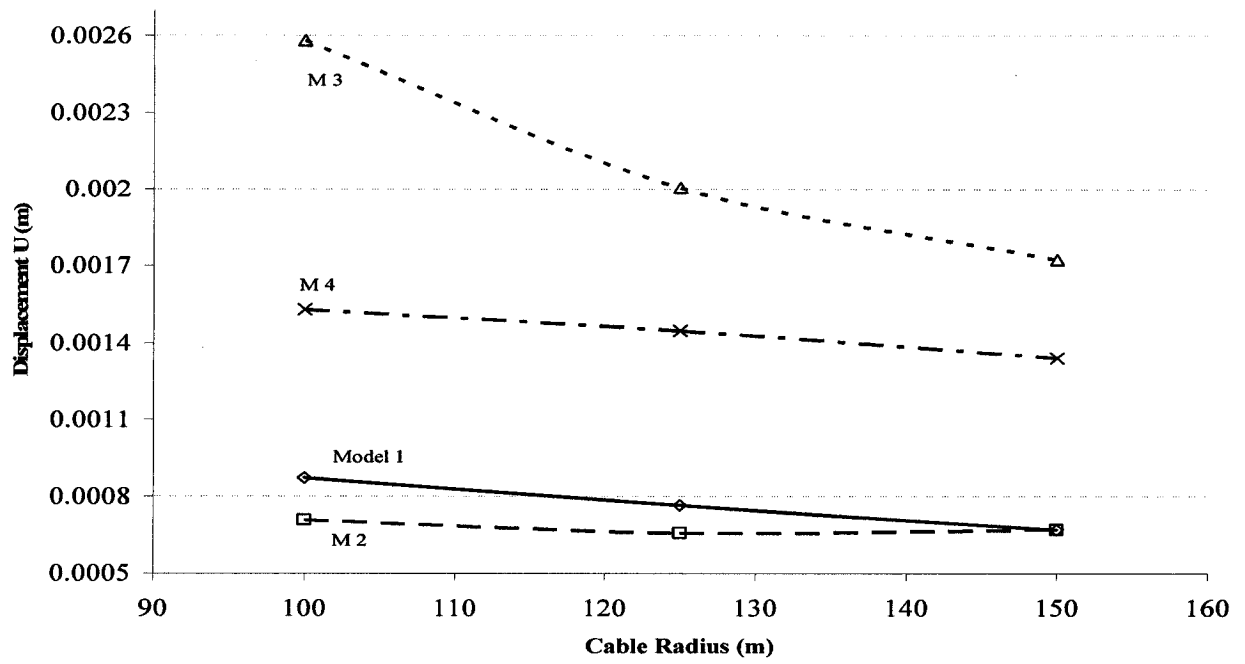


Figure 6.3.1.3 CFRP Cable Radius Effect on Lateral Displacement (U) – LDFA-a, Tha, LLA, Tatara Bridge

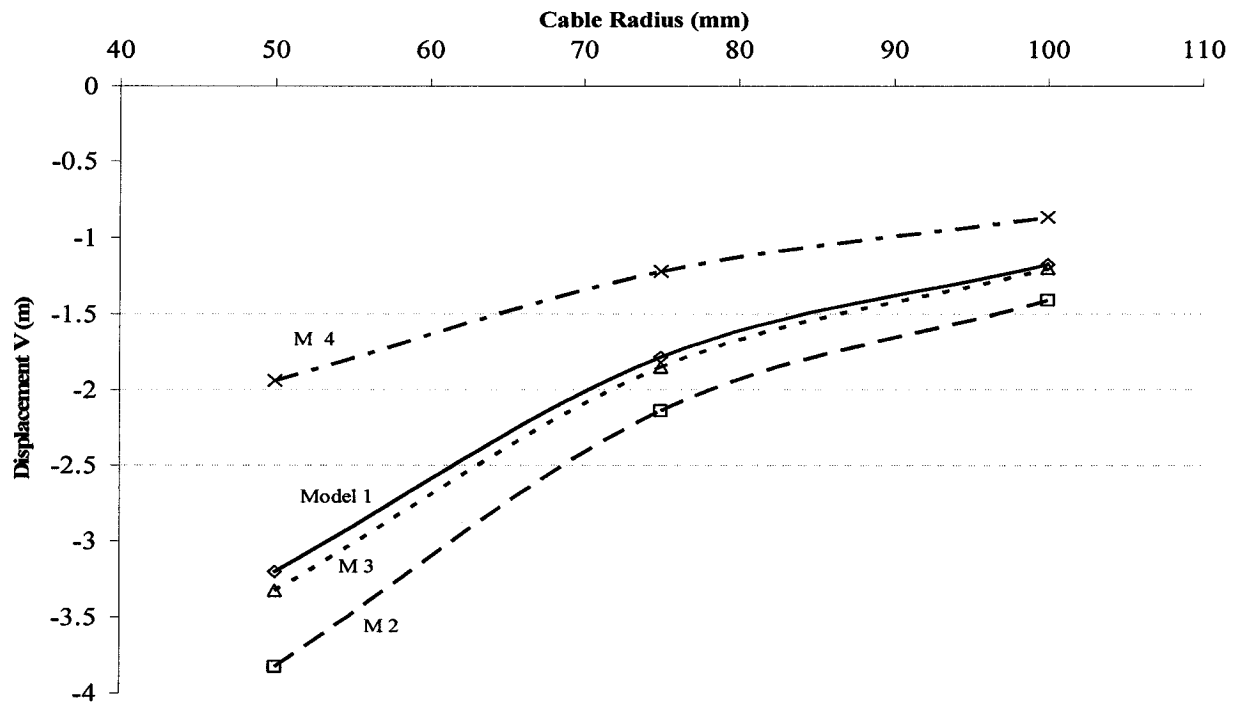


Figure 6.3.1.4 CFRP Cable Radius Effect on Vertical Displacement (V) – LDFA-a, Tha, LLA, Annacis Island Bridge

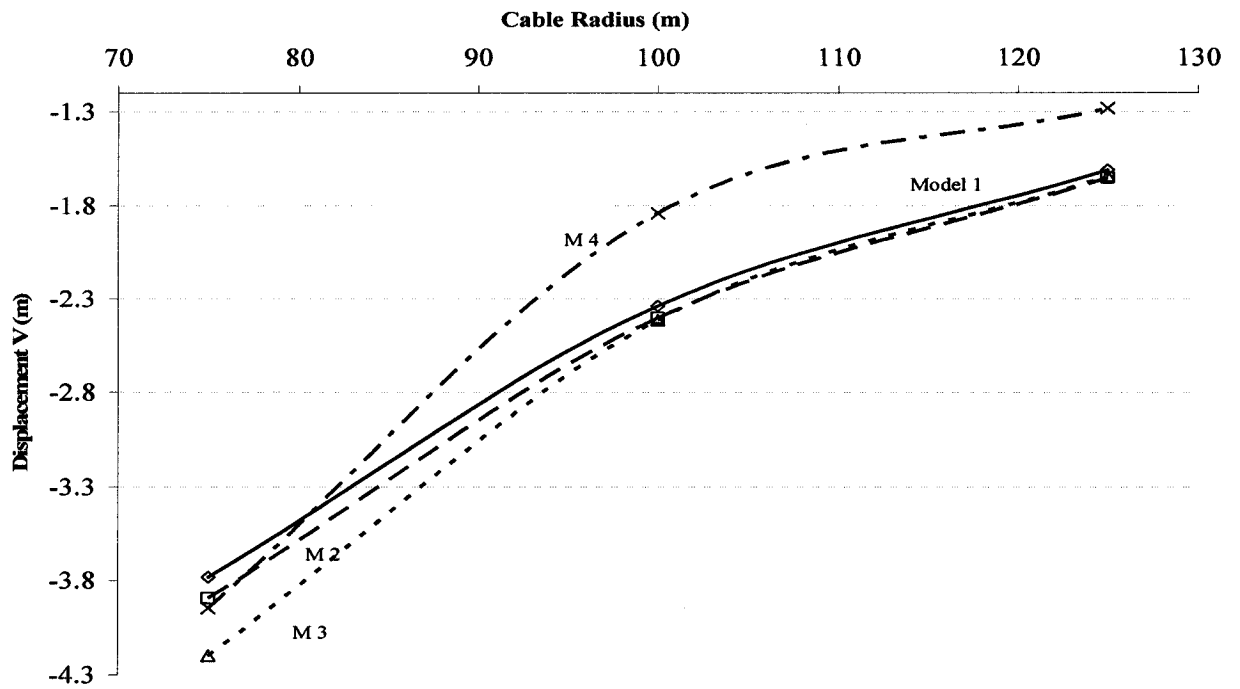


Figure 6.3.1.5 CFRP Cable Radius Effect on Vertical Displacement (V) – LDFA-a, Tha, LLA, Second Chang River Bridge

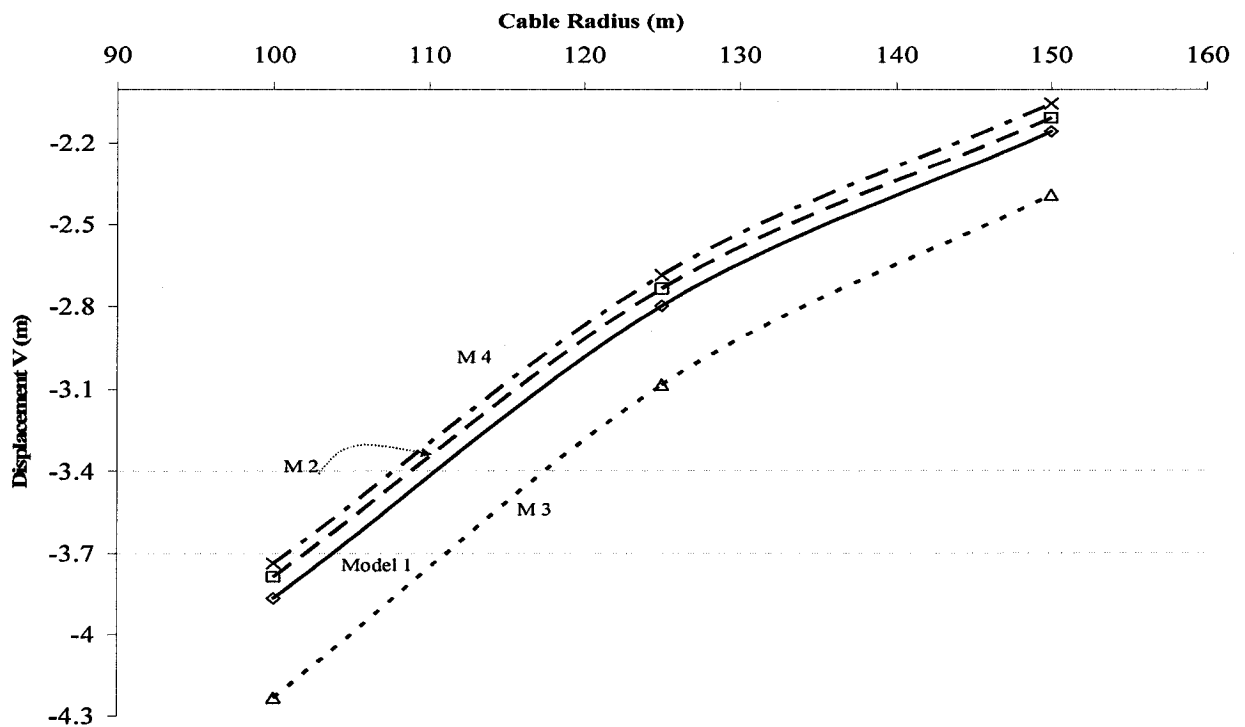


Figure 6.3.1.6 CFRP Cable Radius Effect on Vertical Displacement (V) – LDFA-a, Tha, LLA, Tatara Bridge

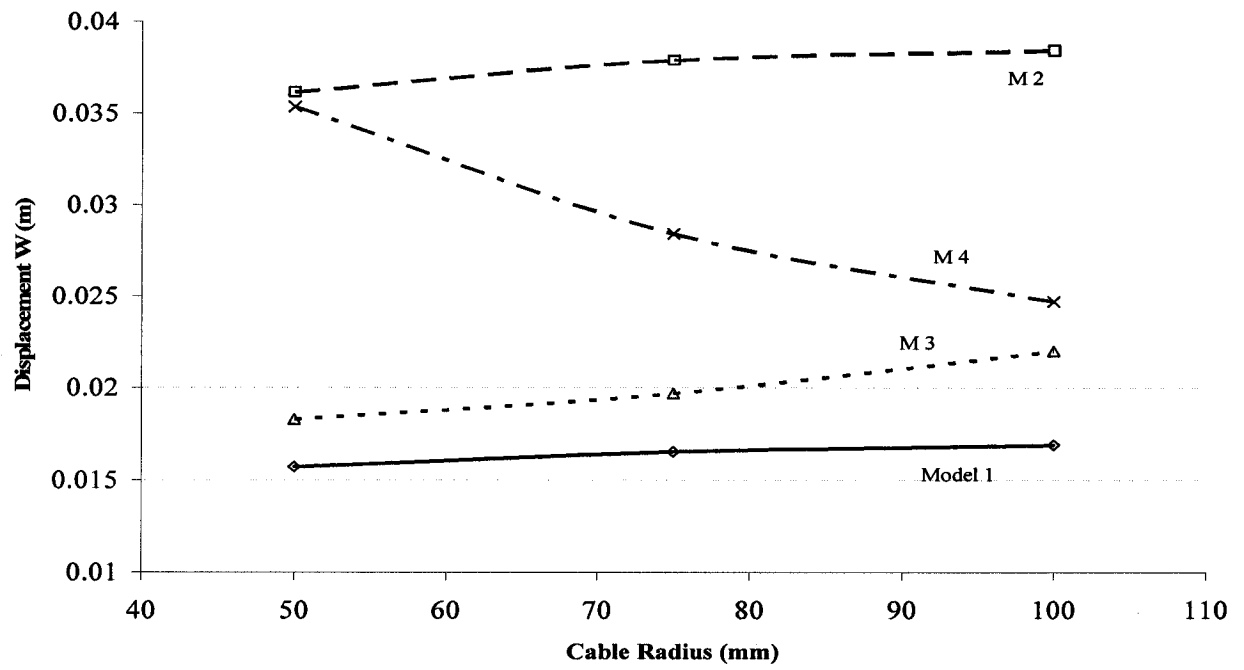


Figure 6.3.1.7 CFRP Cable Radius Effect on Longitudinal Displacement (W)– LDFA-a, Tha, LLA, Annacis Island Bridge

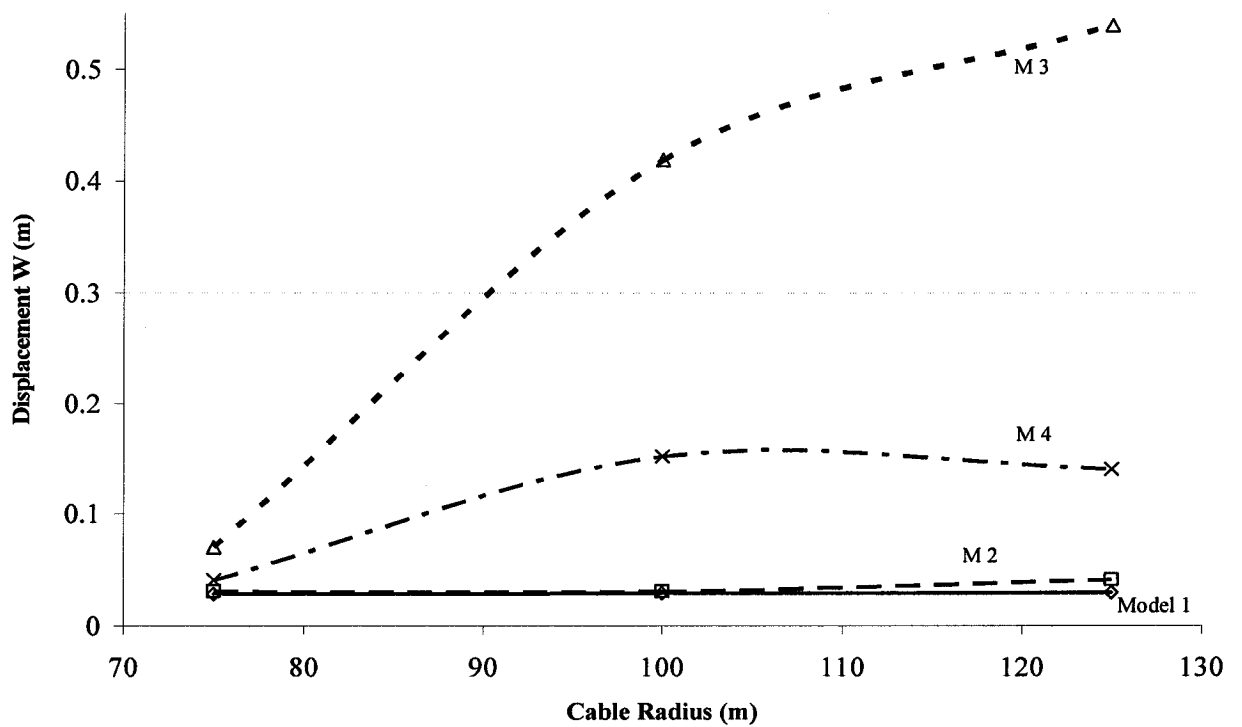


Figure 6.3.1.8 CFRP Cable Radius Effect on Longitudinal Displacement (W) – LDFA-a, Tha, LLA, Second Chang River Bridge

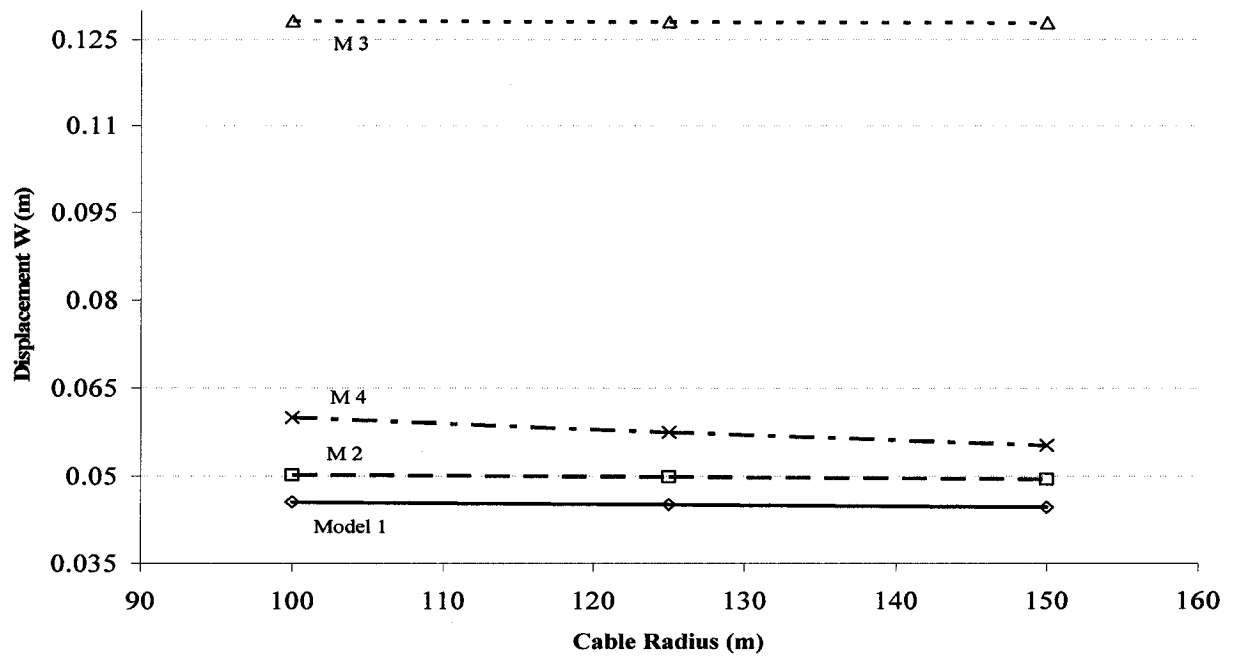


Figure 6.3.1.9 CFRP Cable Radius Effect on Longitudinal Displacement (W) – LDFA-a, Tha, LLA, Tatara Bridge

### 6.3.2 Effect of Cable Radius on the Deck Maximum Tsai-Hill Failure Function of the Laminates, Principal Stresses of the Chopped FRP and Tensile Stresses of the Cable Stays:

It has been found that a general large decrease is evident in the maximum Tsai-Hill Failure Function in all laminates of the laminated box girders of Models I, II and III and the tubular and stiffening laminates of Model IV, for all bridge spans. The increase in the cable diameter results in a decrease of the stresses in the deck: (i) the general huge reduction of the deck deflection leads to a redistribution of the stresses in the deck different laminates and results in a decrease of the level of the maximum Tsai Hill Failure Function, (ii) a reduction of the concentrated stresses around the points of cable anchorage to the deck due to the decrease in the cable tensile stresses, (iii) cables tensile forces are increased although the cable tensile stresses are decreased, which results in a slight increase of the axial forces in the deck that works against the high reduction of the stresses due to their redistribution. On the other hand, each model characteristic, stiffness and weight per unit length give different behavior and different distributions of the Tsai



Hill Failure Functions. Models II and IV have less flexural stiffness, and they show higher change with medium spans. Model III has high mass but less vertical flexural stiffness to mass ratio than Model I and shows a higher change in its THFF with the longest central span.

The top laminates of the box girders of Models I, II and III, together with the stiffening laminate of Model IV, or laminates B, show a very similar reduction pattern of their maximum THFF (Figures 6.3.2.1 to 6.3.3.3) to the reduction pattern of the deck maximum deflection (Figures 6.3.1.4 to 6.3.1.6). For example, as the deflection of Model IV reduces greatly with the Second Chang River Bridge, also THFF of the stiffening laminate shows very similar high reduction. As explained earlier, the major stresses in the deck are due to the vertical flexure (in the Y-Z plane). The increase of the cable radius changes the redistribution of those stresses and reduces the peak value of the maximum regions. This behavior is analogous to the traditional cable-stayed bridge system where the reduction of the maximum deck bending stresses is observed as a function of the increase of the stay-system cables axial stiffness [Troitsky, 1988, and Walther, et al 1999].

The bottom laminate of Models I, II and III and the right tubular laminate of Model IV, or laminates C, show similar reduction patterns in their maximum THFF (Figures 6.3.2.4 to 6.3.3.6) to that of Laminate B, for the same reasons. As shown earlier, the Model III bottom laminate has critical regions of THFF, where small cylindrical sectors connect the horizontal part to the inclined part of the laminate. The stresses in those regions are variable in the deck part located between the deck external supports. When the bridge central span is the longest, THFF decreases with cable radius increase, to the medium examined level, then it increases suddenly with further increase of the cable radius. This reflects the sensitivity of those regions, as in the previous cases. On the other hand, the relation of the maximum THFF to the cable radius increase of Model IV tubular laminates reflects the reduced rate of decrease of this model deflection and flexural stresses, with the Second Chang River Bridge.

The vertical laminates of Models I, II and III and the left tubular laminate of Model IV, or laminates D, also show a reduction in their maximum THFF (Figures 6.3.2.7 to 6.3.3.9). Model IV left tubular laminate gives the same THFF results as those of the right tubular laminate and will not be discussed here. Model I vertical laminates maximum THFF

show almost a constant level with negligible changes for all bridges. The vertical laminates of this model has constant height, and the neutral plane of the deck pass through them. While the change of the flexural stresses due to the redistribution is high in the top and bottom laminates, the changes are small in the vertical flat laminates of constant height. In contrast, the cylindrical laminates of Model III give a relatively high decrease of their THFF although they have constant height. The decrease of the bending stresses greatly reduces the axial stresses, which result from the bending-extension coupling, and hence it gives the shown reduction of THFF. On the other hand, the critical low-height laminates of Model II switch their stresses in some regions located between the deck supports. The critical stresses of those regions are affected by the central span/side span ratio (or span ratio) and the distance between the pylon and the intermediate external support. They switch their stresses with the Second Chang River Bridge, which has the minimum span ratio and the maximum pylon/intermediate support distance. The redistribution in the stresses, resulting from the increase of the cable radius, causes the shown reduction in the very high THFC of those regions. However, the stresses in those critical vertical laminates of Model II appear not having a stress switch problem with the Annacis Island and the Tatara Bridges, for their high span ratios and their different deck support configuration. Consequently, Laminates D of Model II shows very slight change in THFF.

A general reduction of the principal stresses in the chopped FRP deck slab of models I, II, and III and the Chopped part of Model IV, with the increase of CCR, is also evident. Figure 6.3.3.10 show the typical results of such reduction. The decrease of the deck maximum flexural strains is parallel to the decrease of the deck deformations due to the increase of the cable radius, as the deck materials moduli are constant. The chopped FRP is considered in this research as a linear elastic isotropic material and its uncoupled stresses are proportional to the strains. Hence the general reduction of the strains in the Chopped FRP part of the deck for different models results in a general reduction of their stresses.

On the other hand, the stresses in the stay cables themselves are greatly reduced as CCR increases (Figures 6.3.3.11 to 6.3.3.13). This increase is expected as the cable cross sectional area is increased. However the resultant tensile forces in the stay cables are increased with the increase of the cable radius (Figures 6.3.3.14 to 6.3.3.16). The increase

of the cable stiffness leads to a redistribution of the contributions of the stay-system and the deck, in the overall bridge system. Due to their very high stiffness to weight ratio, the CFRP cable radius increase results in a large increase of their restraining capability. Hence they “absorb” more forces from the deck, reducing its flexure and increasing the bridge stiffness.

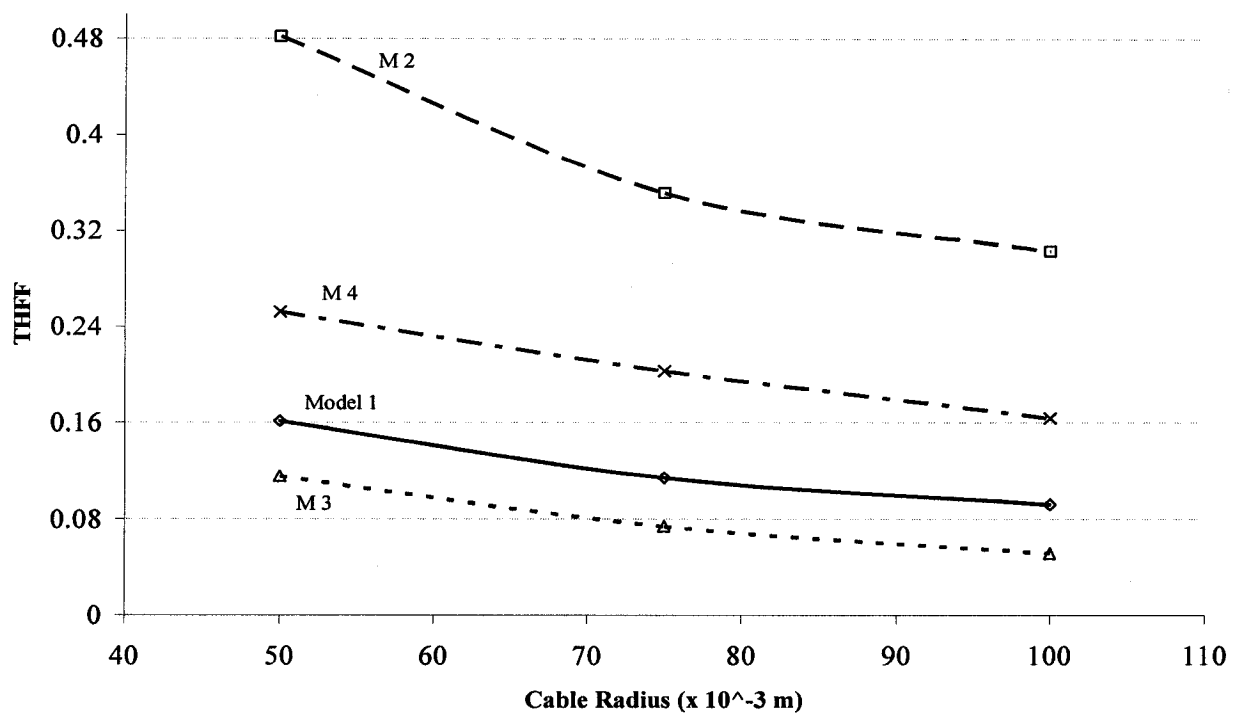


Figure 6.3.2.1 CFRP Cable Radius Effect on Tsai-Hill - Laminate B- LDFA-a, Tha, LLA, Annacis Island Bridge

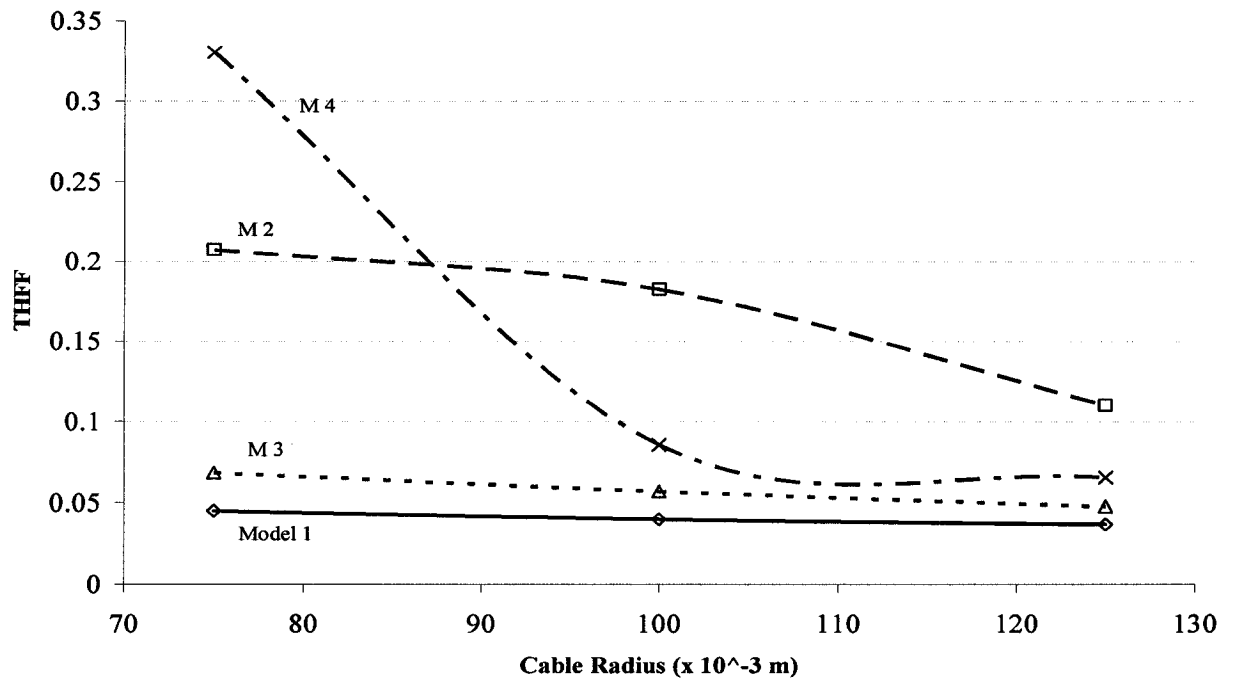


Figure 6.3.2.2 CFRP Cable Radius Effect on THFF – Laminate B – LDFA-a, Tha, LLA, Second Chang River Bridge

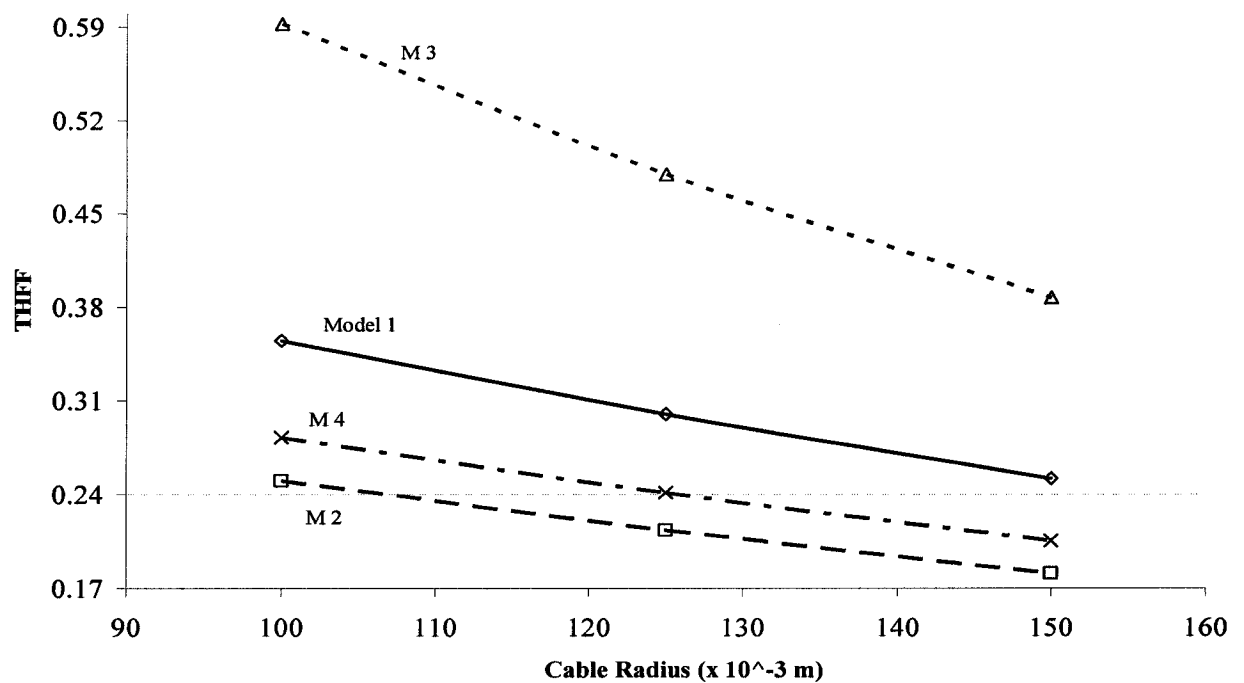


Figure 6.3.2.3 CFRP Cable Radius Effect on THFF – Laminate B – LDFA-a, Tha, LLA, Tatara Bridge

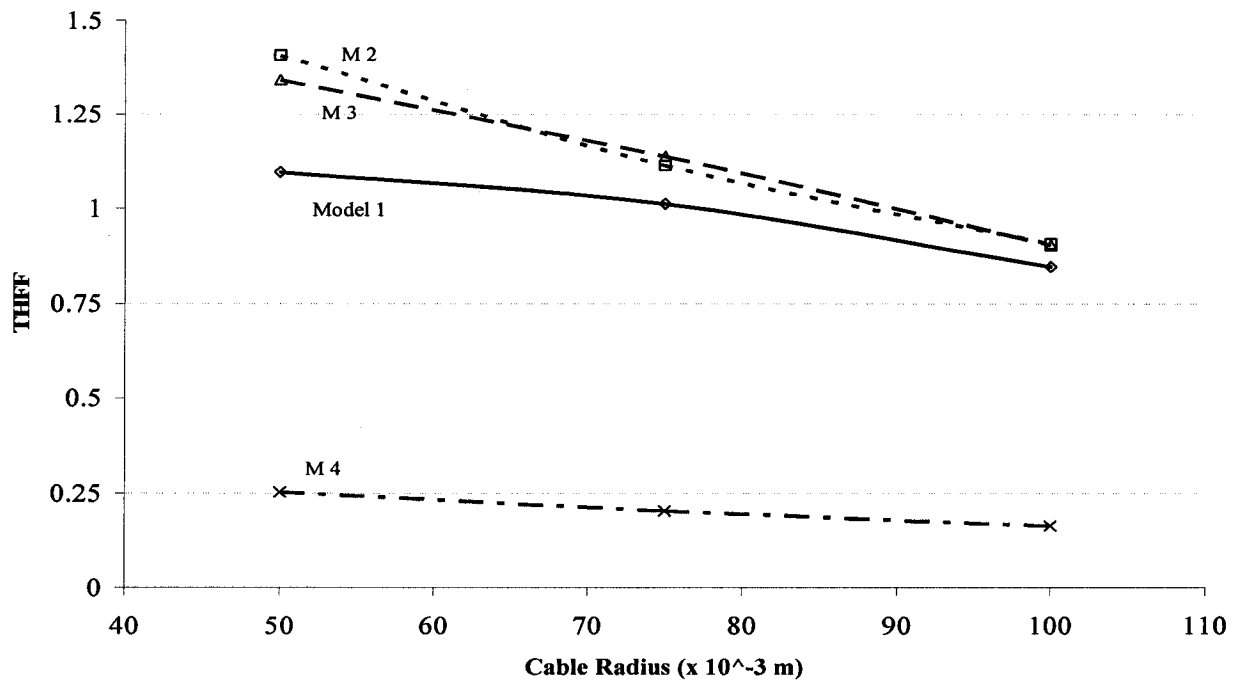


Figure 6.3.2.4 CFRP Cable Radius Effect on Tsai-Hill - Laminate C – LDFA-a, Tha, LLA, Annacis Island Bridge

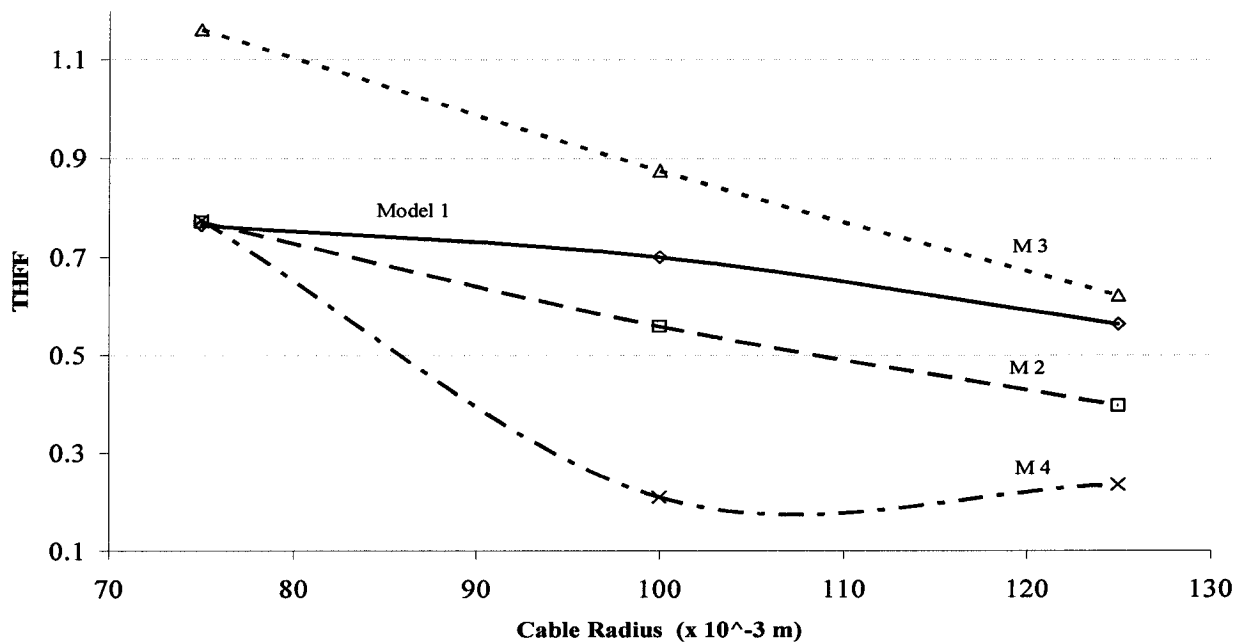


Figure 6.3.2.5 CFRP Cable Radius Effect on THFF – Laminate C – LDFA-a, Tha, LLA, Second Chang River Bridge

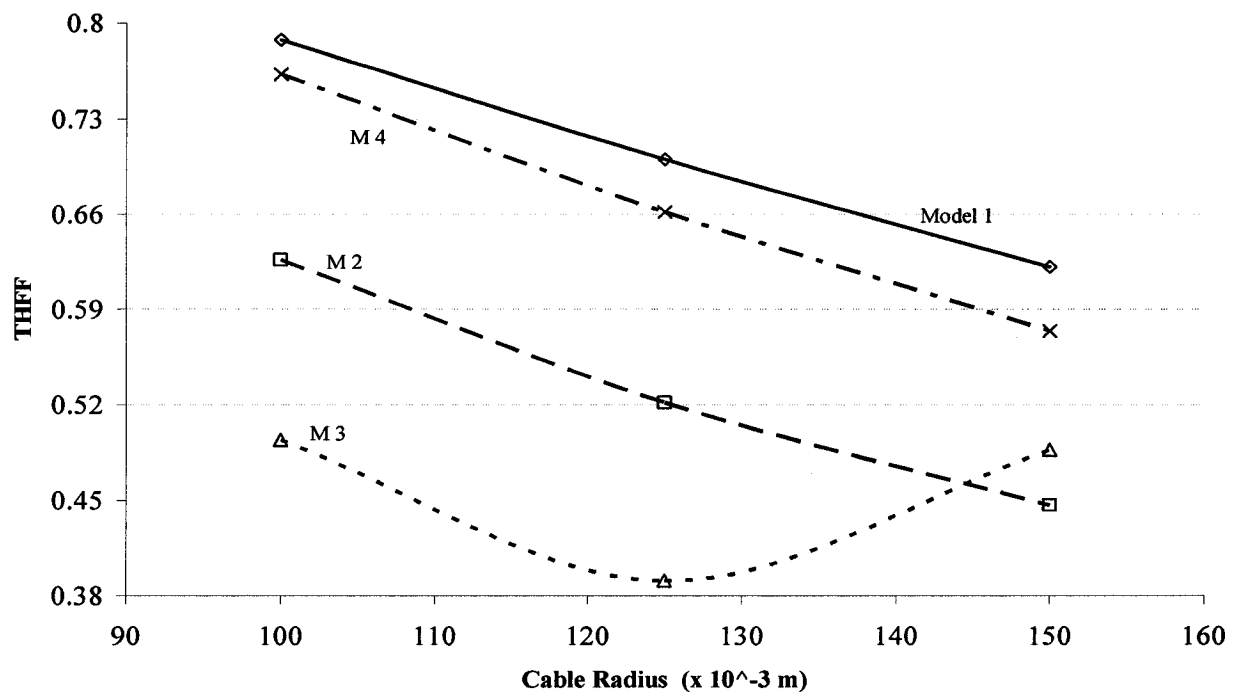


Figure 6.3.2.6 CFRP Cable Radius Effect on THFF – Laminate C – LDFA-a, Tha, LLA, Tatara Bridge

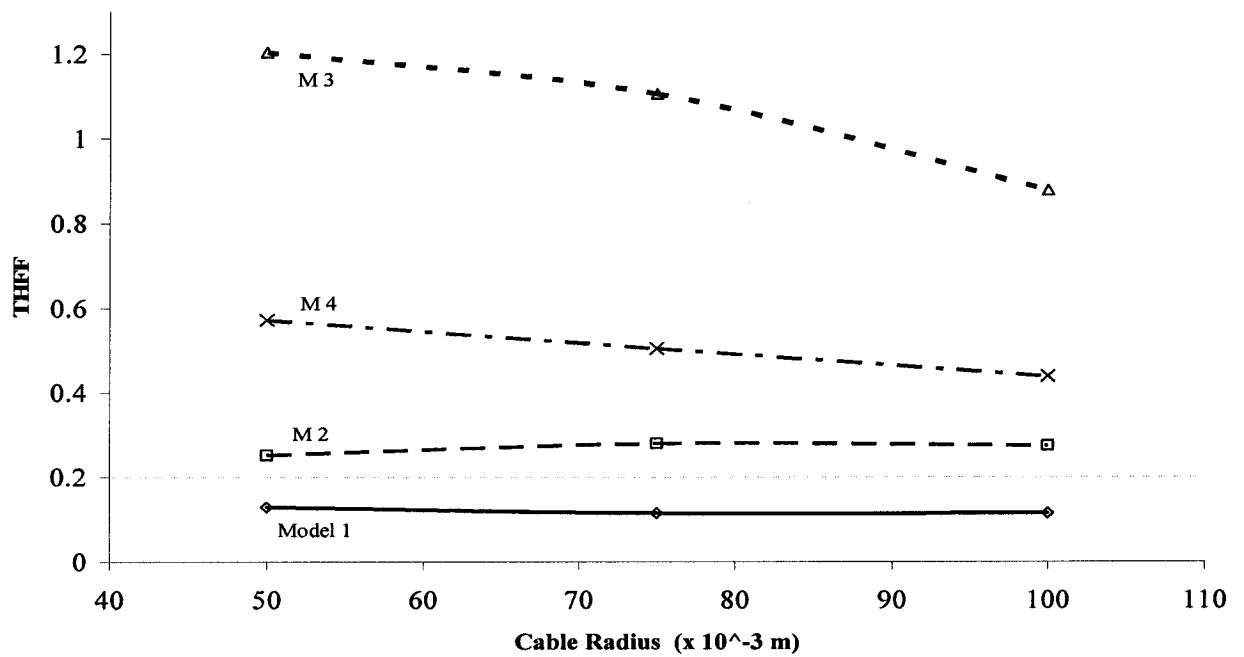


Figure 6.3.2.7 CFRP Cable Radius Effect on Tsai-Hill - Laminate D – LDFA-a, Tha, LLA, Annacis Island Bridge

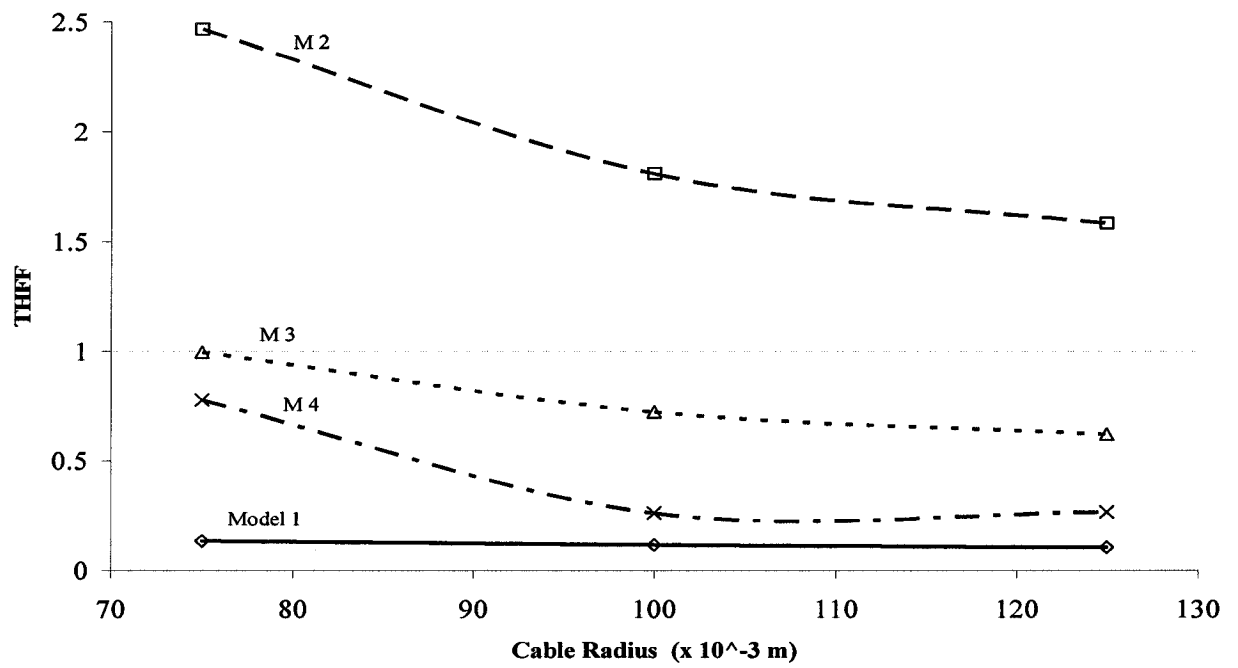


Figure 6.3.2.8 CFRP Cable Radius Effect on THFF – Laminate D – LDFA-a, Tha, LLA, Second Chang River Bridge

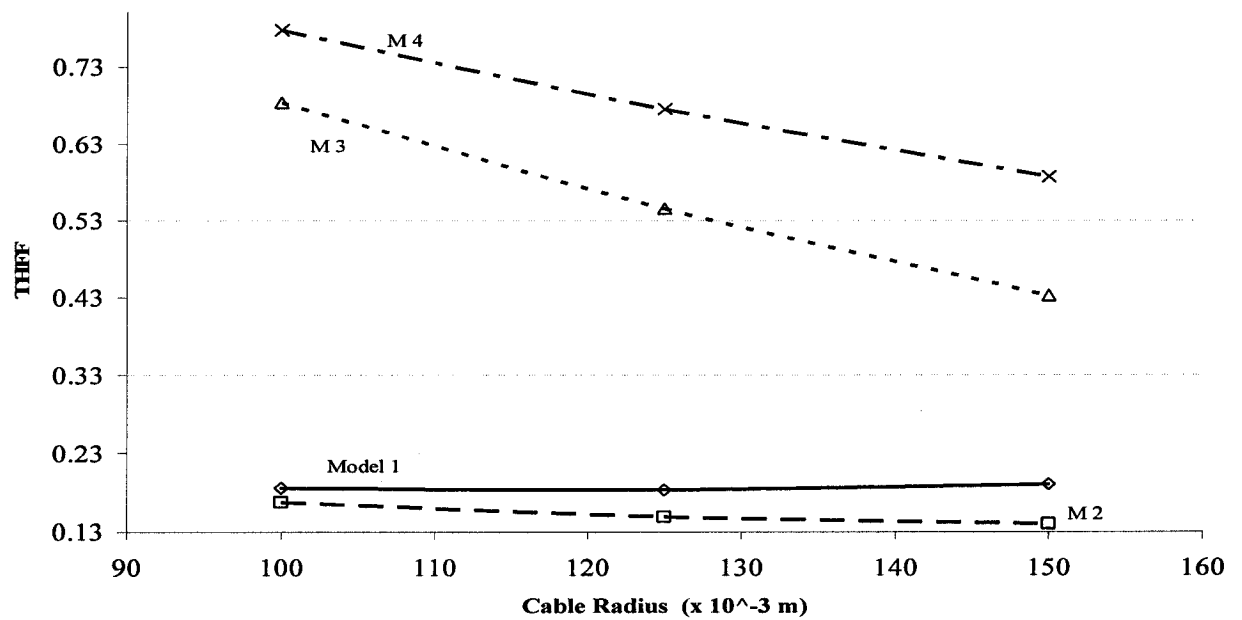


Figure 6.3.2.9 CFRP Cable Radius Effect on THFF – Laminate D – LDFA-a, Tha, LLA, Tatara Bridge

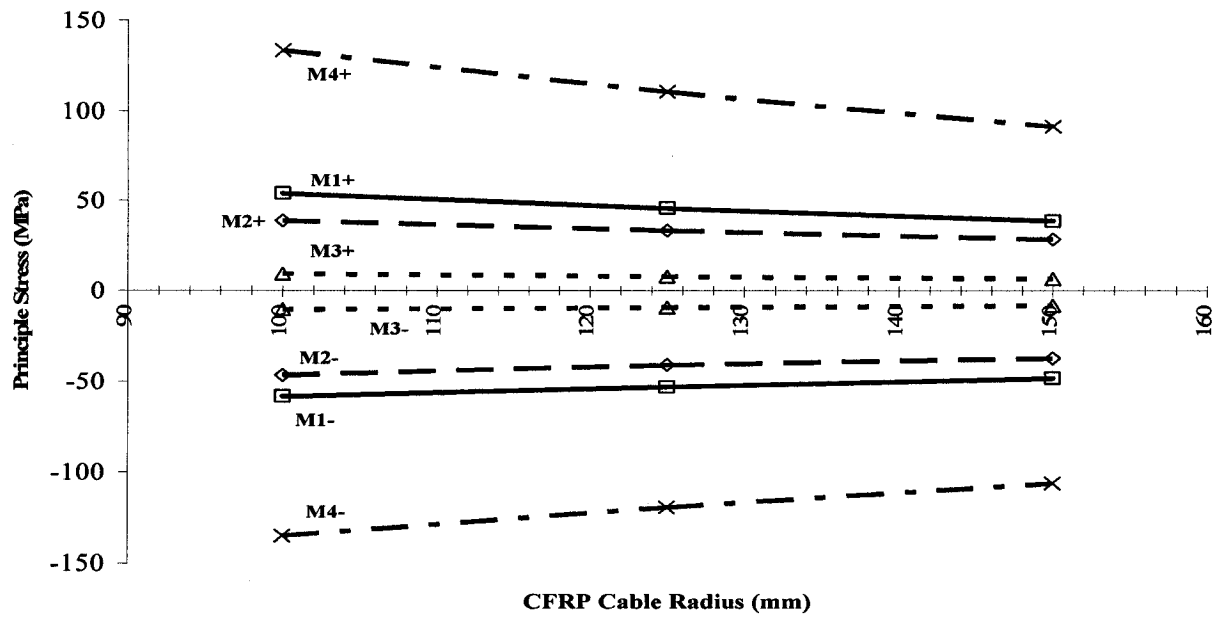


Figure 6.3.2.10 CFRP Cable Radius Effect on Principal Stresses at the Chopped FRP Deck Slab for Models I, II and III and the deck section for Model IV- LDFA-a, Tha, Tatara Bridge

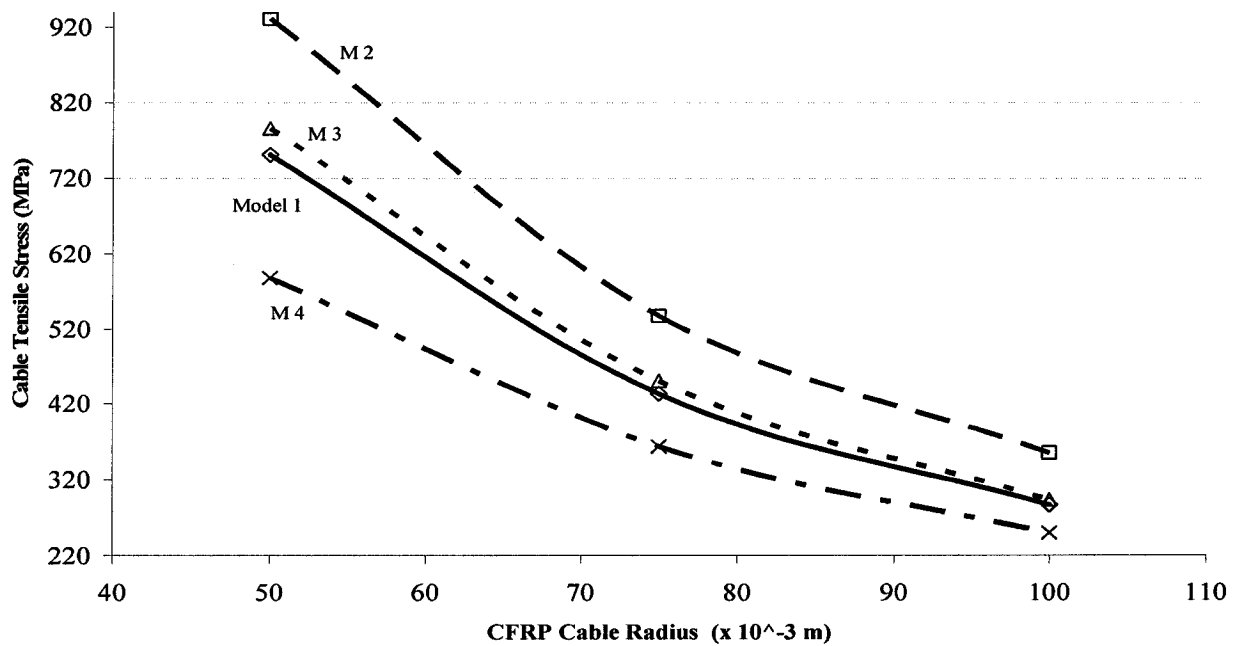


Figure 6.3.2.11 CFRP Cable Radius Effect on Maximum Cable Tension for different models, LDFA-a, Tha, Annacis Island Bridge



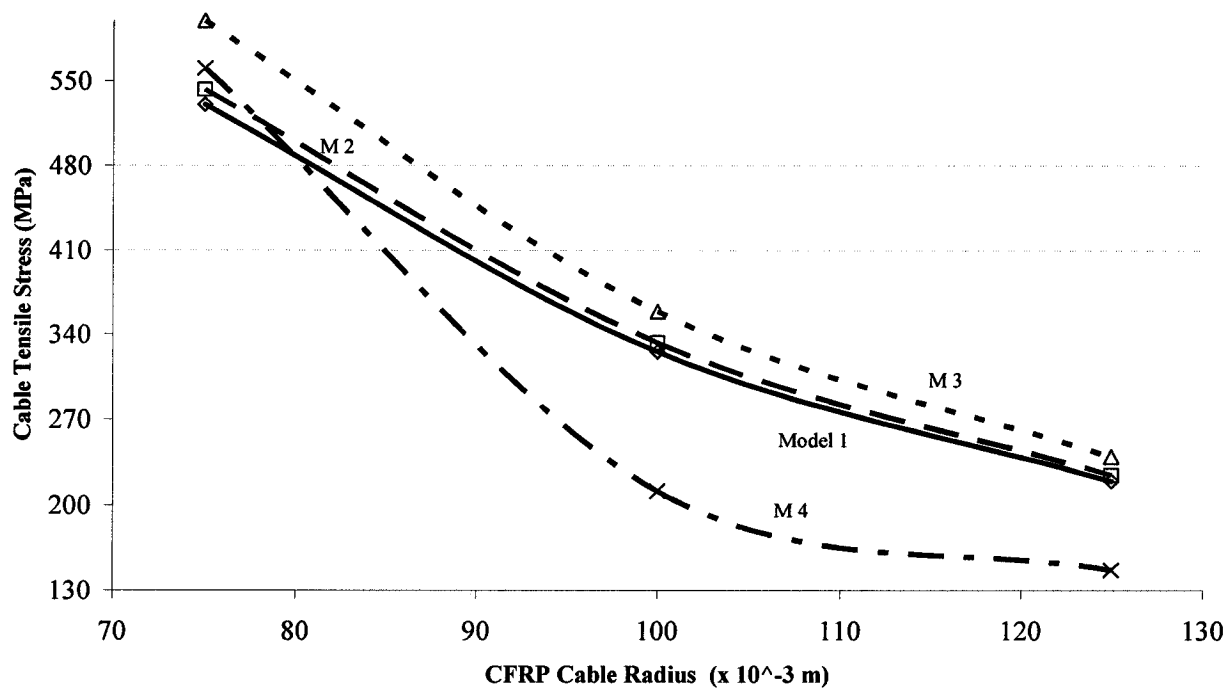


Figure 6.3.2.12 CFRP Cable Radius Effect on Maximum Cable Tension for different models, LDFA-a, Tha, Second Chang River Bridge

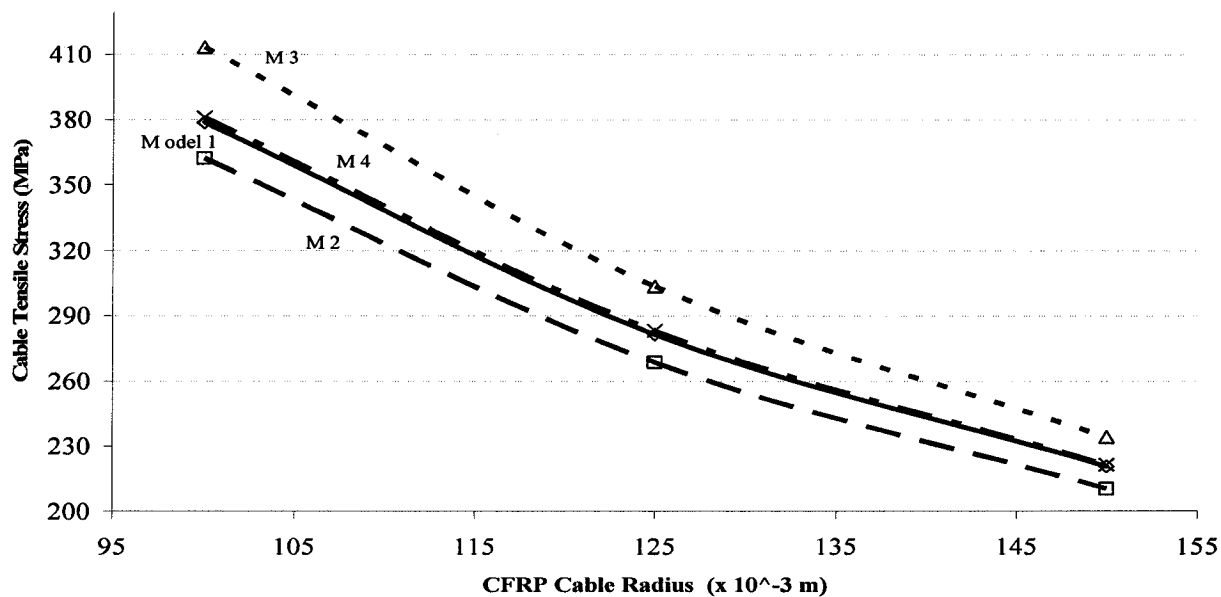


Figure 6.3.2.13 CFRP Cable Radius Effect on Maximum Cable Tension for different models, LDFA-a, Tha, Tatara Bridge

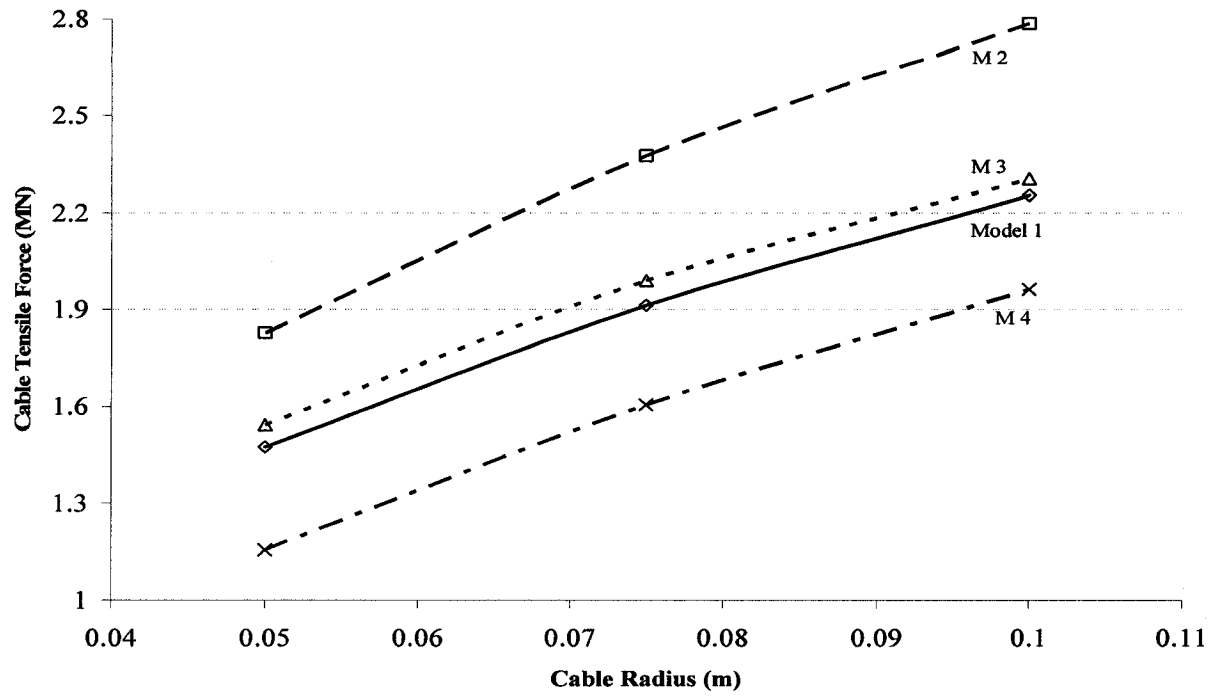


Figure 6.3.2.14 CFRP Cable Radius Effect on Maximum Cable Tensile Force for different models, LDFA-a, Tha, Annacis Island Bridge

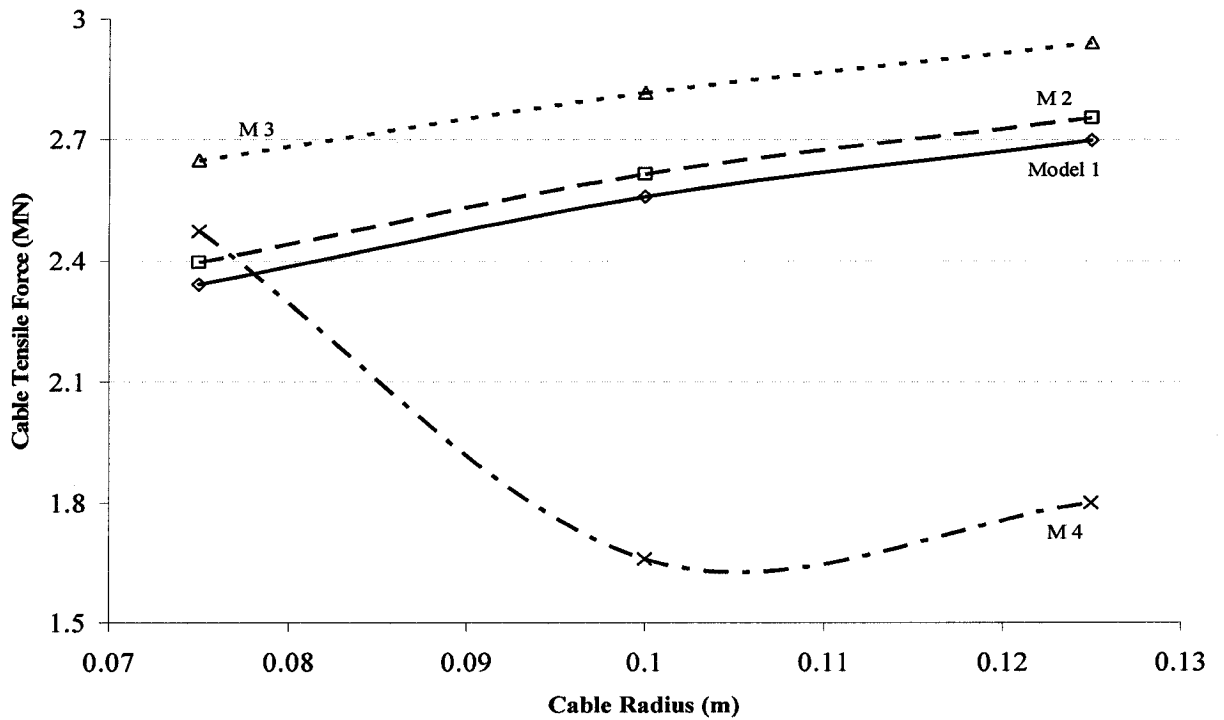


Figure 6.3.2.15 CFRP Cable Radius Effect on Maximum Cable Tensile Force for different models, LDFA-a, Tha, Second Chang River Bridge

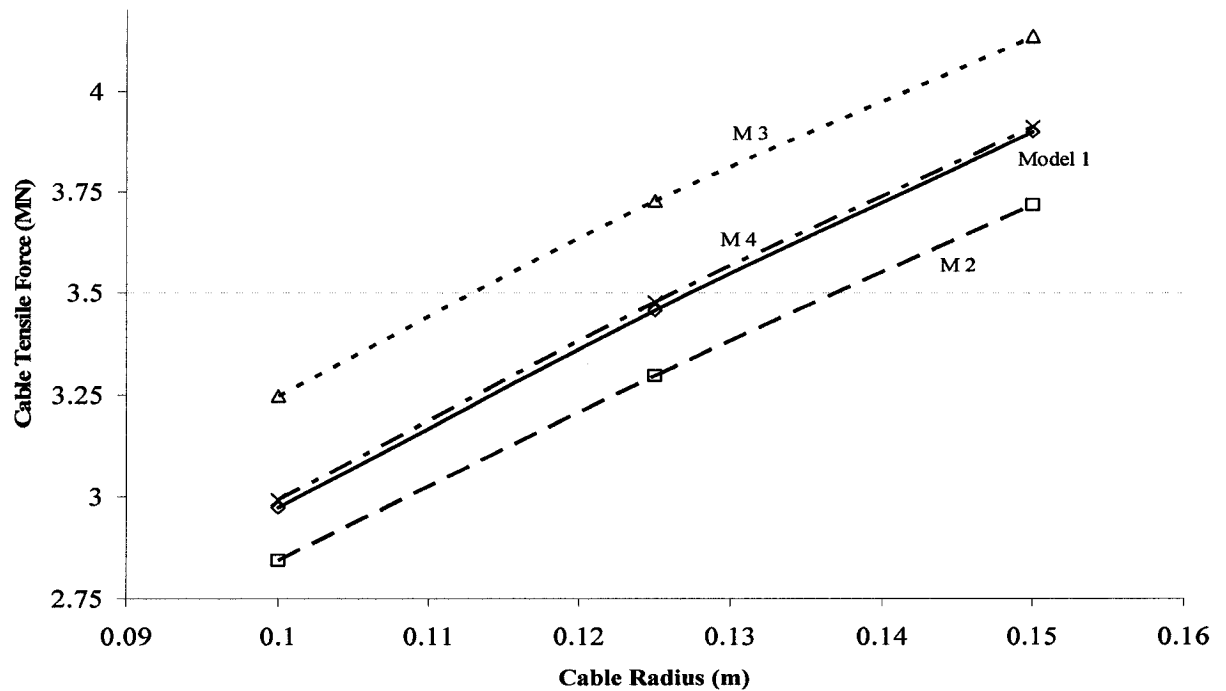


Figure 6.3.2.16 CFRP Cable Radius Effect on Maximum Cable Tensile Force for different models, LDFA-a, Tha, Tatar Bridge

### 6.3.3 Effect of Cable Radius on the Vertical Natural Frequencies:

It has been found that all Vertical Natural Frequencies (VNF) increase at high rates as the CFRP Cable Radius (CCR) increases. In some cases, the increase of VNF could reach 90% of the VNF related to the minimum examined radius. Typical results are shown in Figures 6.3.3.1 to 3 and Figures 6.3.3.5 and 6, which are very similar to the un-shown results. Two irregular cases do not follow the general reduction rule: (i) Model III second VNF decreases then increases as CCR increases with the shortest examined bridge central span (Figure 6.3.3.4), and (ii) Model IV's third, fourth and fifth VNF decrease then rise to a higher level than the start level, as CCR increases, with the longest examined central span (Figure 6.3.3.7 to Figure 6.3.3.9).

Recalling the explanations in the previous sections, CFRP cables have very high stiffness to mass ratios that lead to improve the static and the dynamic characteristics of cable stayed bridge system. The increase of the stay cables radius would (i) increase the efficiency of stay system elastic-support in the central as well as the side spans, by increasing the deck restraint, and (ii) highly increase the bridge system stiffness, although

a minimum mass is added. As a result the increase of the elastic restraint increases the natural frequencies. Increasing the ratio between the stiffness to the mass, also directly leads to increase the natural frequencies of the system. This clarify the general increase in most of the cases. However, the large effect of the bending-extension coupling for the cylindrical laminates works in opposite to the bridge system stiffness increase. This effect is small and only becomes apparent at some high frequencies. This explains the reduction then increase of the high frequencies of Model IV as it has a small stiffness to mass ratio with the Tataara Bridge. Model III also shows similar behavior only with the second VNF with the Annacis Island Bridge because of the many cylindrical laminates in this model and the high restraining action, resulting in a relatively high coupling effect.

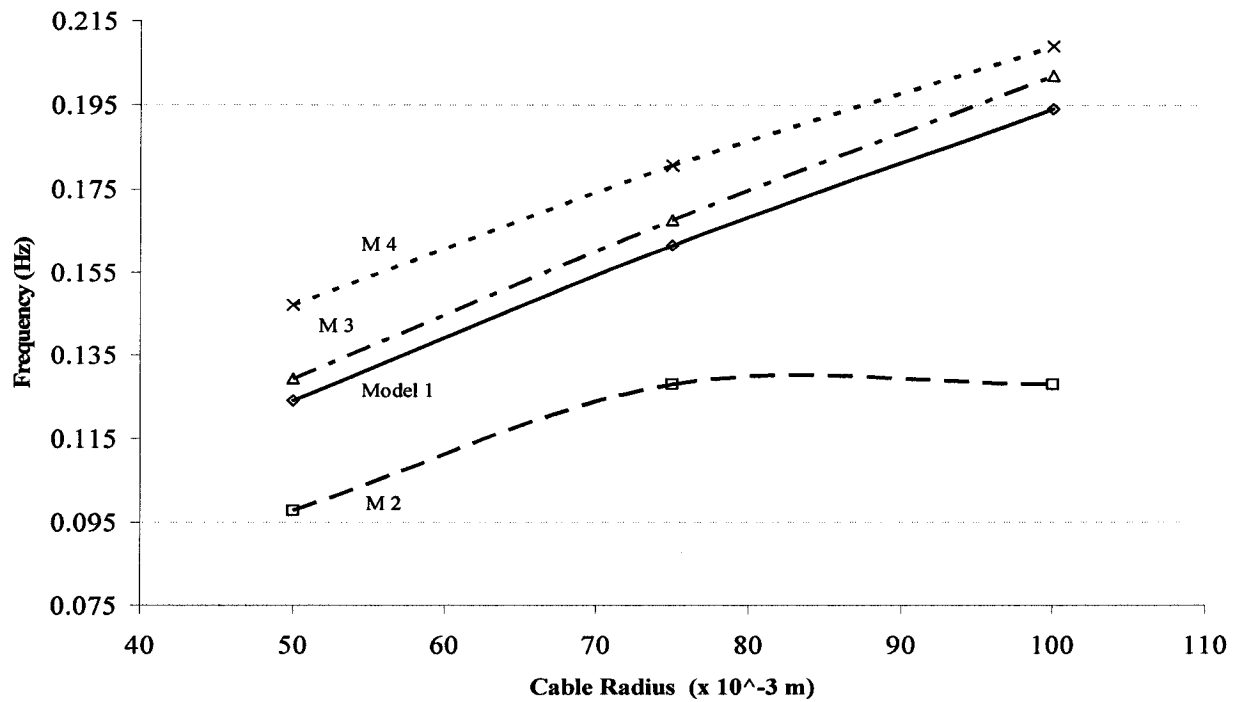


Figure 6.3.3.1 CFRP Cable Radius Effect on the 1st Vertical Natural Frequencies, LDFA-a, Tha -Annacis Island Bridge

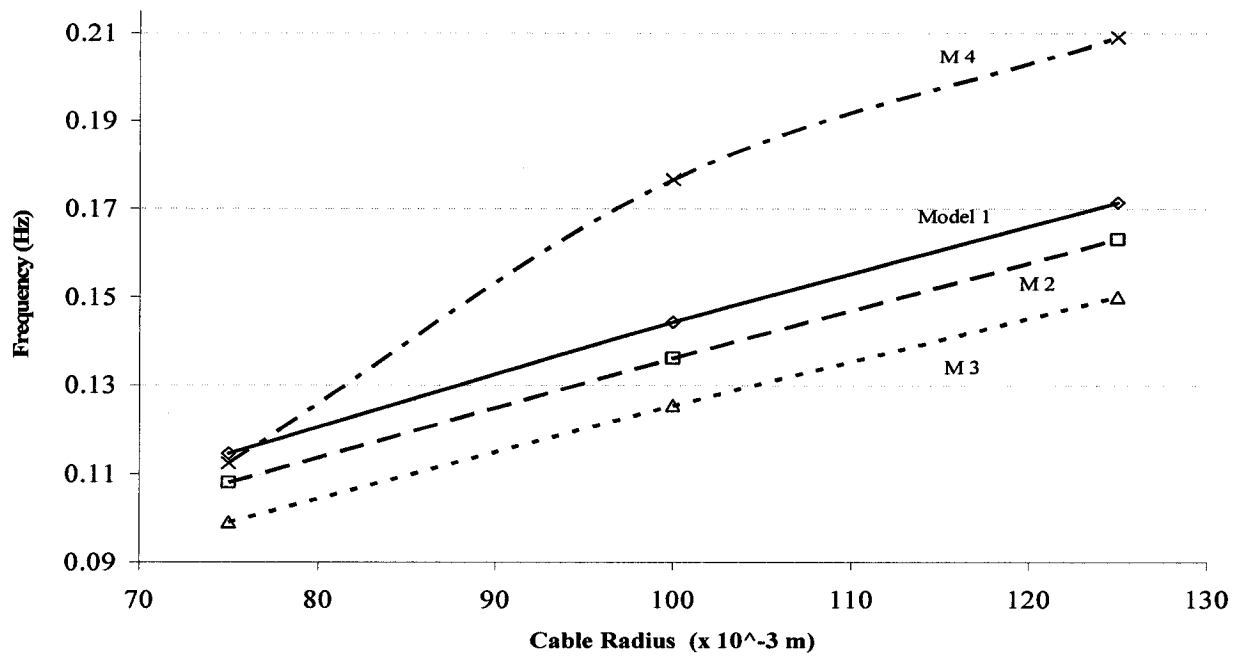


Figure 6.3.3.2 CFRP Cable Radius Effect on the 1st Vertical Natural Frequencies, LDFA-a, Tha - Second Chang River Bridge

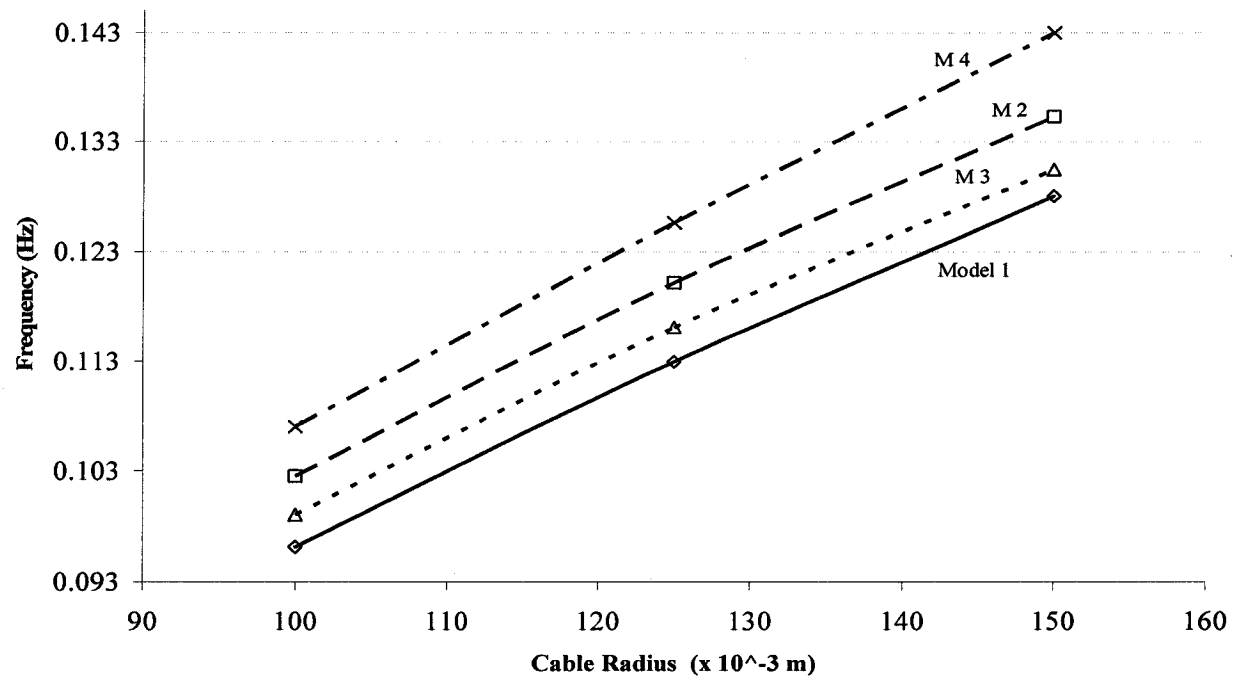


Figure 6.3.3.3 CFRP Cable Radius Effect on the 1st Vertical Natural Frequencies, LDFA-a, Tha - Tatara Bridge

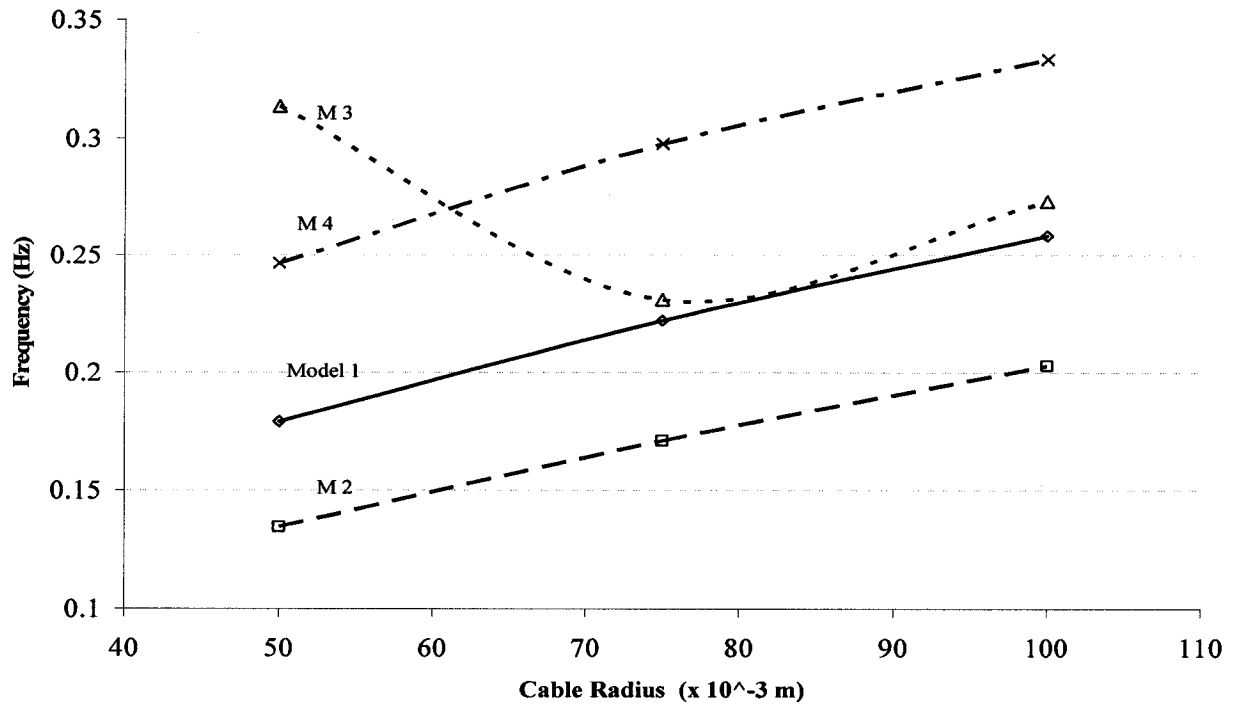


Figure 6.3.3.4 CFRP Cable Radius Effect on the 2nd Vertical Natural Frequencies, LDFA-a, Tha - Annacis Island Bridge

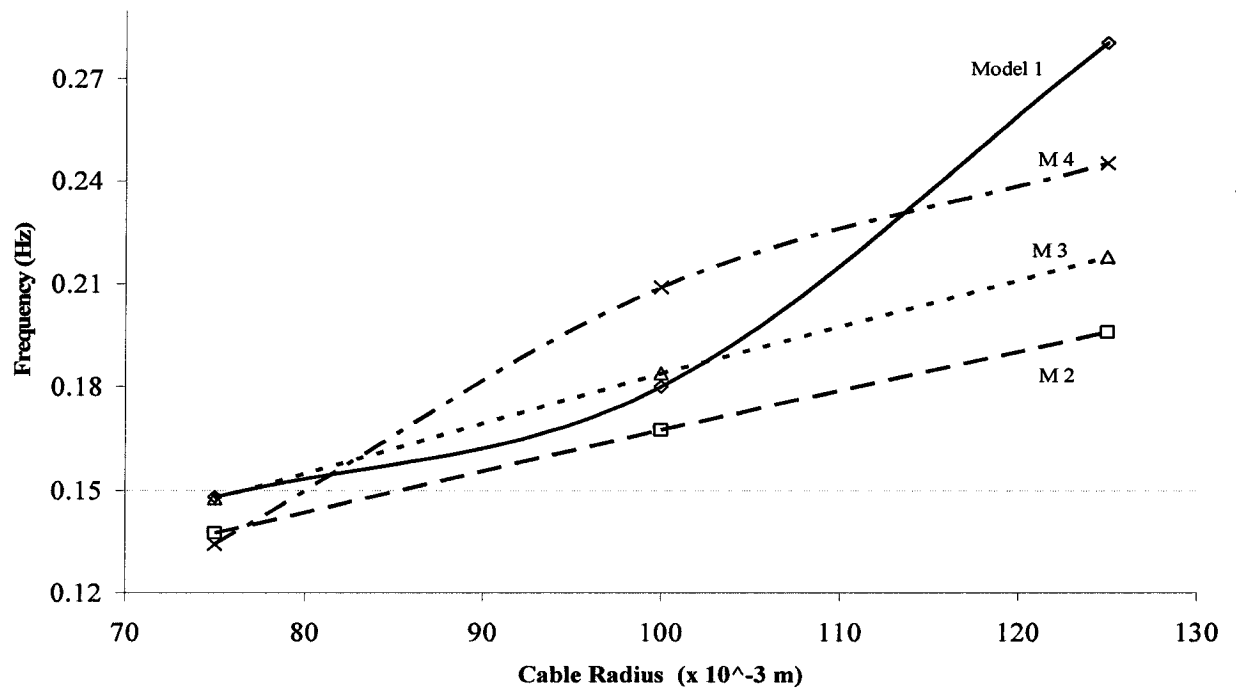


Figure 6.3.3.5 CFRP Cable Radius Effect on the 2nd Vertical Natural Frequencies, LDFA-a, Tha - Second Chang River Bridge

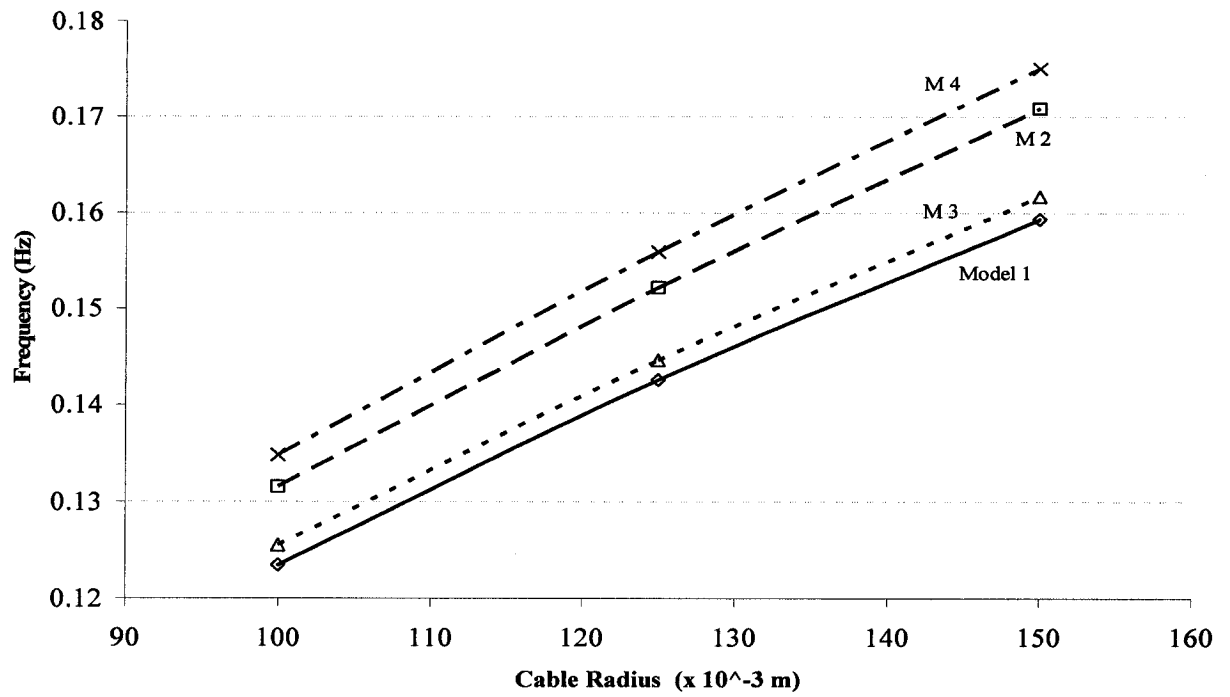


Figure 6.3.3.6 CFRP Cable Radius Effect on the 2nd Vertical Natural Frequencies, LDFA-a, Tha - Tatara Bridge

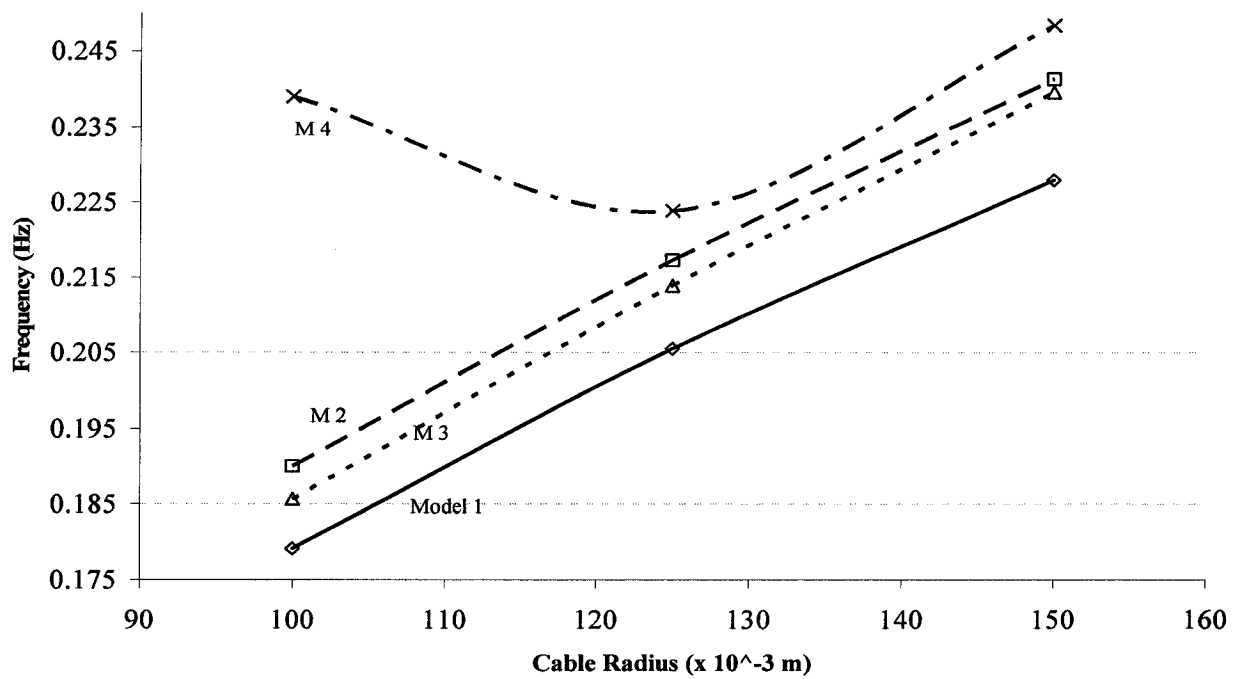


Figure 6.3.3.7 CFRP Cable Radius Effect on the 3rd Vertical Natural Frequencies, LDFA-a, Tha - Tatara Bridge

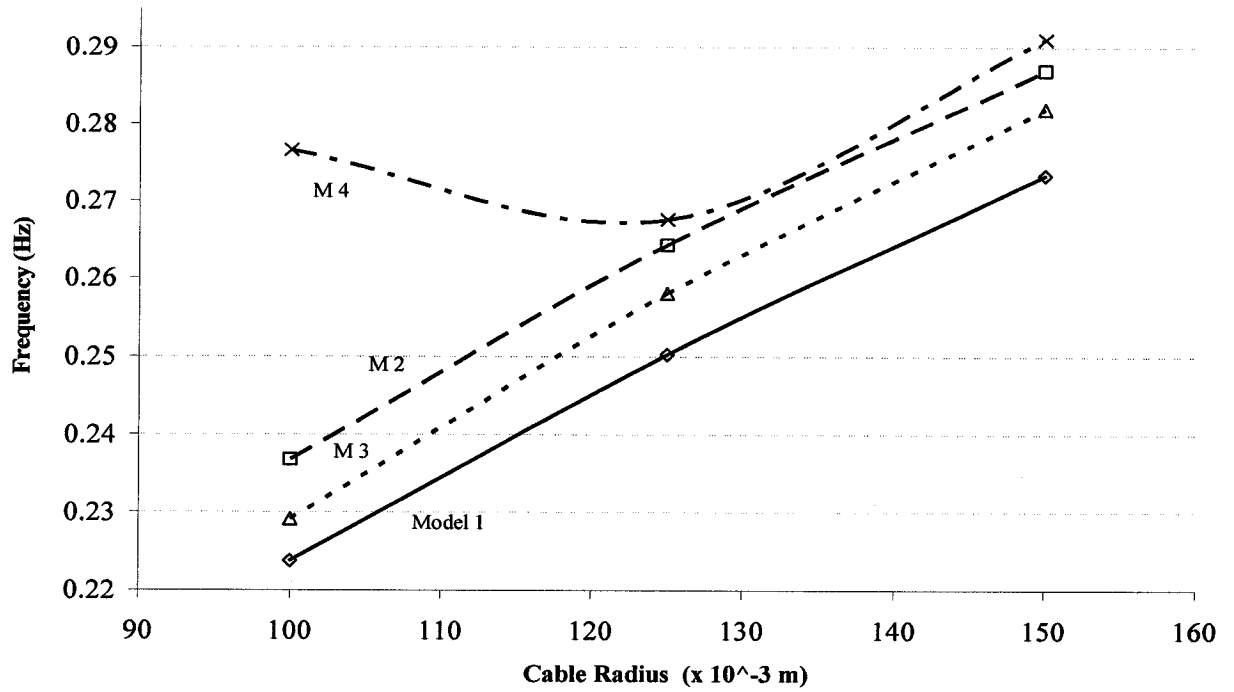


Figure 6.3.3.8 CFRP Cable Radius Effect on the 4th Vertical Natural Frequencies, LDFA-a, Tha - Tatara Bridge

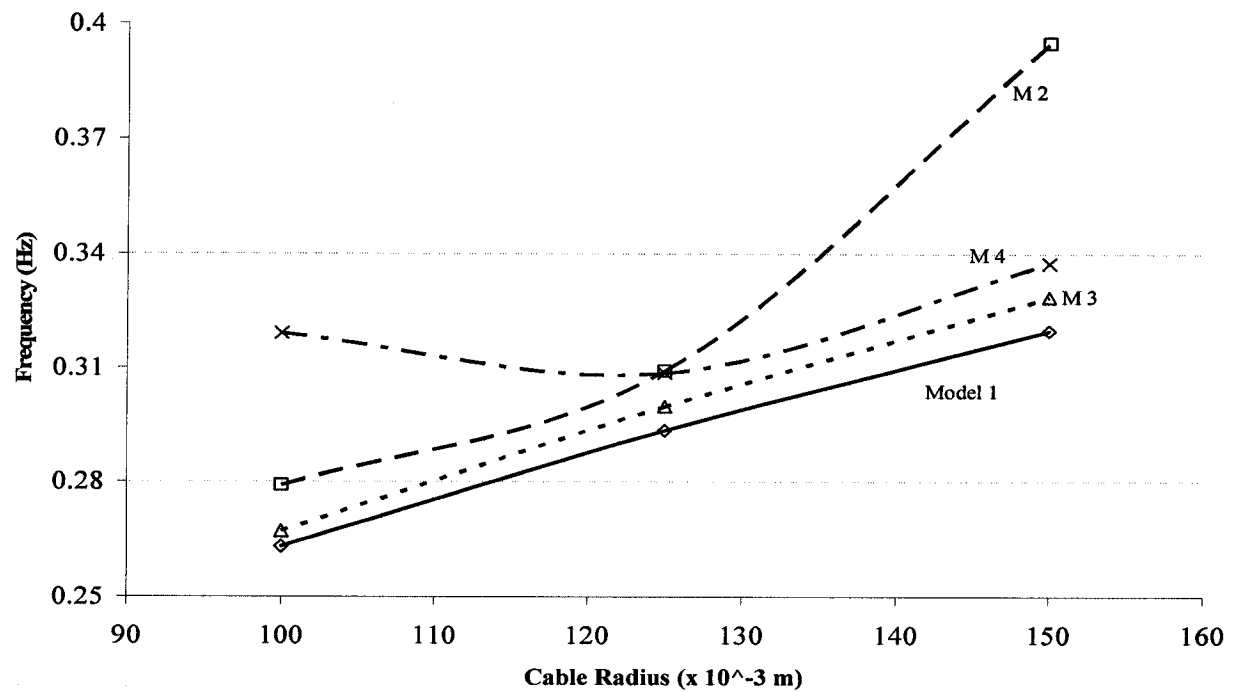


Figure 6.3.3.9 CFRP Cable Radius Effect on the 5th Vertical Natural Frequencies, LDFA-a, Tha - Tatara Bridge



#### **6.3.4 Effect of Cable Radius on the Torsional Natural Frequencies:**

Generally the Torsional Natural Frequencies (TNF) increase as the CFRP Cable Radius (CCR) increases, with some exceptions for Models II and III, where TNF either increases then decreases or decreases then increases. Whatever the case, the TNF related with the biggest CCR is higher than that of the minimum CCR and usually the fluctuations are related to the medium level of CCR. All the following figures (Figures 6.3.4.1 to 6.3.4.15) show the apparent increase for most of the studied cases.

Again the general increase of the torsional natural frequencies can be explained by the improvement of the elastic restraint provided by the stay cables and the increase of bridge system stiffness, with minimum increase in the system mass, as the CFRP cable radius increases. Model II gives an increase then a decrease in some of its TNF with the shortest span length (Figures 6.3.4.5, 8 and 11). This model has basically a low torsional stiffness as well as a low mass. With the shortest span, the improvement of the stiffness becomes ineffective, as the increase of the mass is relatively high due to the increase of the cable radius. However, as the bridge central span length increases, the stiffness to mass ratio of the Model II bridge system becomes high and is slightly affected by the increase of the mass due to the increase of the cable radius. On the other hand, Model III has high torsional stiffness but it also has relatively high mass. The influence of the bend-twist coupling is also effective in this model for its many inner cylindrical laminates. The increase of the bridge system mass due to the increase of CCR has no apparent influence on the torsional natural frequencies variations in the short and medium bridge central span lengths. For such cases, the significant enhancement of the stiffness is dominant, which results in successive increase in the TNFs. However, with the longest central span length, the system mass becomes high and the stiffness to mass ratio turns out to be very sensitive to the added mass, due to CCR increase and the reduction of the stiffness due to the coupling effect. Therefore, a critical balance is apparent and results in a decrease of TNF for this model with the longest central span length and the biggest cable radius (Figure 6.3.4.6, 9, 12 and 15).

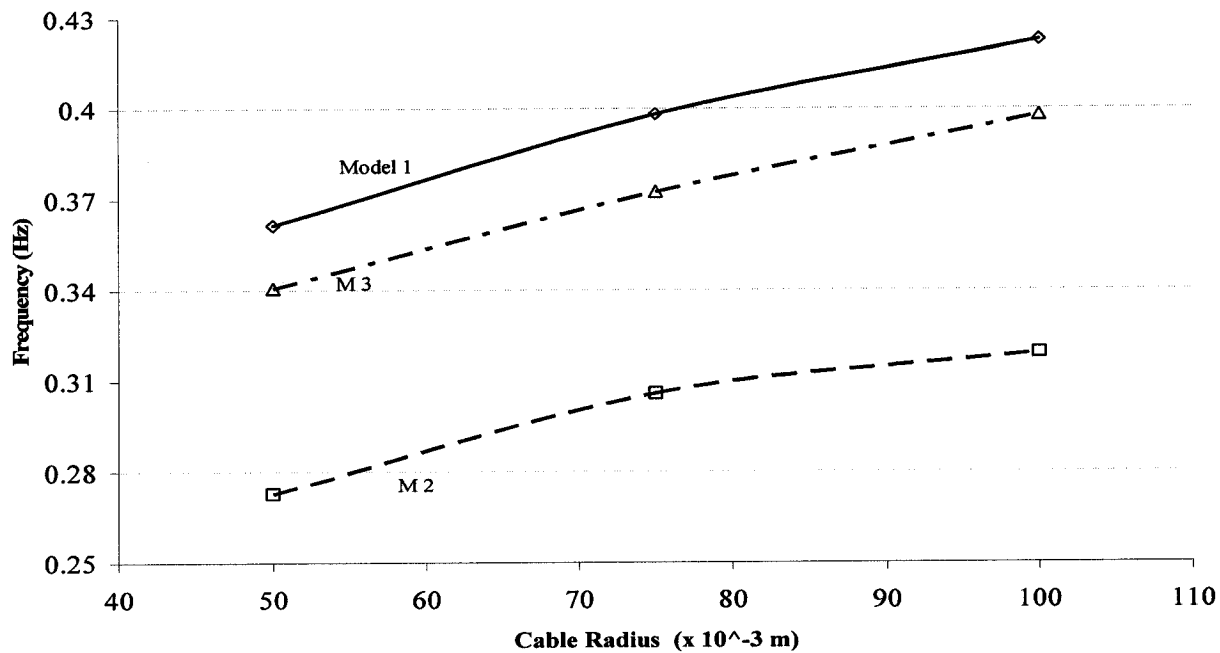


Figure 6.3.4.1 CFRP Cable Radius Effect on the 1st Torsional Natural Frequencies, LDFA-a, Tha - Annacis Island Bridge

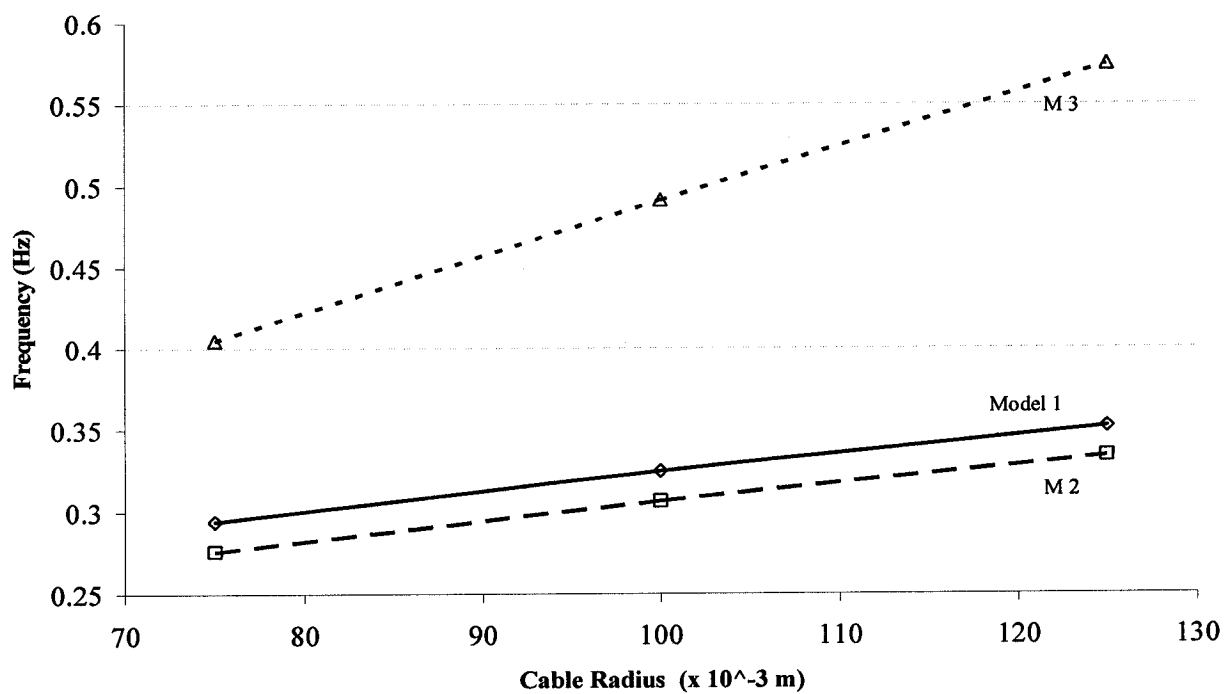


Figure 6.3.4.2 CFRP Cable Radius Effect on the 1st Torsional Natural Frequencies, LDFA-a, Tha - Second Chang River Bridge

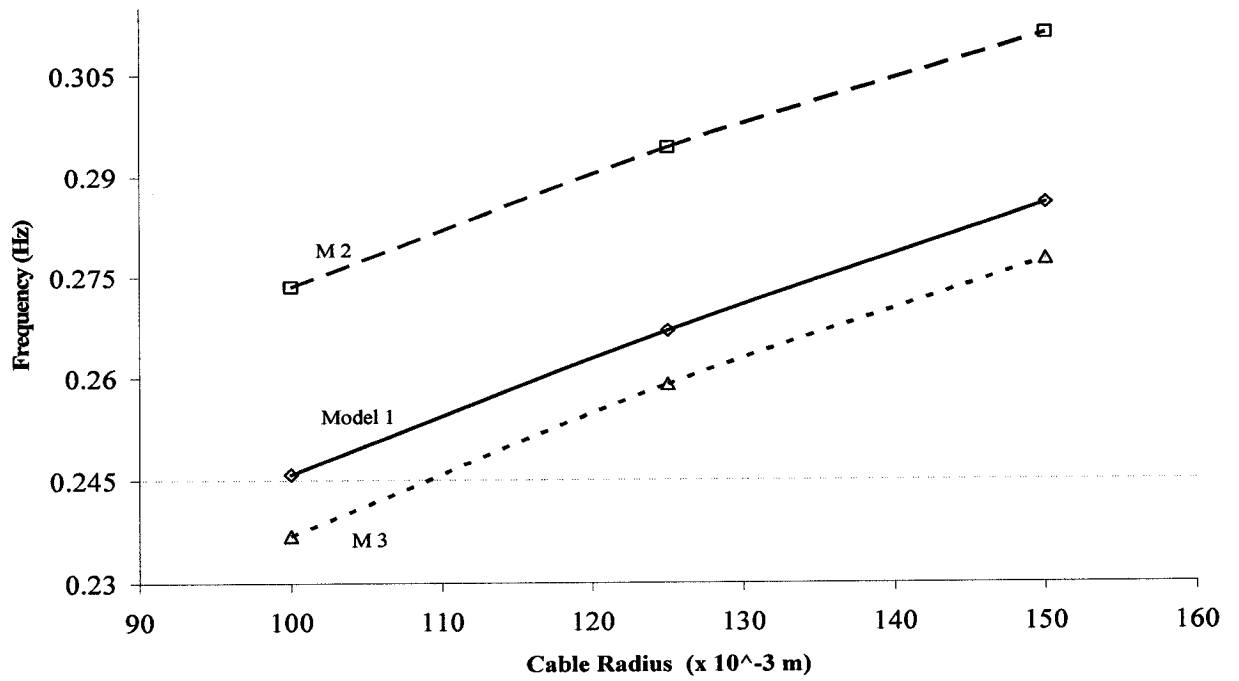


Figure 6.3.4.3 CFRP Cable Radius Effect on the 1st Torsional Natural Frequencies, LDFA-a, Tha - Tatara Bridge

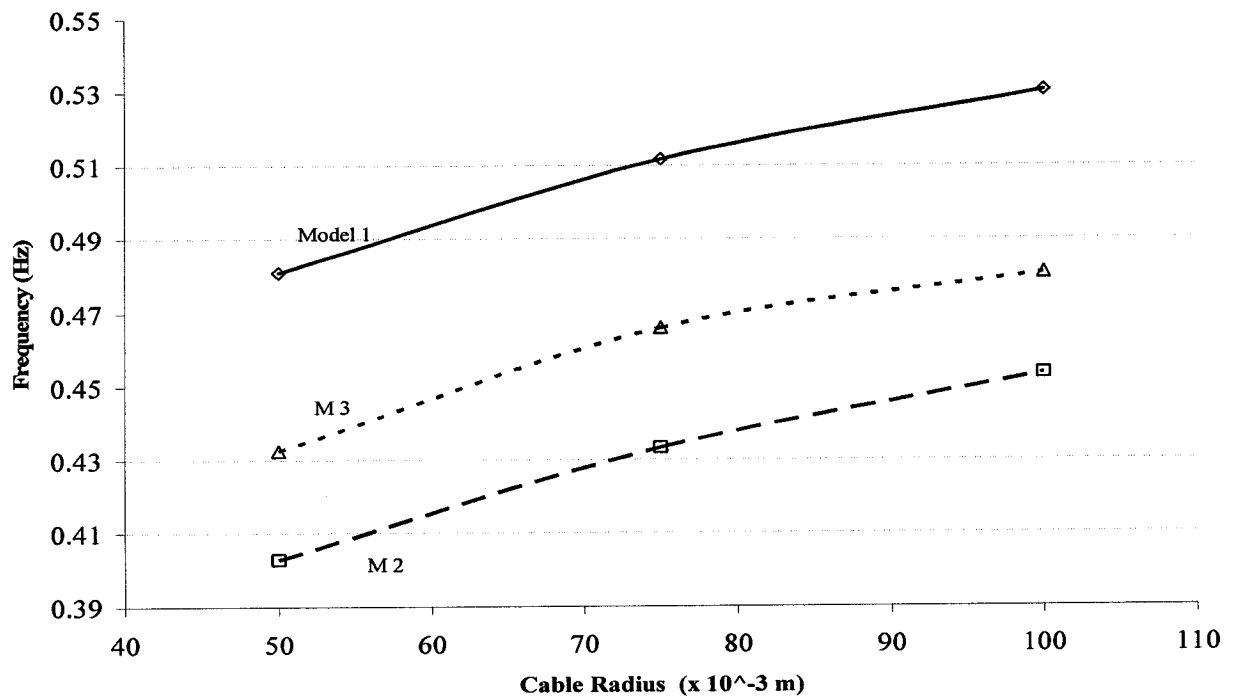


Figure 6.3.4.4 CFRP Cable Radius Effect on the 2nd Torsional Natural Frequencies, LDFA-a, Tha - Annacis Island Bridge

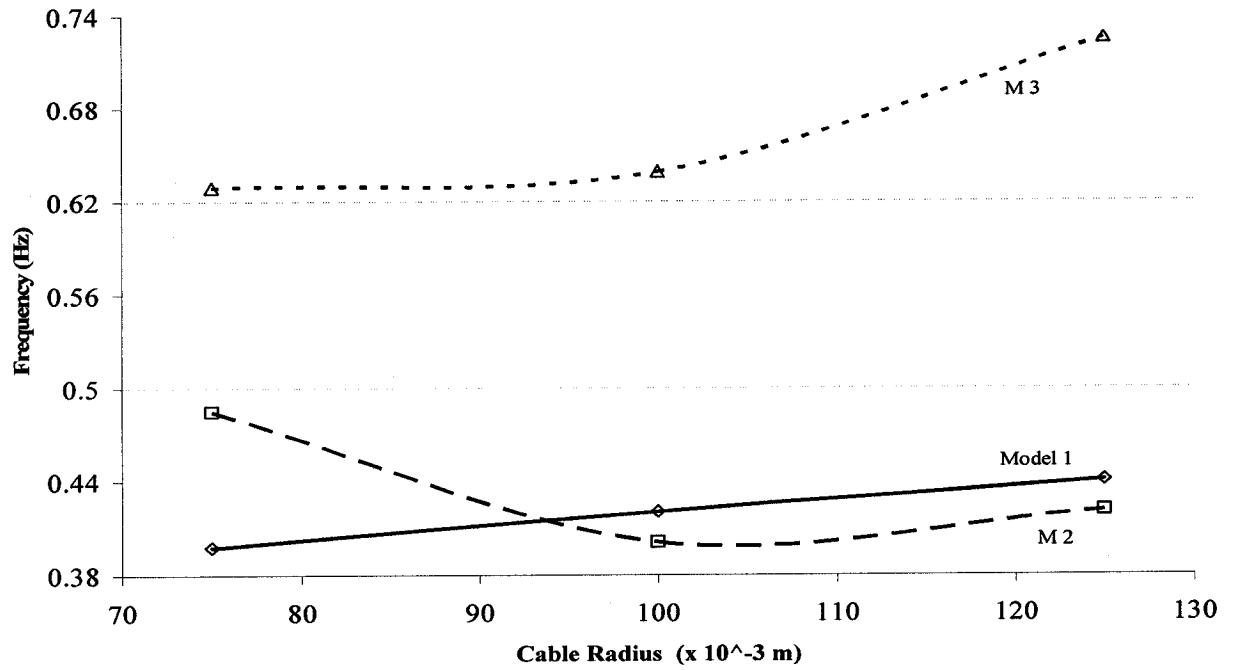


Figure 6.3.4.5 CFRP Cable Radius Effect on the 2nd Torsional Natural Frequencies, LDFA-a, Tha - Second Chang River Bridge

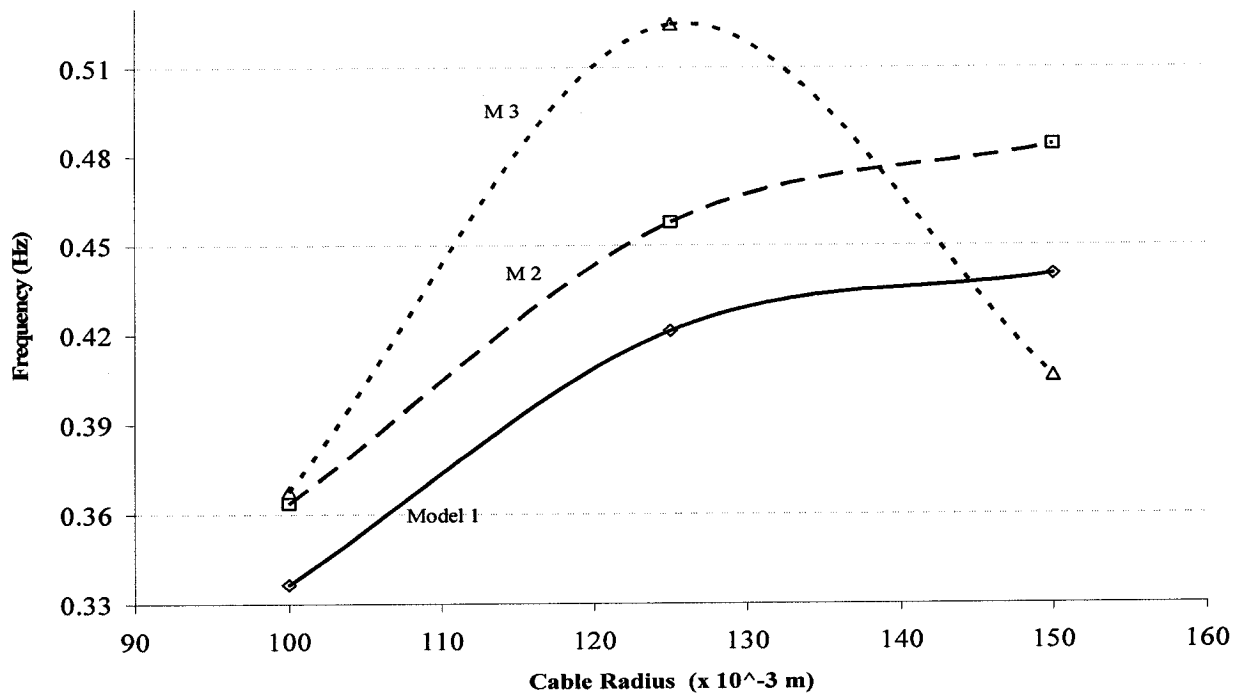


Figure 6.3.4.6 CFRP Cable Radius Effect on the 2nd Torsional Natural Frequencies, LDFA-a, Tha - Tatara Bridge

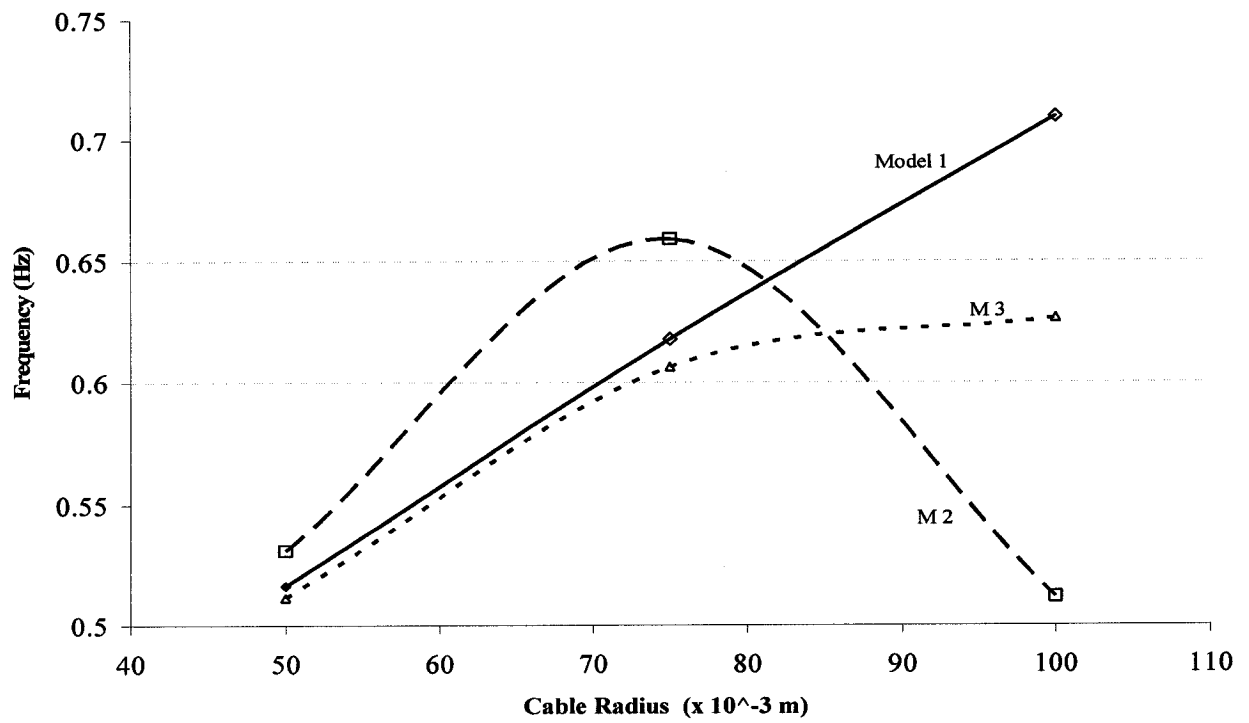


Figure 6.3.4.7 CFRP Cable Radius Effect on the 3rd Torsional Natural Frequencies, LDFA-a, Tha - Annacis Island Bridge

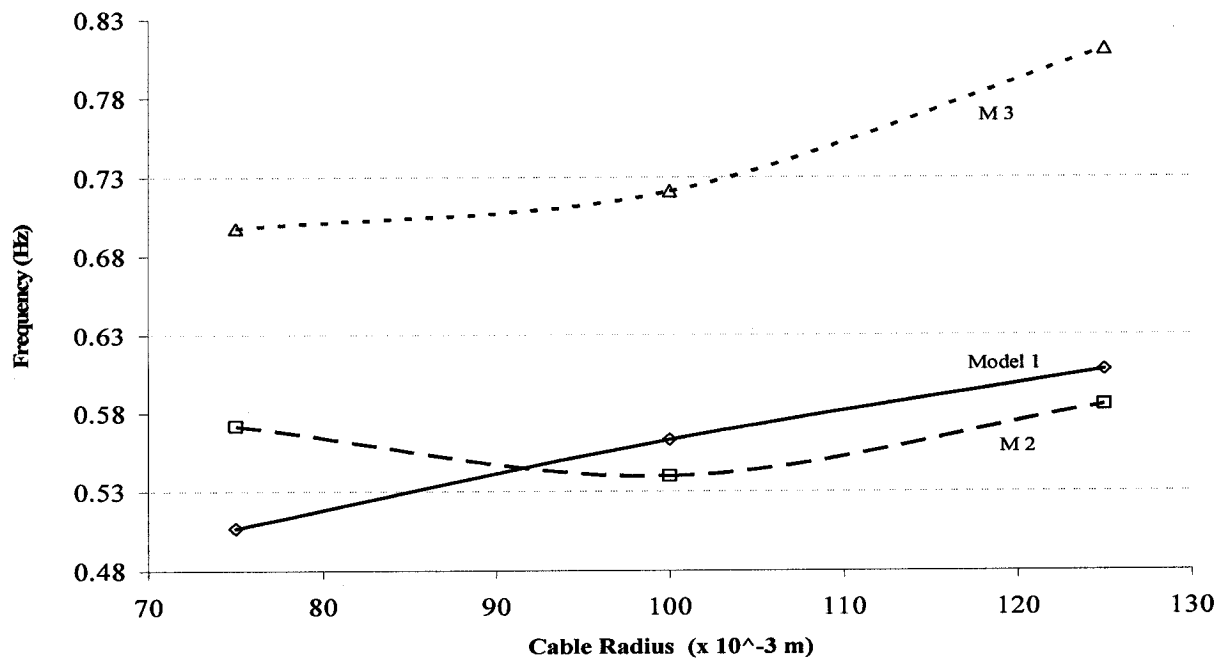


Figure 6.3.4.8 CFRP Cable Radius Effect on the 3rd Torsional Natural Frequencies, LDFA-a, Tha - Second Chang River Bridge

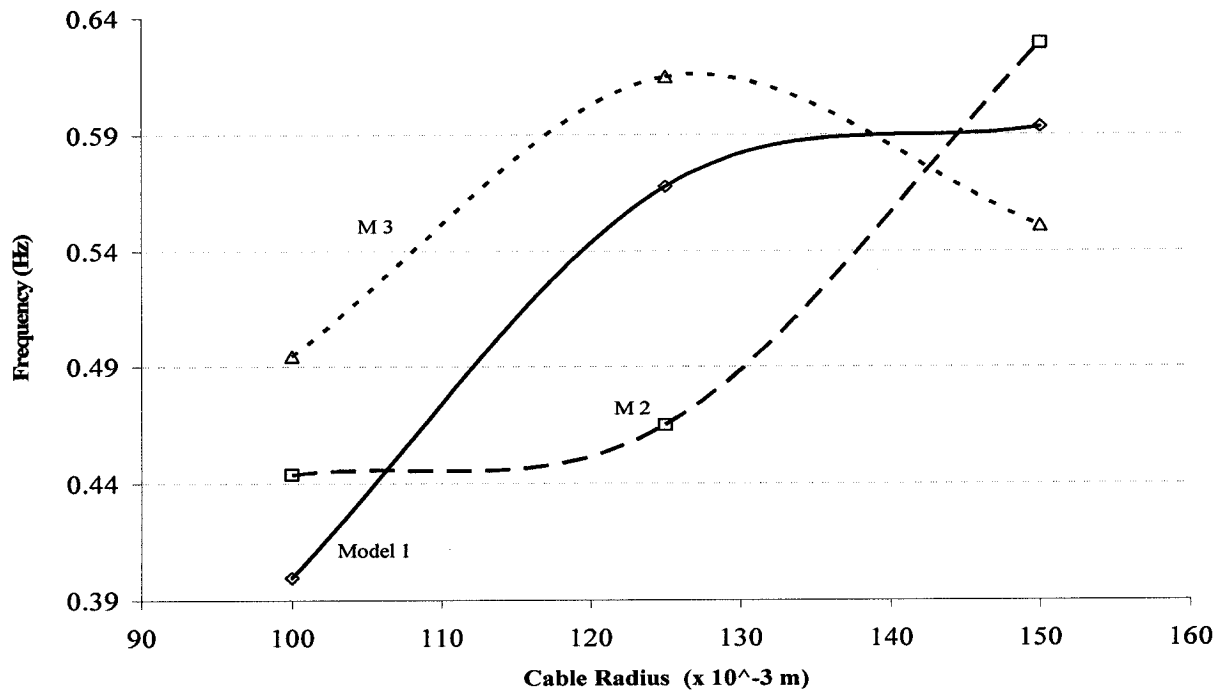


Figure 6.3.4.9 CFRP Cable Radius Effect on the 3rd Torsional Natural Frequencies, LDFA-a, Tha - Tatara Bridge

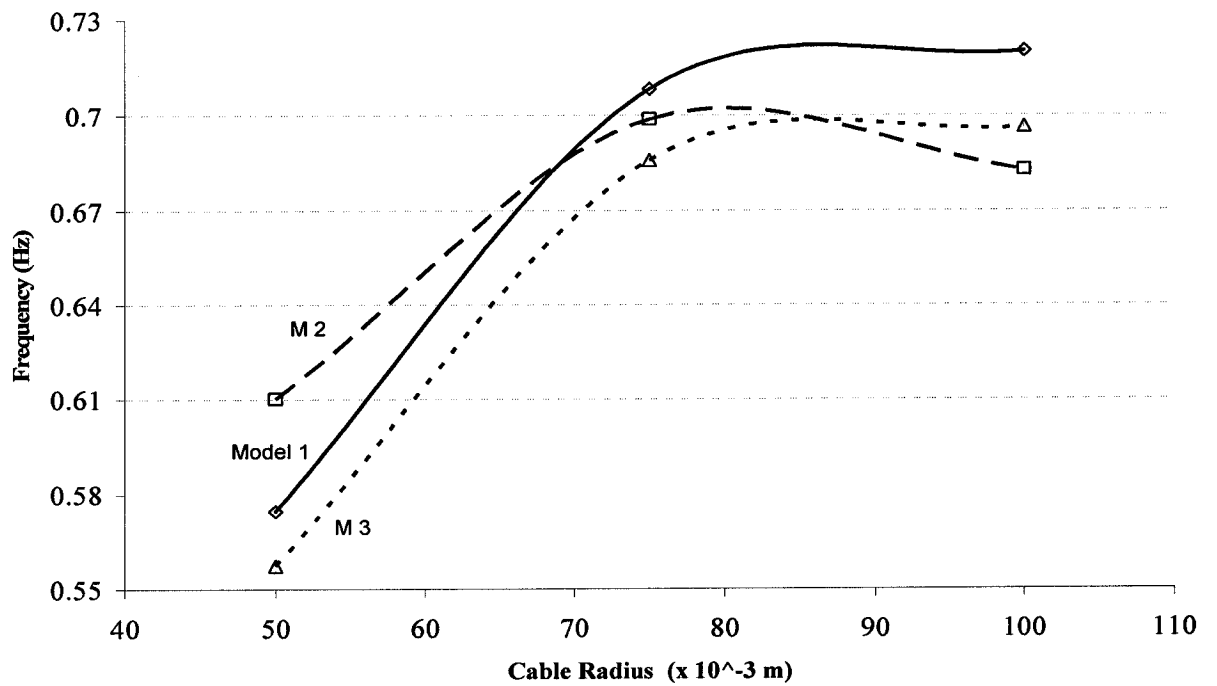


Figure 6.3.4.10 CFRP Cable Radius Effect on the 4th Torsional Natural Frequencies, LDFA-a, Tha - Annacis Island

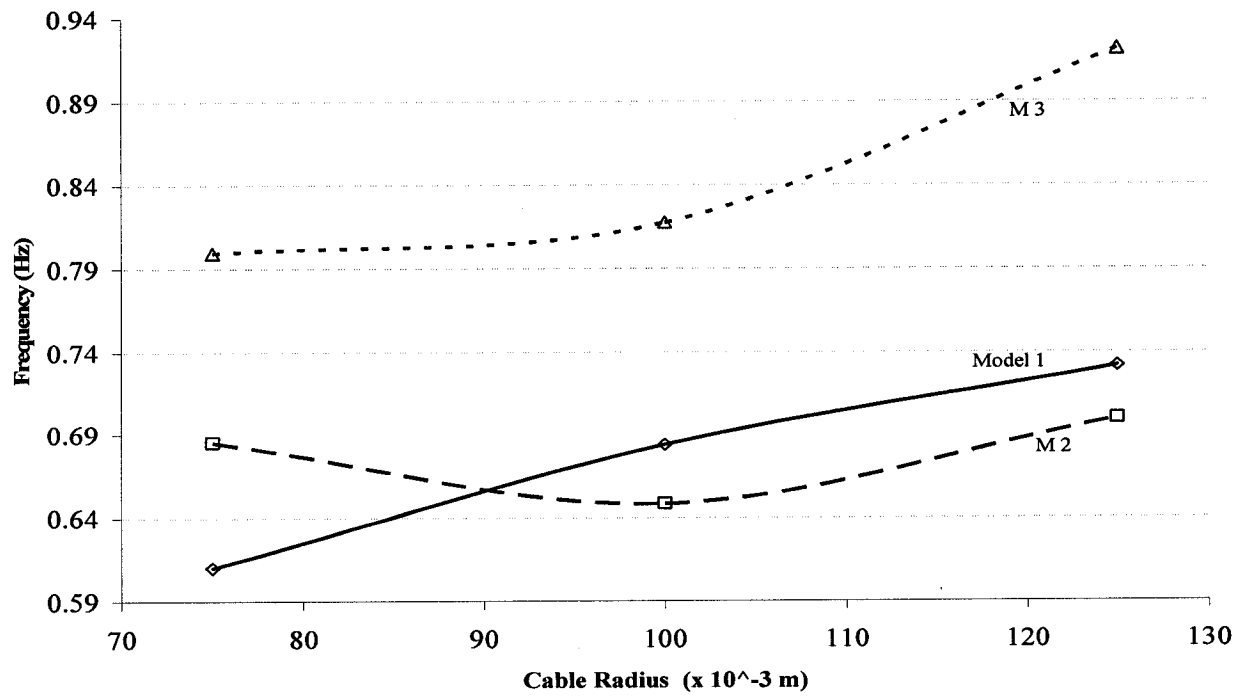


Figure 6.3.4.11 CFRP Cable Radius Effect on the 4th Torsional Natural Frequencies, LDFA-a, Tha - Second Chang River Bridge

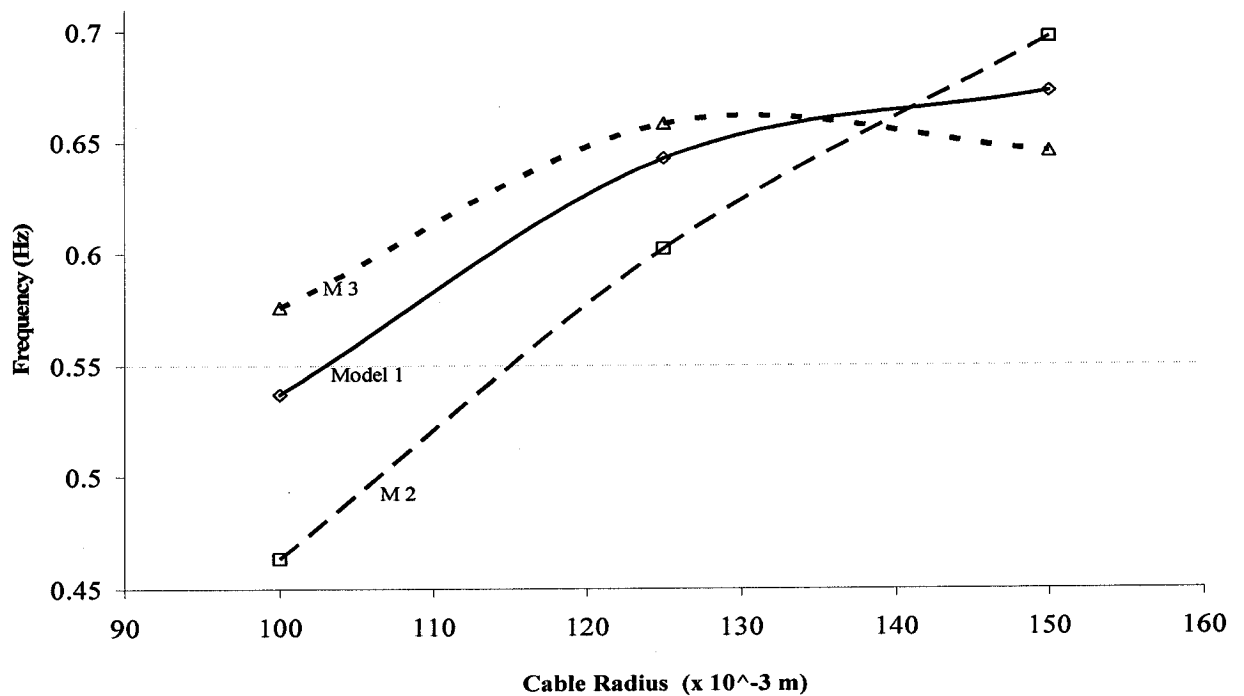


Figure 6.3.4.12 CFRP Cable Radius Effect on the 4th Torsional Natural Frequencies, LDFA-a, Tha - Tatara Bridge

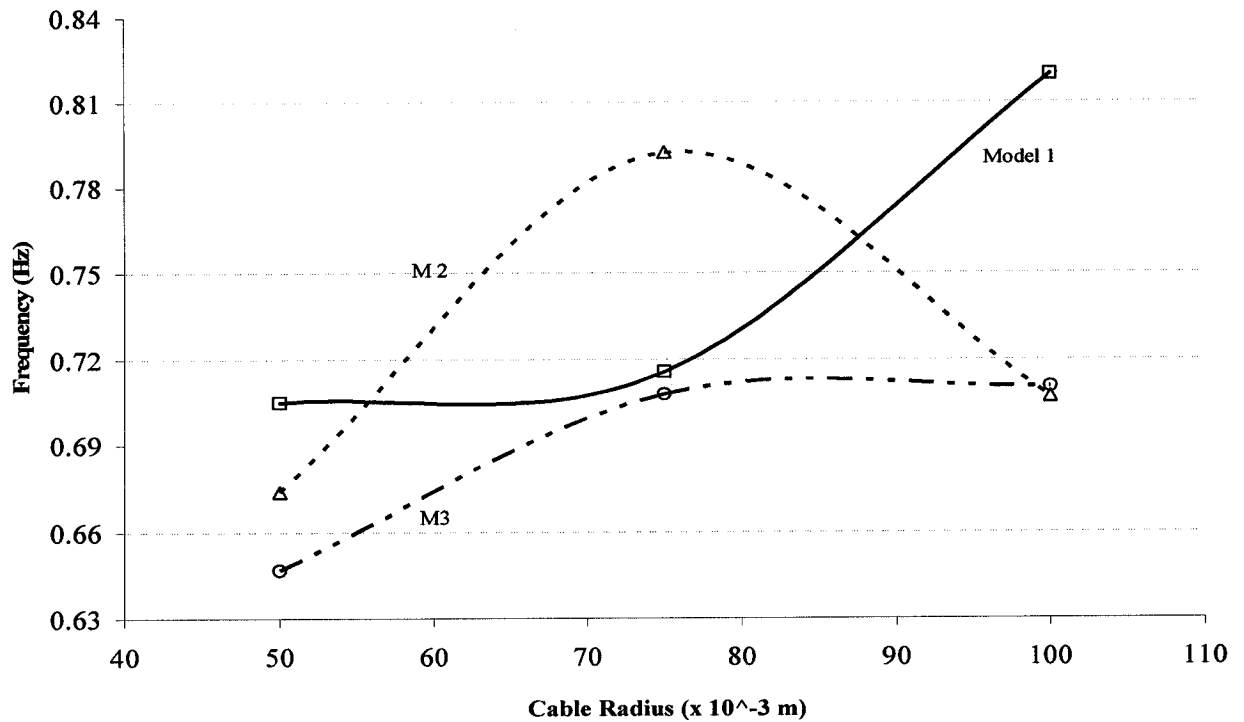


Figure 6.3.4.13 CFRP Cable Radius Effect on the 5th Torsional Natural Frequencies, LDFA-a, Tha - Annacis Island Bridge

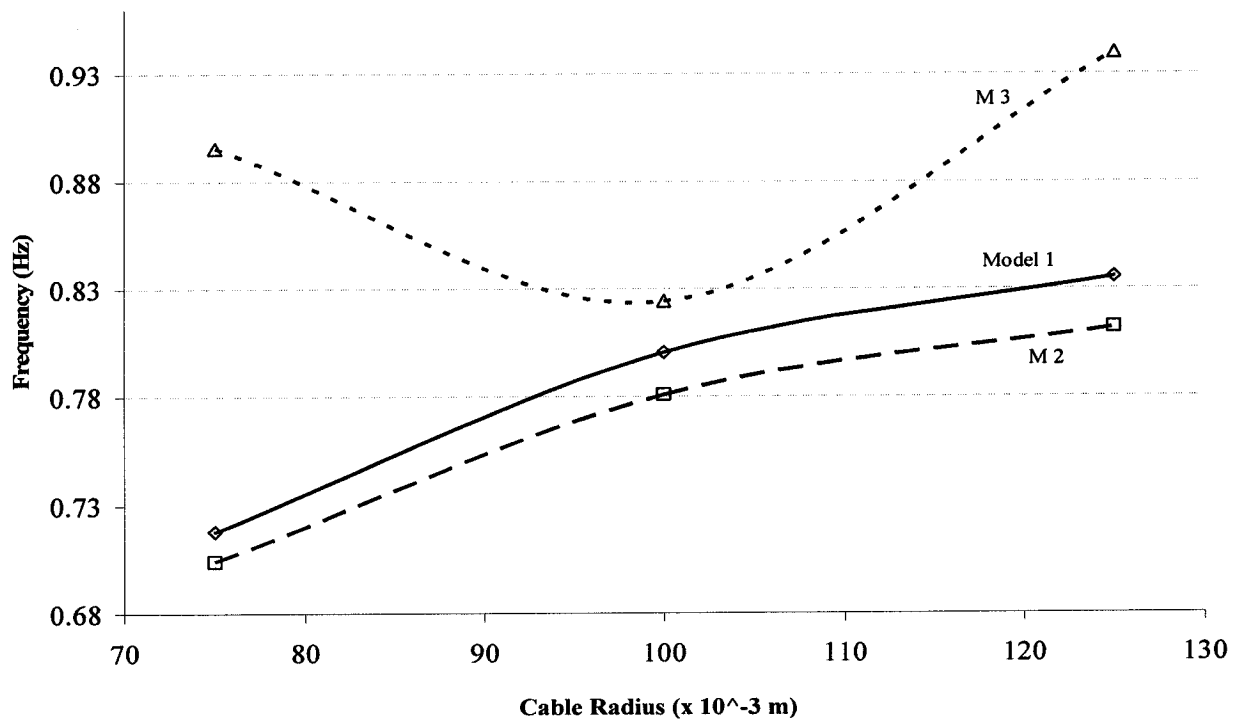


Figure 6.3.4.14 CFRP Cable Radius Effect on the 5th Torsional Natural Frequencies, LDFA-a, Tha - Second Chang River Bridge



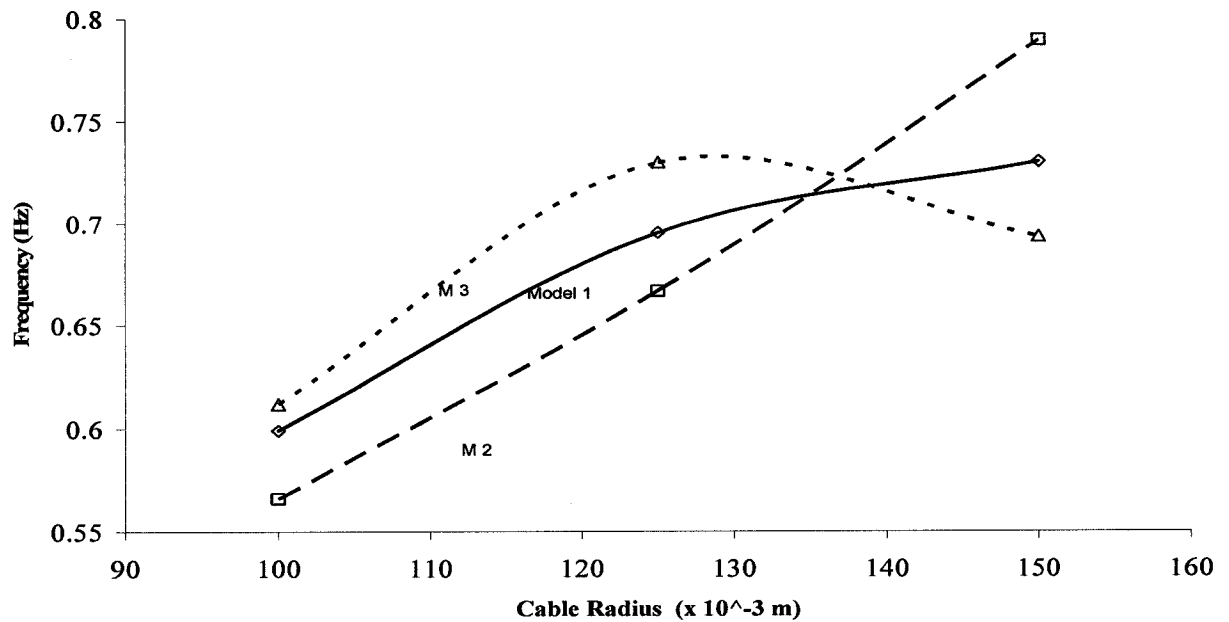


Figure 6.3.4.15 CFRP Cable Radius Effect on the 5th Torsional Natural Frequencies, LDFA-a, Tha - Tatara Bridge

### 6.3.5 Effect of Cable Radius on the Horizontal Natural Frequencies:

It has been found that the Horizontal Natural Frequencies (HNF) are constant or show very slight variation as the CFRP cable radius increases, except Model IV, which shows an increase in its HNF for the medium and long bridge central span lengths. Model I gives slight decrease then increase, and Model III gives slight increase in their first HNF with the very long central span.

The increase of the CFRP stay cables radius added high stiffness and low mass and increases the elastic restraint or supports provided by the cables along the deck. This effect is apparent on the vertical flexural and the torsional characteristics of the deck, however the horizontal (or the lateral) deck restraint has minimum improvement. Models I and III have the highest lateral flexural stiffnesses. Their HNF are affected very slightly once their mass becomes effective and their stiffness to mass ratio becomes sensitive in the longest span length. Model I has more mass than Model III and its HNF for the longest bridge span shows slight reduction, but with the largest examined cable radius, its HNF rises (Figure 6.3.5.3). The added stiffness and mass improves the stiffness to mass

ratio of Model III and gives a slight increase of the HNF with the Tataru Bridge (Figure 6.3.5.2 and 3).

On the other hand, Model IV has the least lateral (or horizontal) flexural stiffness in comparison to the other models. Its relatively low mass makes its stiffness to mass ratio improve with the medium central span length of the Second Chang River Bridge and the longest central span of the Tataru Bridge (Figures 6.3.5.2 to 5). As the stiffness to mass ratio of the deck is decreased, with the increase of the span length, and the stay-cables are much longer than with the medium and short central spans, the increase of the cable radius added relatively higher mass and resulted in reducing the increase of the HNF (Figure 6.3.5.5).

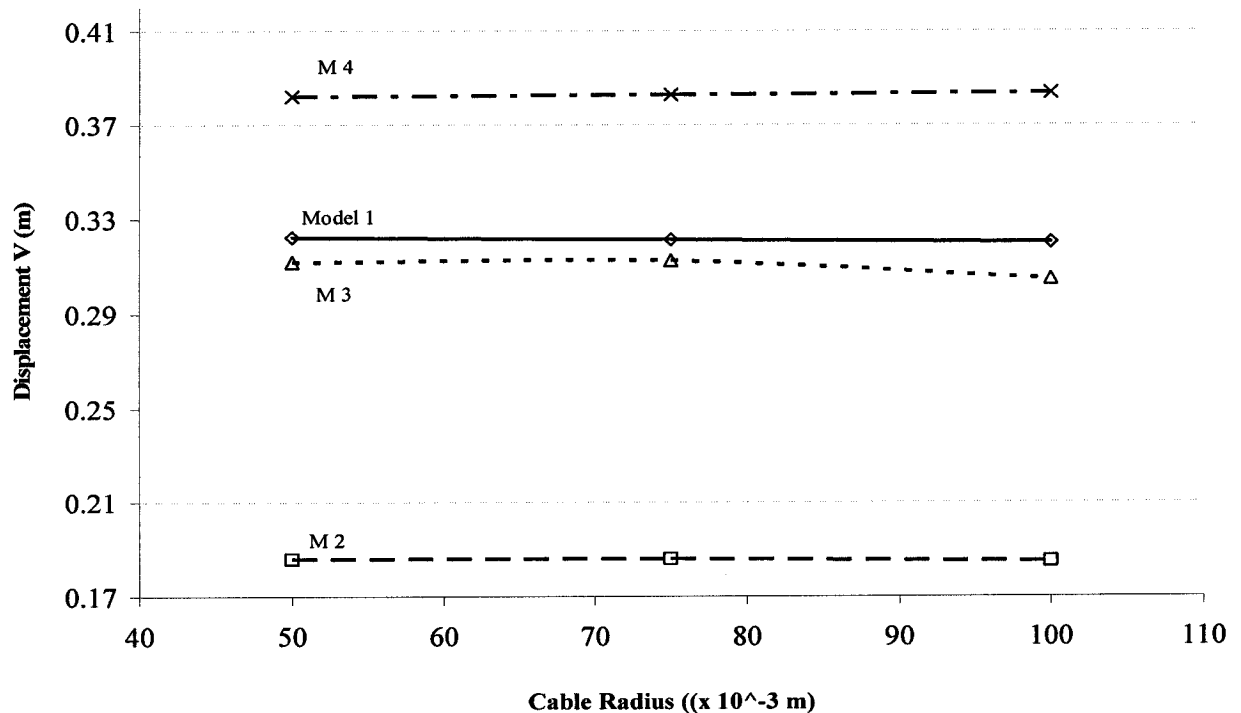


Figure 6.3.5.1 CFRP Cable Radius Effect on the 1st Horizontal Natural Frequencies, LDFA-a, Tha - Annacis Island Bridge

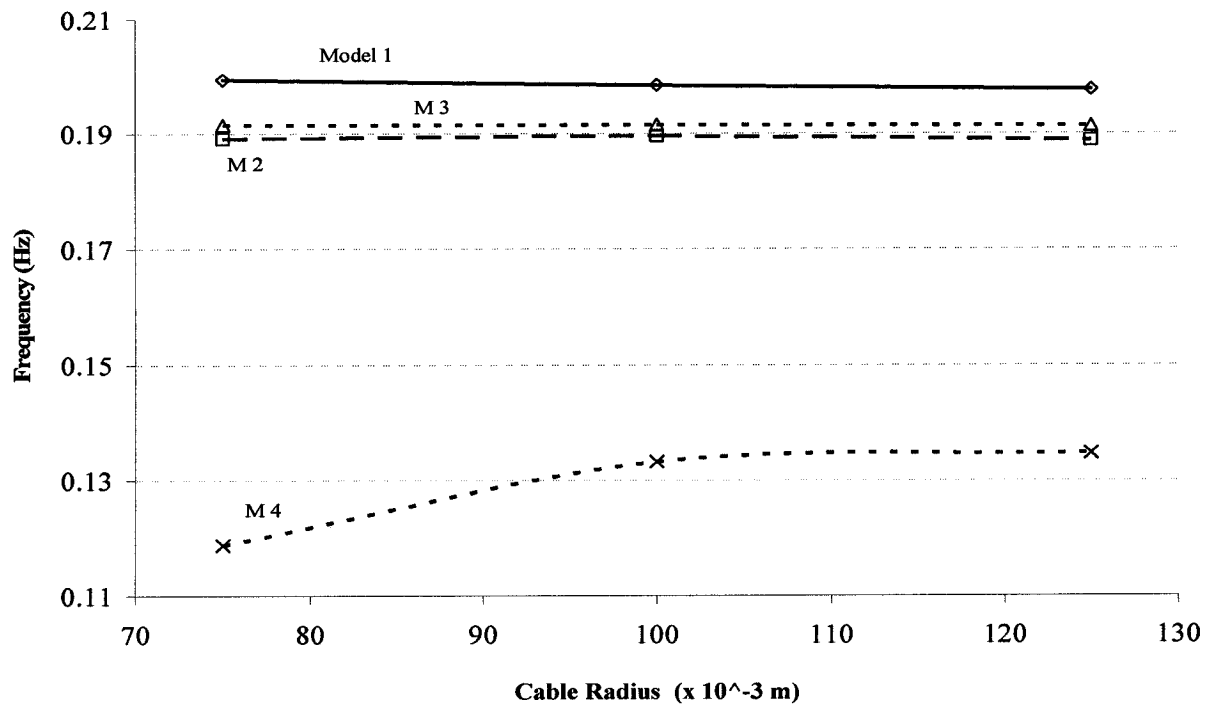


Figure 6.3.5.2 CFRP Cable Radius Effect on the 1st Horizontal Natural Frequencies, LDFA-a, Tha - Second Chang River Bridge

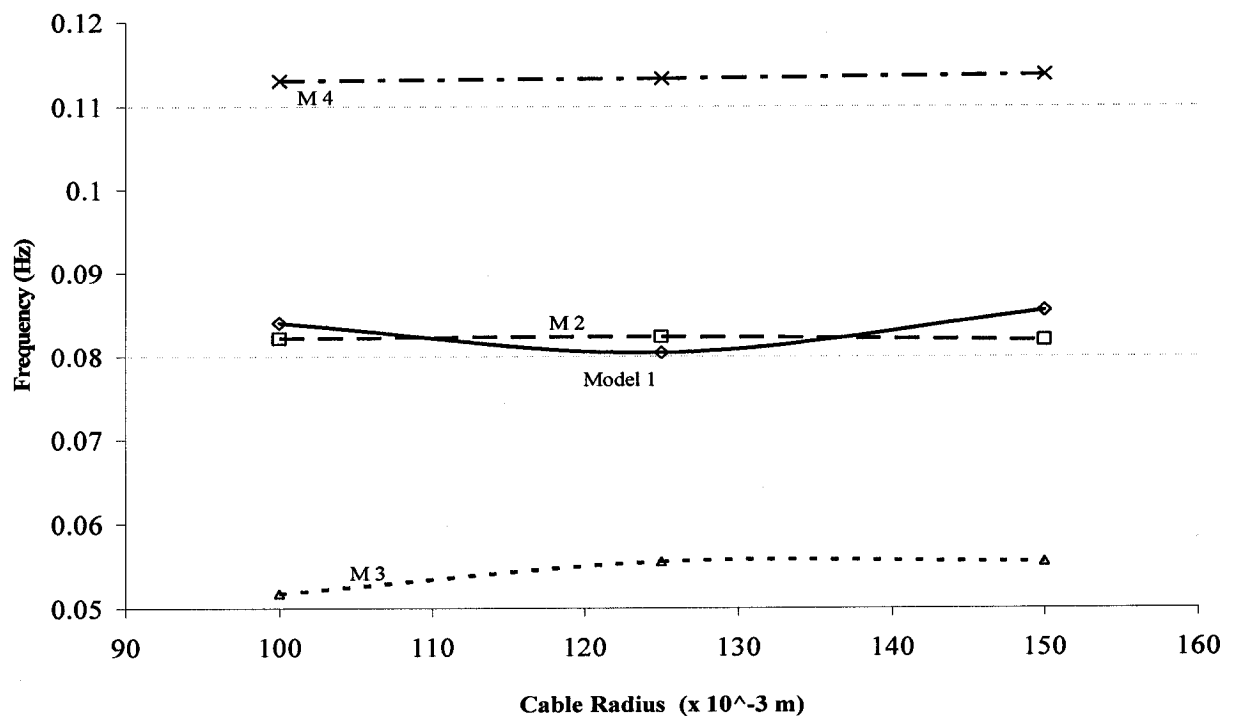


Figure 6.3.5.3 CFRP Cable Radius Effect on the 1st Horizontal Natural Frequencies, LDFA-a, Tha - Tatara Bridge

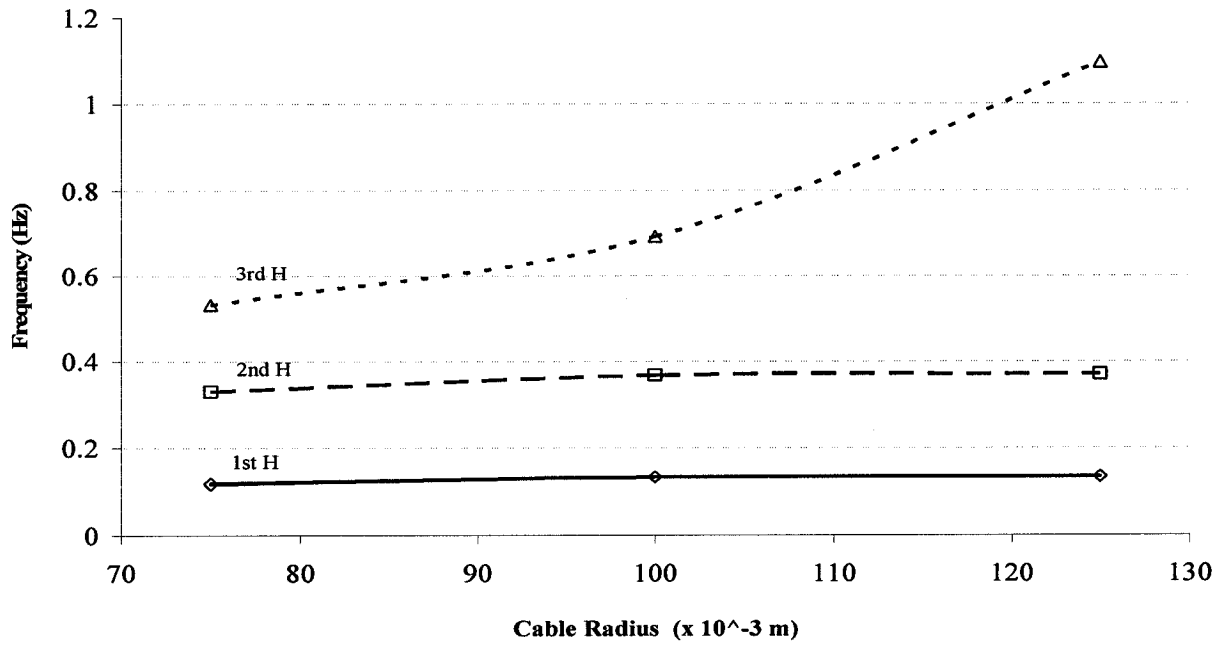


Figure 6.3.5.4 CFRP Cable Radius Effect on Natural Frequencies - Model IV, LDFA-a, Tha - Second Chang River

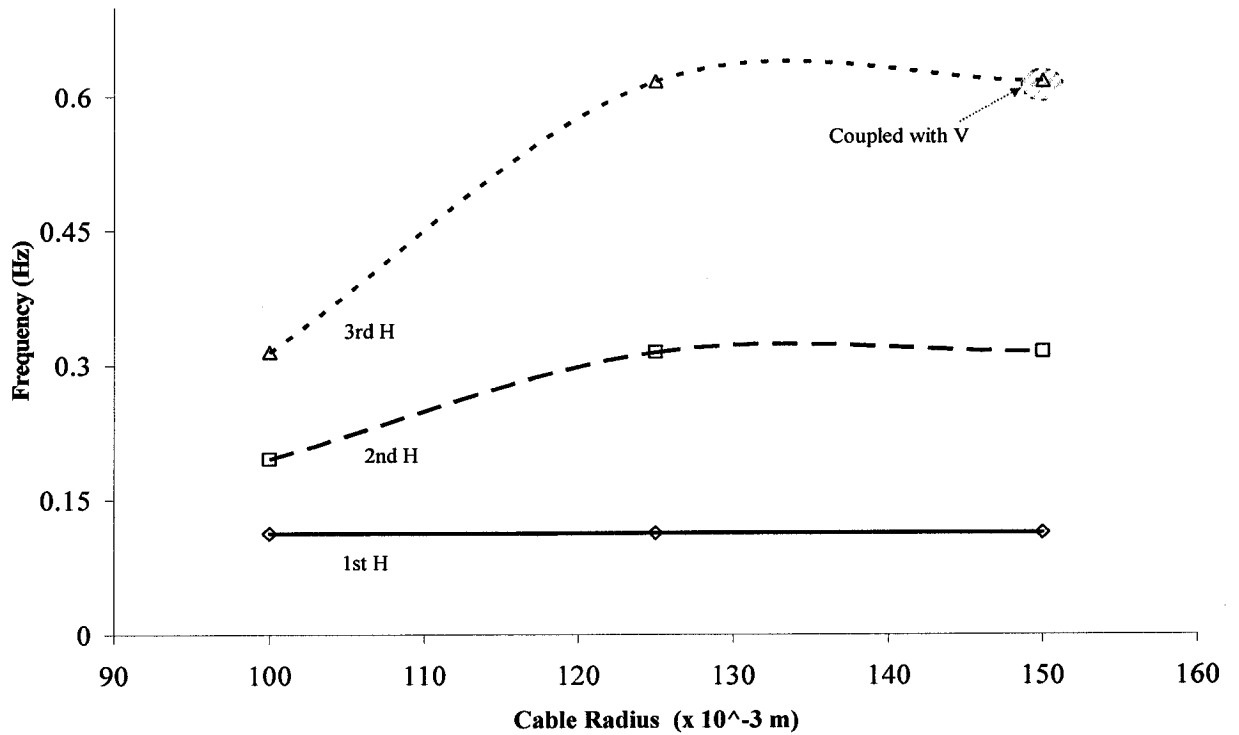


Figure 6.3.5.5 CFRP Cable Radius Effect on Natural Frequencies - Model IV, LDFA-a, Tha - Tatara Bridge

## 6.4 Pylons Stiffness:

Three pylon stiffness stages are used to study the effect of the pylon stiffness on the hybrid long span cable stayed bridge different variables (see Chapter 3, section 3.4) (i) one third of the maximum pylon stiffness (Flexible Pylon), or stage I, (ii) two thirds of the maximum pylon stiffness (Medium Stiff Pylon), or stage II, and (iii), the maximum pylon stiffness (Stiff Pylon), or stage III. To minimize the effect of the interaction between the pylon stiffness increase and the CFRP cable stiffness change, the cable radius is taken during the present investigation as the maximum of the studied range for every bridge (see section 6.4).

The following results reflects the sensitive balance between (i) the general effect of the pylon stiffness increase, or the enhancement of the elastic support restraint of the deck, (ii) the effect of the central-to-side span weight equilibrium through the stay-system and the pylon, which is dependent on the pylon stiffness, (iii) the effects of the deck moments of inertia and the distances between the elastic supports (or the cable-deck anchorage points), and (iv) the level of the tensile stress in the cable, which is an important parameter of the cable stiffness (as the cable radius is constant).

The increase of the pylon stiffness increases the elastic restraint of the cable stays to the deck; however it also affects the balance between the weights of the central span to the weight of the side spans. Consequently, as the pylon stiffness decreases, the effect of the ratio between the central span length to the side span length and the weight per unit length of the deck becomes more effective. The central span length to the side span ratio is 2.4 for the shortest bridge central span length, or the Annacis Island Bridge, 2.0 for the medium central span length, the Second Chang River Bridge, and 2.8 for the longest span length bridge, the Tatara Bridge. With stiffer pylons, the deformations and stresses become more dependent on the deck section moments of inertia, the deck dead load and the distances between the elastic supports. On the other hand the effect of the pylon stiffness on the deck is not uniform, and it is reduced significantly far from the pylons because (i) the pylon displacement in the direction of the traffic is increased with the height, and hence the restrain-effect of the pylon on the deck is decreased, (ii) the stay-cable length is increased with the height or similarly with the distance from the pylon, hence the capability of the cable to transfer the pylon stiffness effect is further reduced.

#### **6.4.1 Effect of Pylon Stiffness on the Maximum Deck Deformations:**

As the pylon stiffness increases, it has been found: (i) the maximum absolute lateral displacements  $U$  are decreased, with some exceptions related to Models III and IV, where they decrease then increase with the shortest and medium central span lengths (Figures 6.4.1.1 to 6.4.1.3). (ii) The maximum vertical displacements (or deflections) show reductions with the increase of the pylon stiffness with some exceptions; where Model IV and Model I show a decrease then an increase with the shortest and the longest examined spans respectively (Figures 6.4.1.4 to 6.4.1.6). (iii) The maximum absolute longitudinal displacements  $W$  show small decreases as the pylon stiffness increases with two exceptions. Model IV gives a very high reduction in its  $W$  with the shortest central span length and Model III shows a high increase for a medium central span length (Figures 6.4.1.7 to 6.4.1.9).

Model IV's intermediate flexible "deck-slab" part makes the attached stiffening laminate lateral displacement sensitive to the side boxes restraining effect. Similar to the rise of the cable stiffness, the increase of the pylon stiffness increases the bending stresses in the intermediate laminate. Due to the bending-extension and the bending-in-plane shear coupling, the lateral displacement increases. However, for the reasons explained earlier, as the span length increases, the effect of the pylon stiffness decreases in the central region of the mid-span. This explains the reduced variation of the maximum lateral displacement as the bridge central span increases (Figures 6.4.1.2 and 6.4.1.3).

The deflection is affected greatly by the balance between the effect of the central-to-side span ratio and the pylon stiffness increase effect. This is apparent with Model I, which has the heaviest section and high vertical flexural stiffness, when the bridge central span is the longest. As the pylon stiffness is low, its share in balancing the central span weight is reduced. Since the central to side span ratio is the highest with the Tatara Bridge, the deflection is increased in this case. On the other hand, when the pylon stiffness is high, it contributes to the balancing of the central span weight but it also reduces the load balancing effect of the side spans. However, for this very long span, the pylon stiffness effect is minimum at the centre of the mid span, far from the pylon, as explained earlier, and the deflection of this region is more dependent on the deck stiffness and the cable

pre-tension (which is constant here). Hence an increase of the maximum deflection occurs (Figures 6.4.1.6). Furthermore, Model I does not show the same behavior with the shortest and medium spans as its stiffness is high and it can transfer the restraining effect over longer lengths, which appears enough for the shortest and medium central span cases. Model IV shows similar behavior but with the shortest span. This model has low stiffness but also much lower weight per unit length than Model I. A similar explanation is made for the relation between the side span balancing force and the pylon stiffness can be applied on Model IV behavior. However the effect of the low stiffness is apparent, as the deflection is highly increased with the stiff pylon. In addition, the high flexibility of the intermediate part (or the slab-grid) of this section also contributes to increase the maximum deflection as it occurs in this part of the deck. With the longer bridge central span length, Model IV low weight becomes highly effective and reduces the maximum deflection, greatly reducing the effect of the other parameters.

Model III in all previous cases, and because of its multi-tubular section high bending-extension coupling effect, shows an increase with its maximum longitudinal displacement  $W$ , as the deck restraint increases with the medium span length. This model also has high flexural stiffness that is reducing the local effect of the reduced pylon-restraining effect with the long cables. Model IV show a reduction of its maximum  $W$ , which is usually laid in the intermediate part of the section between the two “girders”, at the central part of the bridge, with the shortest span length. Increasing the elastic restraint of the “girders” due to the increase of the pylon stiffness is not uniform over the span length. Hence, the reduced restraint at the central part of the deck reduces the bending-extension coupling effects and leads to reduce the maximum longitudinal displacement.

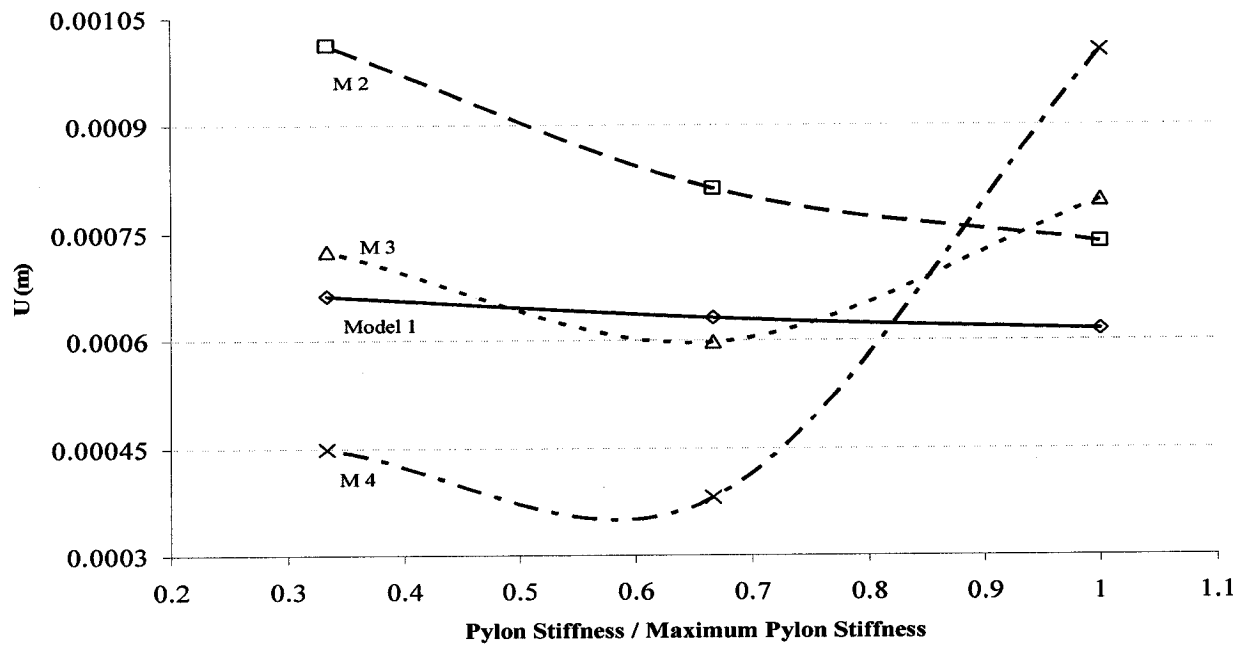


Figure 6.4.1.1 Pylon Stiffness Effect on maximum lateral displacement  $U$ - for different models-LDFA-a, Tha, LLA, Annacis Island Bridge

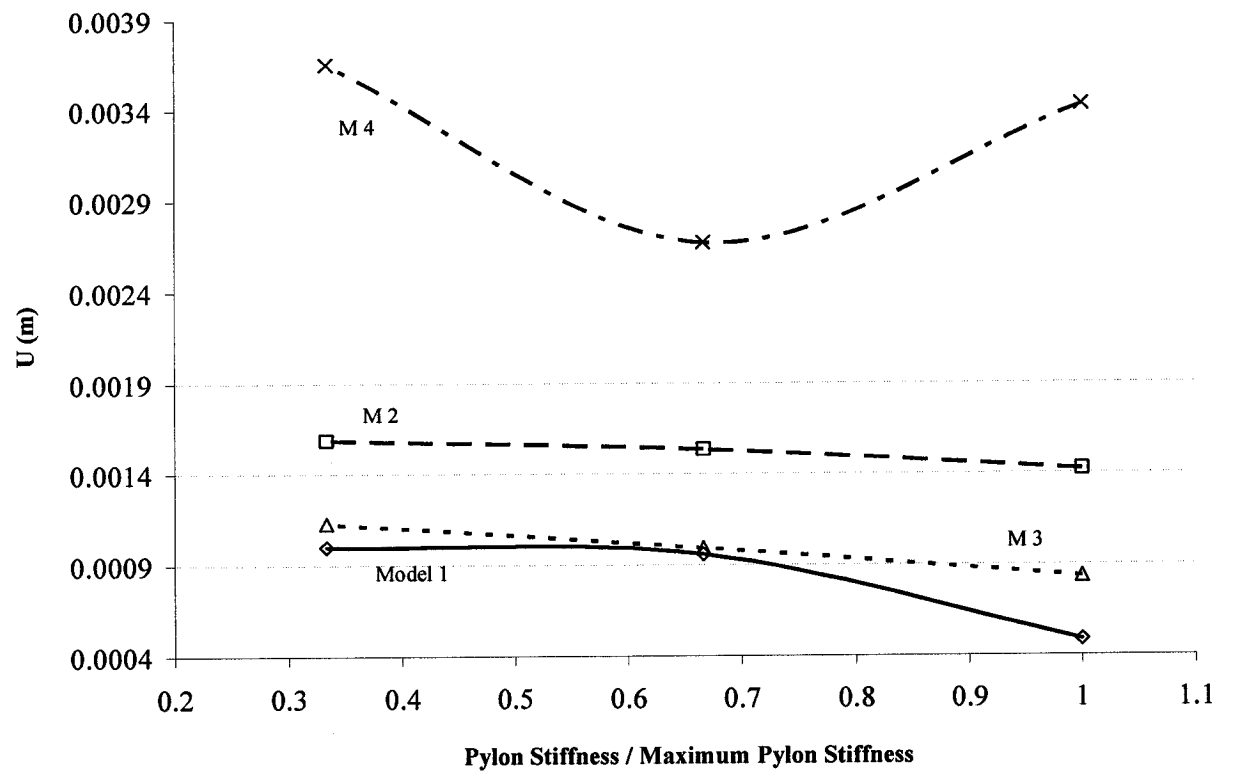


Figure 6.4.1.2 Pylon Stiffness Effect on maximum lateral displacement  $U$ - for different models- LDFA-a, Tha, LLA, Second Chang River Bridge



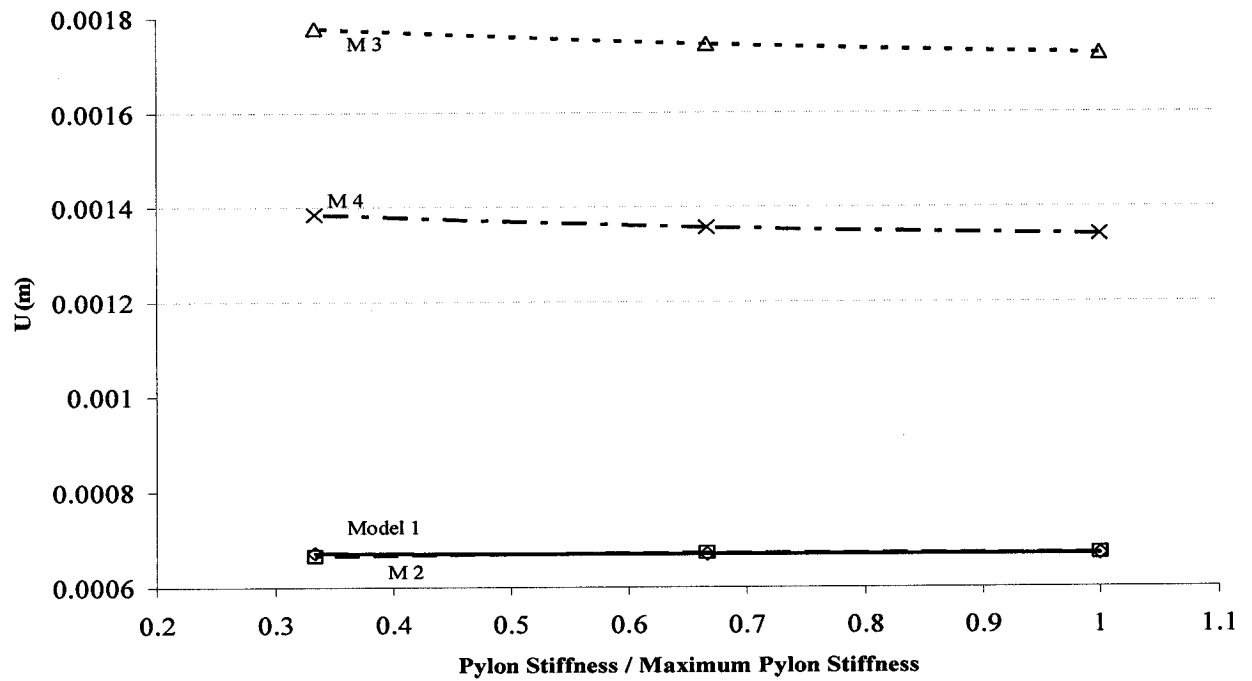


Figure 6.4.1.3 Pylon Stiffness effect on maximum lateral displacement  $U$ - for different Models- LDFA-a, Tha, LLA, Tatara Bridge

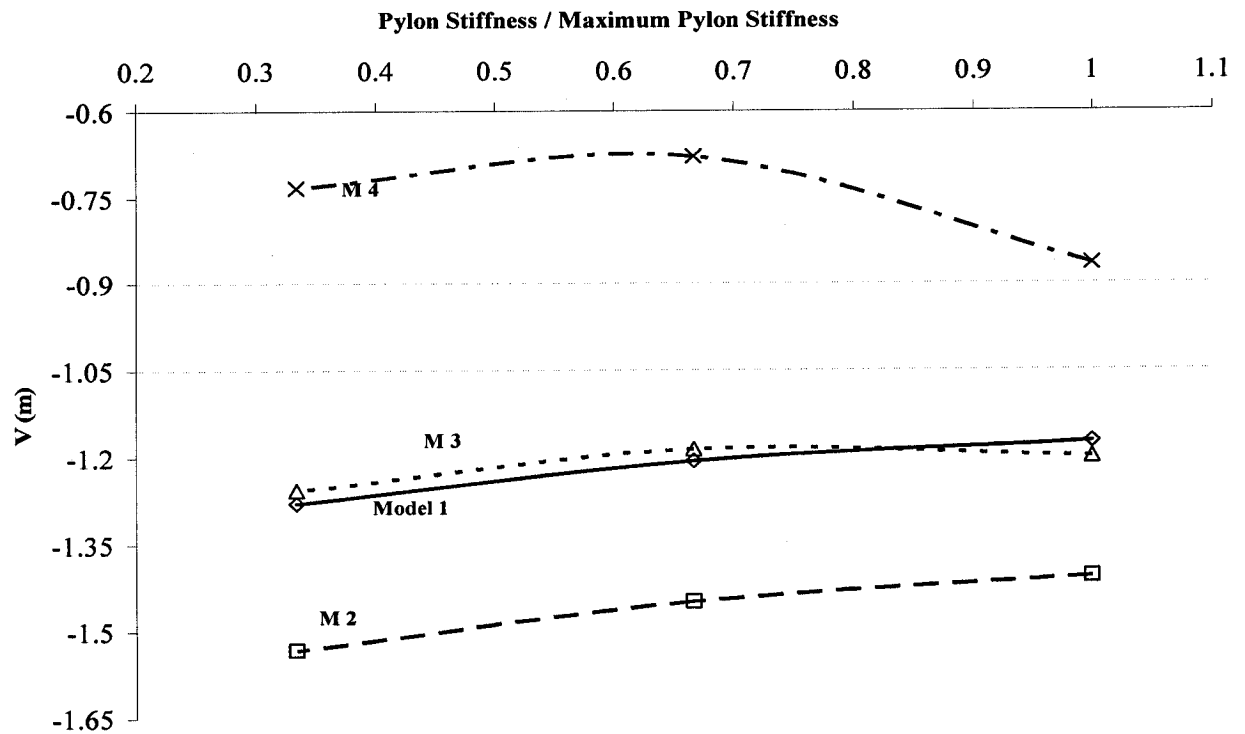


Figure 6.4.1.4 Pylon Stiffness effect on maximum vertical displacement  $V$ - for different Models- LDFA-a, Tha, LLA, Annacis Island Bridge

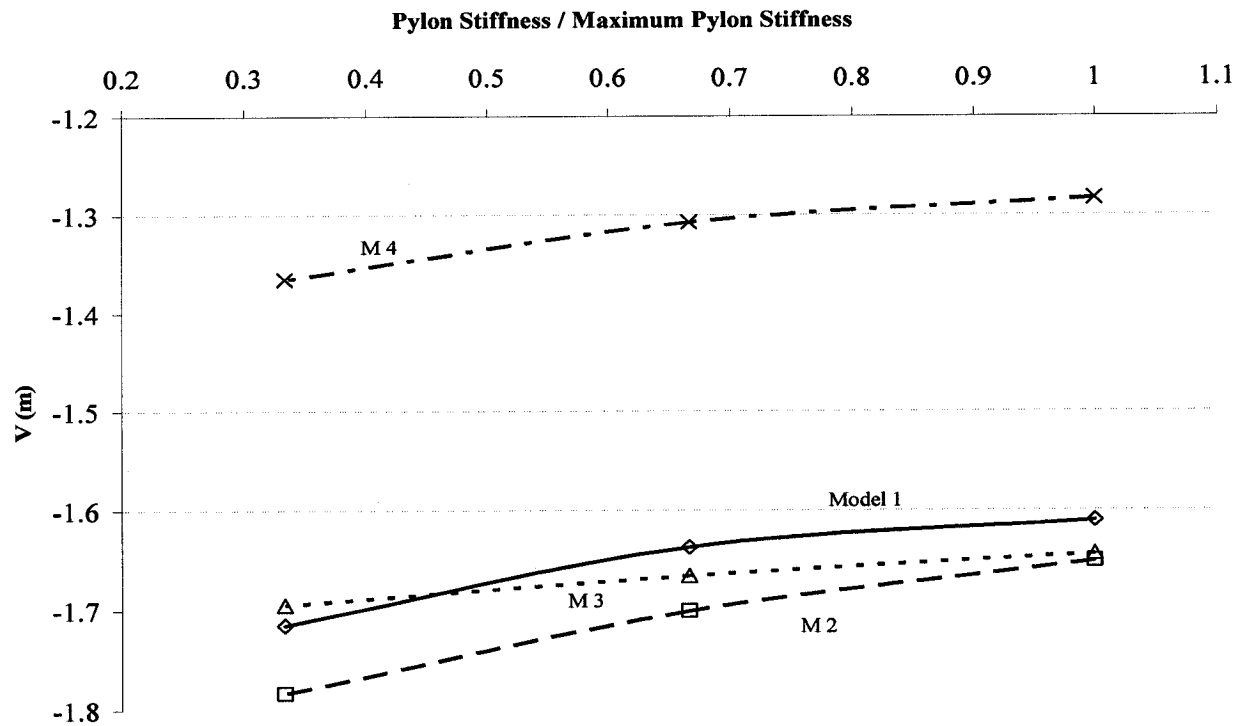


Figure 6.4.1.5 Pylon Stiffness Effect on maximum vertical displacement V- for different models- LDFA-a, Tha, LLA, Second Chang River Bridge

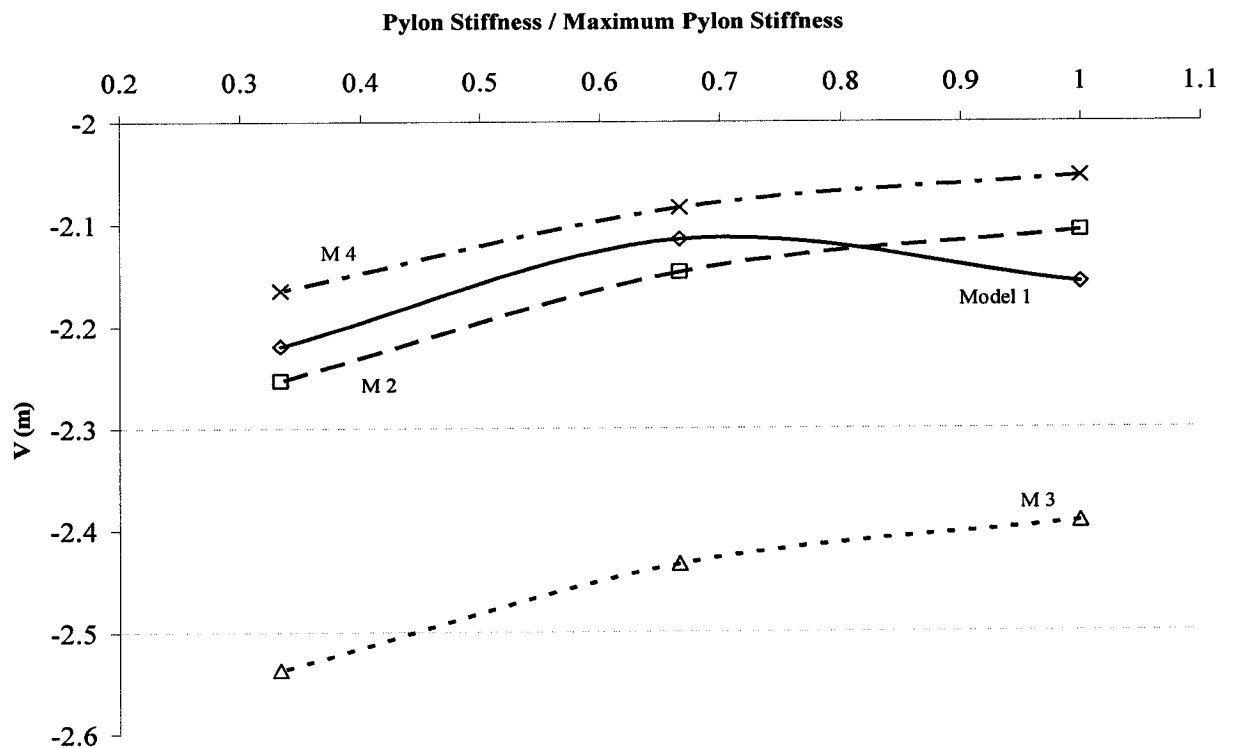


Figure 6.4.1.6 Pylon Stiffness effect on maximum vertical displacement V- for different Models- LDFA-a, Tha, LLA, Tatara Bridge

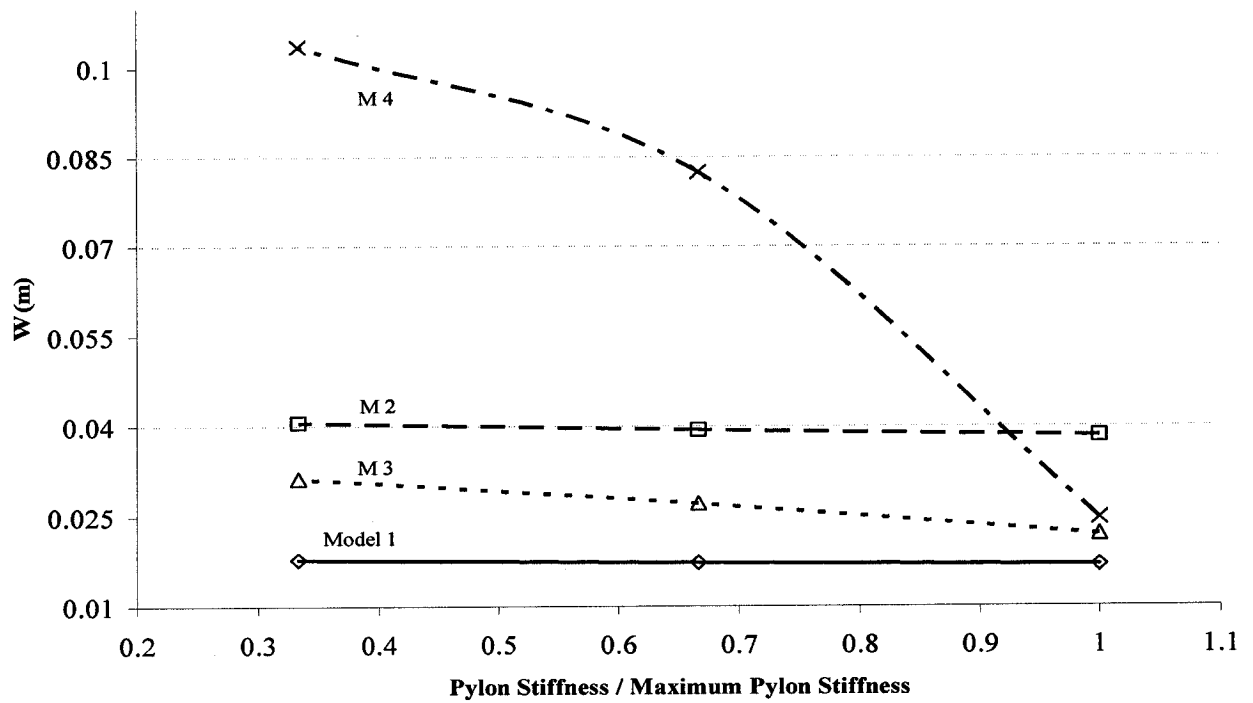


Figure 6.4.1.7 Pylon Stiffness effect on maximum Displacement W- for different Models-LDFA-a, Tha, LLA, Annacis Island Bridge

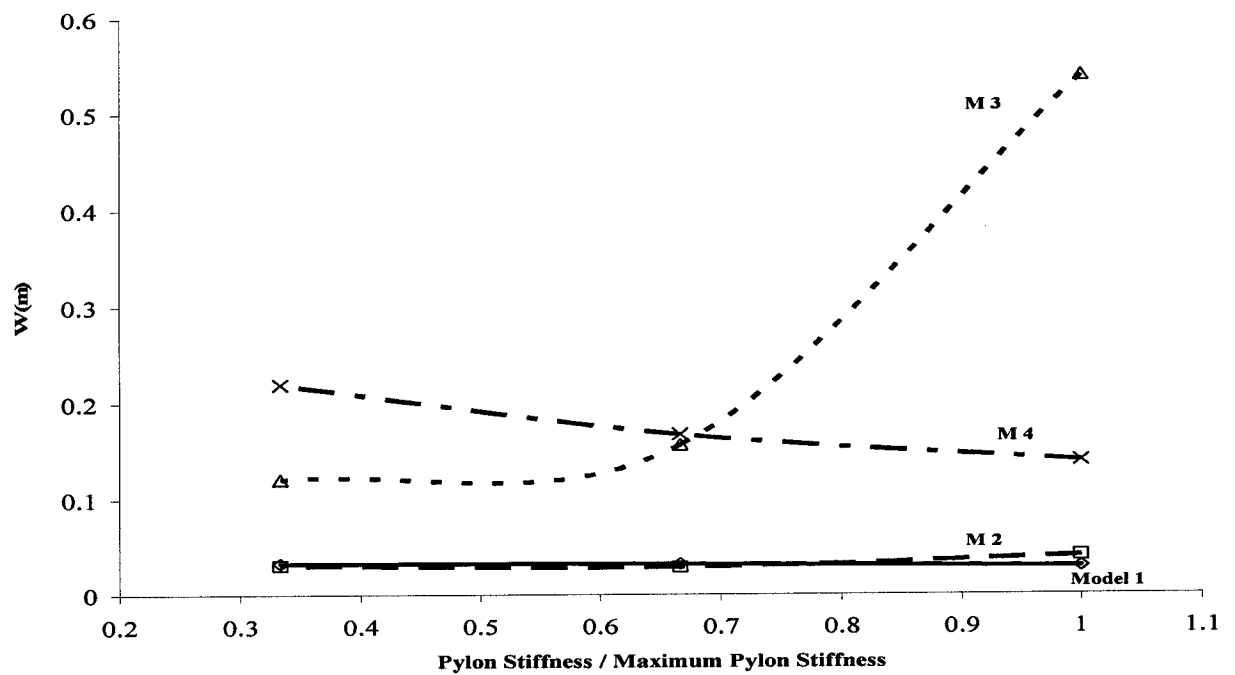


Figure 6.4.1.8 Pylon Stiffness effect on maximum longitudinal displacement W- for different models- LDFA-a, Tha, LLA, Second Chang River Bridge

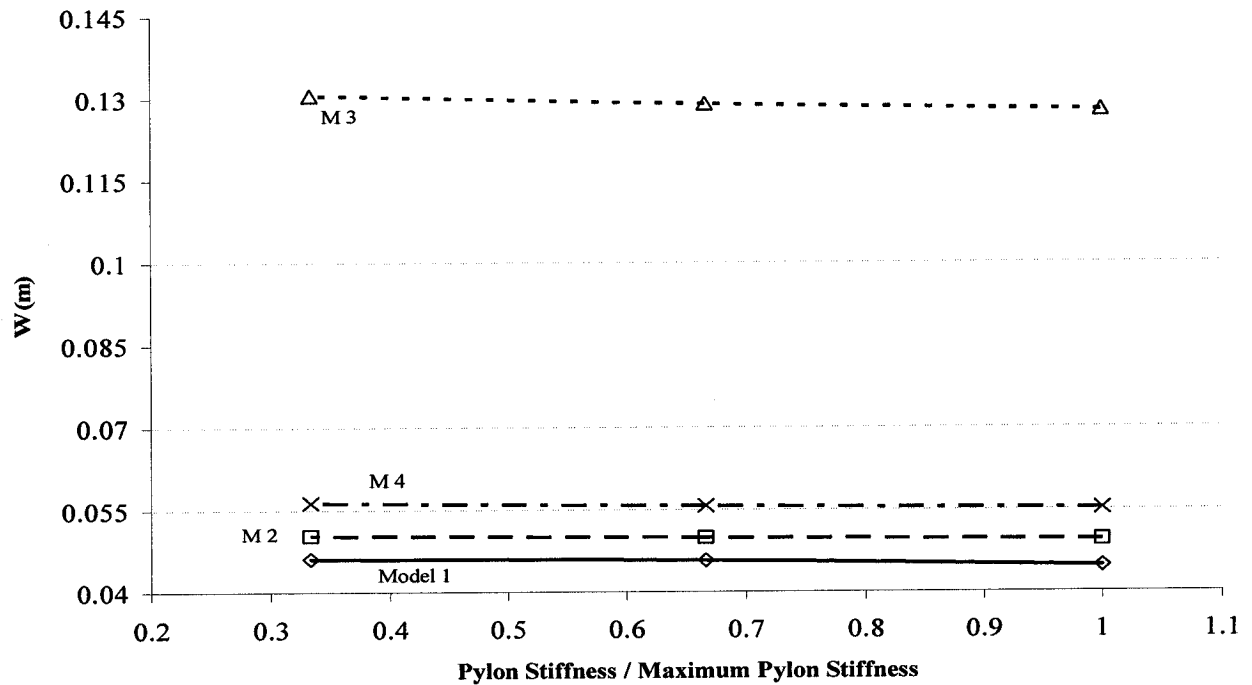


Figure 6.4.1.9 Pylon Stiffness effect on maximum longitudinal displacement W- for different Models- LDFA-a, Tha, LLA, Tatara Bridge

#### 6.4.2 Effect of Pylon stiffness on the maximum Tsai-Hill Failure Function:

It has been found that no effect or very slight decrease of THFF is dominant when the pylon stiffness is increased, with some exceptions. The results of the maximum Tsai Hill Failure Function (THFF) of different laminates, different models and different spans can be summarized as follows: (i) the THFF of the top Laminate of the Laminated Box Girder of Models I, II and III, and the stiffening laminate of Model IV are constant or show slight changes as the pylon stiffness increases. There are some exceptions where Model I shows an increase with the medium and long central span lengths, Model II shows a decrease with the medium central span, and Model IV shows a relatively high increase with the shortest span lengths (Figures 6.4.2.1 to Figure 6.4.2.3). (ii) the THFF of the bottom Laminate of the Laminated Box Girder of Models I, II and III and the left tubular laminate of Model IV, are decreased slightly as the pylon stiffness increases. Again there are some exceptions where Model I shows an increase in its THFF with the medium and long central span lengths, Model IV shows an increase with the shortest central span length, and Model II gives a high reduction with the longest span length (Figures 6.4.2.4

to Figure 6.4.2.6). (iii) The THFF of the vertical Laminates of the Laminated Box Girder of Models I, II and III and the right tubular laminate of Model IV show a slight decrease as the pylon stiffness increases. Again there are some exceptions, where Models I shows an increase in its THFF with the longest central span length, Model II shows extremely high increase in its THFF as the pylon stiffness increases from stage II to stage III, and Model IV shows an increase with the shortest span length (Figures 6.4.2.7 to Figure 6.4.2.9).

Slight decrease of the deck stresses as the pylon stiffness increases is observed with the traditional short span cable stayed bridge system designed using a big steel girder deck (i.e. the cross sectional area and all the moments of inertia are big) [Walther et al 1999]. This is very close to the results of Models I and III of all laminates, for the shortest span length (Figures 6.4.2.1, 4 and 7). Model I- maximum THFF of the top and bottom laminates show an increase with the medium central span (Figures 6.4.2.2 and 5) and a high increase with the longest central spans (Figures 6.4.2.3 and 6), as the pylon stiffness increases. The increase of the pylon stiffness increases the restraint of the deck, particularly close to the pylon zone of maximum THFF regions. In addition, the effect of the pylon stiffness on the balance between the central span with the side spans increases the flexural stresses of these “stiff decks”, as the deck transfers more loads with stiffer pylons. This leads to a further increase of the axial stresses at the top and bottom surfaces, and hence increases the THFF of the top and the bottom laminates.

Model II's vertical laminates have a critical condition, as discussed in the previous sections. The stresses of the short vertical laminates, in the two sides of the section close to the bridge ends (in the longitudinal direction), can switch from tension to compression causing a sudden rise of THFF. The pylon stiffness increase results in an increase of the elastic restraint and changes the distribution of the stresses in the deck. In the side spans where the “pylon regions” and the deck external supports exist, the tendency of those critical laminates to change their axial stress fields' direction is very high. This explains the sudden rise of THFF of the vertical laminates with the “wide deck section” of the Second Chang River Bridge. With the other bridge central span lengths, they show very slight decrease in their THFF as the axial stresses remain low and do not switch to compression.

The vertical laminates of Model I show a slight change in their flexural stresses, and the changes of their axial stresses due to the increase of the pylon stiffness is low as well. As the bridge central span length increases for this relatively heavy section, the stresses also slightly increase and a rise in the THFF is observed (Figure 6.4.2.9). On the other hand, Model III's cylindrical laminates have a similar stress state to Model I's vertical laminates. However the deck section is lighter, the restraining action due to the pylons is relatively low and the resulting bending-extension coupling is low as well. Accordingly the THFF shows a decrease for all bridge span lengths.

Model IV shows an increase in its THFF of all its laminates with the Annacis Island Bridge, as the pylon stiffness increases. This model has low stiffness but also lower weight per unit length. It is also sensitive to the increase in the elastic restraint and the balance of the weights of the central and the side spans due to the increase of the pylon stiffness. The bending stress distributions are changed, which result in an increase of the axial stresses, and hence they increase the THFF. However as the bridge central span increases, the light weight and the reduction of the restraining effect of the pylons, due to the increase of the stay-cables lengths, reduces the bending stresses, and results in a slight decrease of THFF.

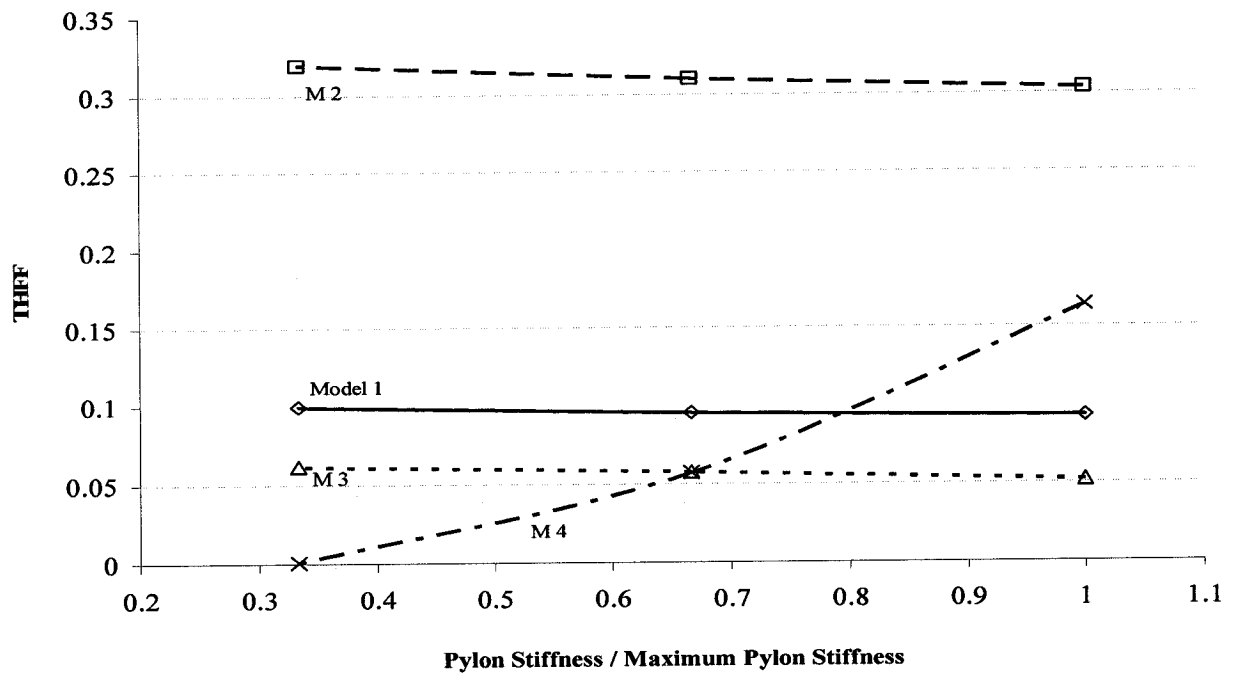


Figure 6.4.2.1 Pylon Stiffness Effect on maximum THFF - Top Laminate of Models I, II & III and Stiffening Laminate of Model IV- LDFA-a, Tha, LLA, Annacis Island Bridge

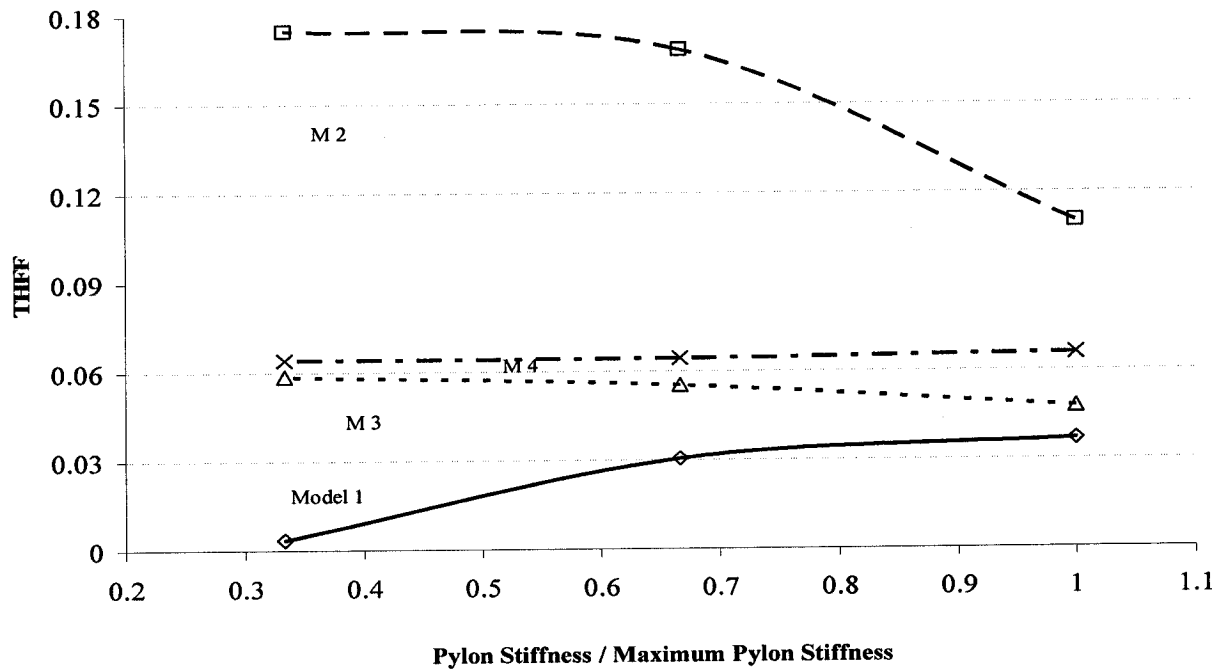


Figure 6.4.2.2 Pylon Stiffness Effect on maximum THFF - Top Laminate of Models I, II & III and Stiffening Laminate of Model IV- LDFA-a, Tha, LLA, Second Chang River Bridge

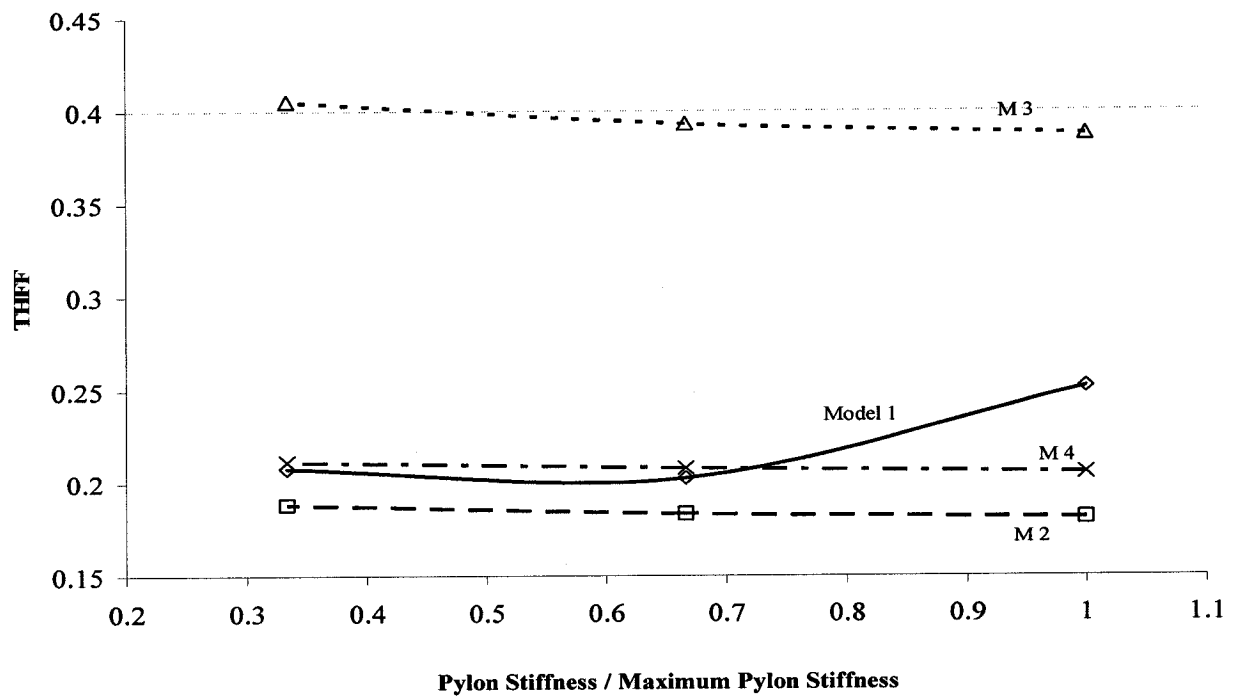


Figure 6.4.2.3 Pylon Stiffness Effect on maximum THFF - Top Laminate of Models I, II & III and Stiffening Laminate of Model IV- LDFA-a, Tha, LLA, Tataru Bridge

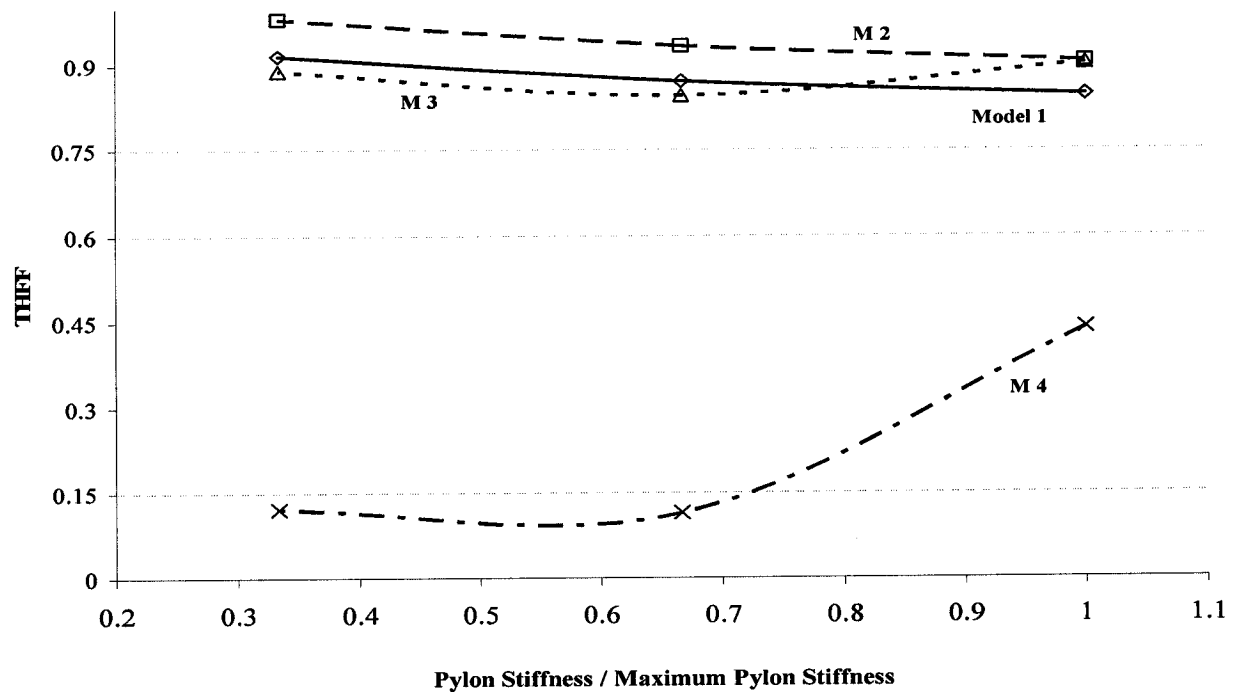


Figure 6.4.2.4 Pylon Stiffness effect on maximum THFF - Bottom Laminate of Models I, II & III and Left Tubular Laminate of Model IV- LDFA-a, Tha, LLA, Annacis Island Bridge



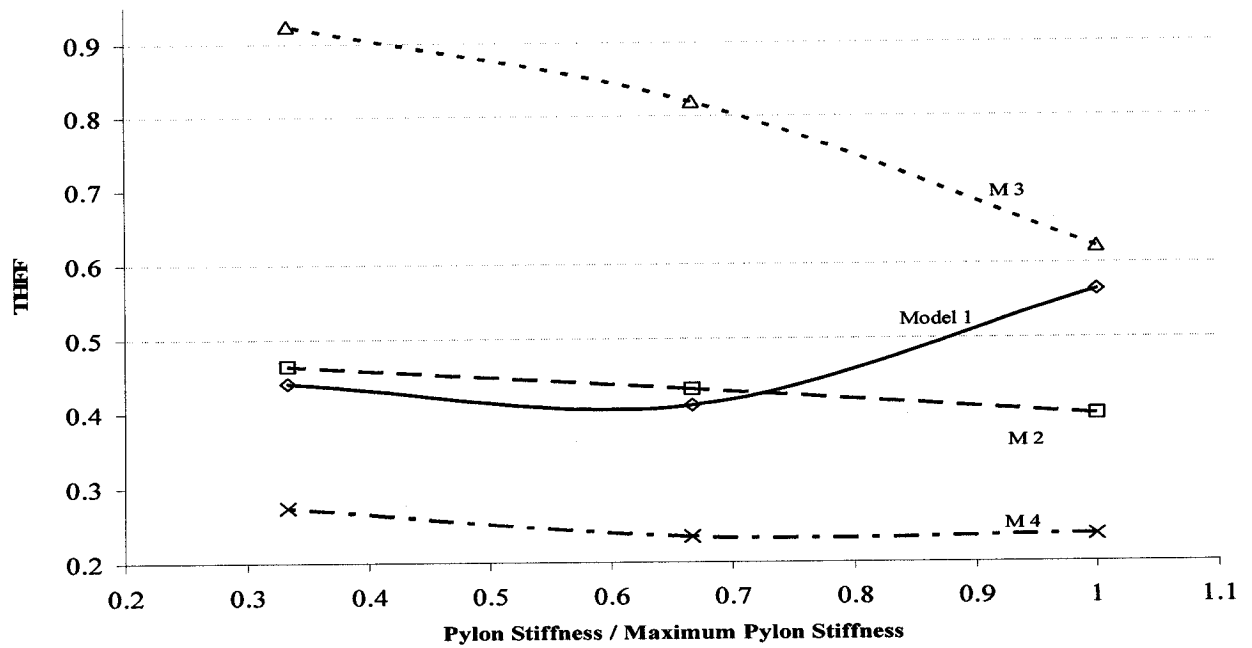


Figure 6.4.2.5 Pylon Stiffness Effect on maximum THFF - Bottom Laminate of Models I, II & III and Left Tubular Laminate of Model IV- LDFA-a, Tha, LLA, Second Chang River Bridge

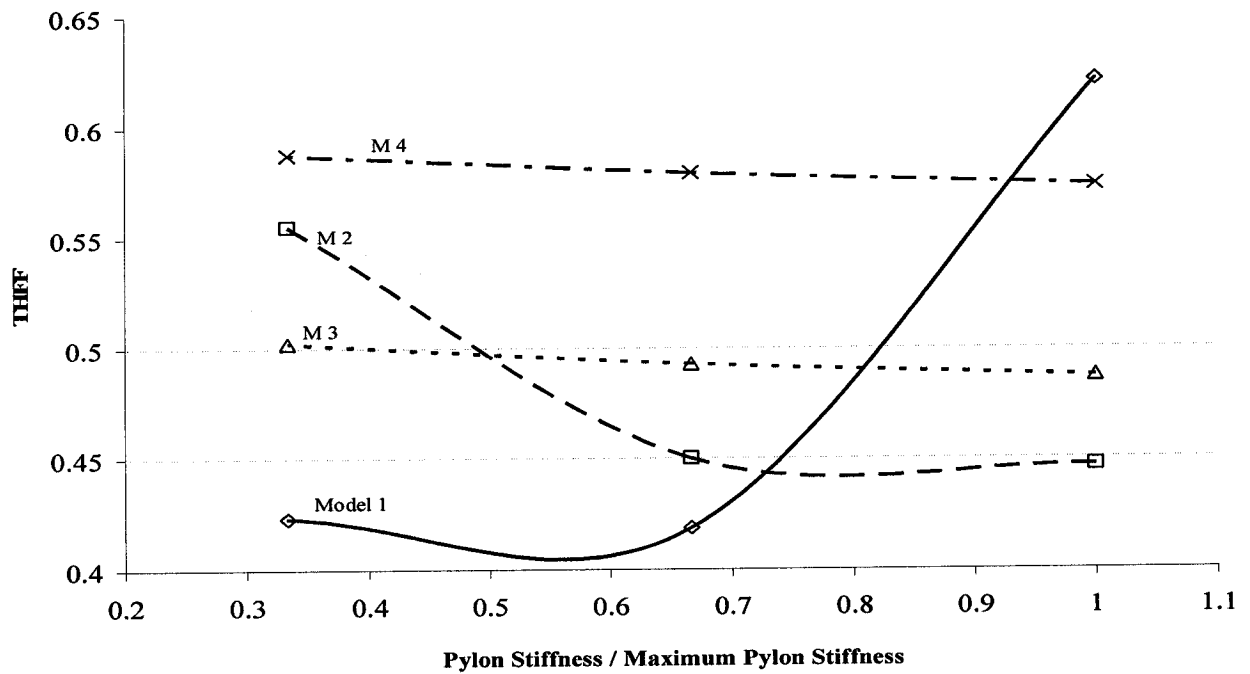


Figure 6.4.2.6 Pylon Stiffness Effect on maximum THFF - Bottom Laminate of Models I, II & III and Left Tubular Laminate of Model IV-LDFA-a,Tha, LLA, Tatara Bridge

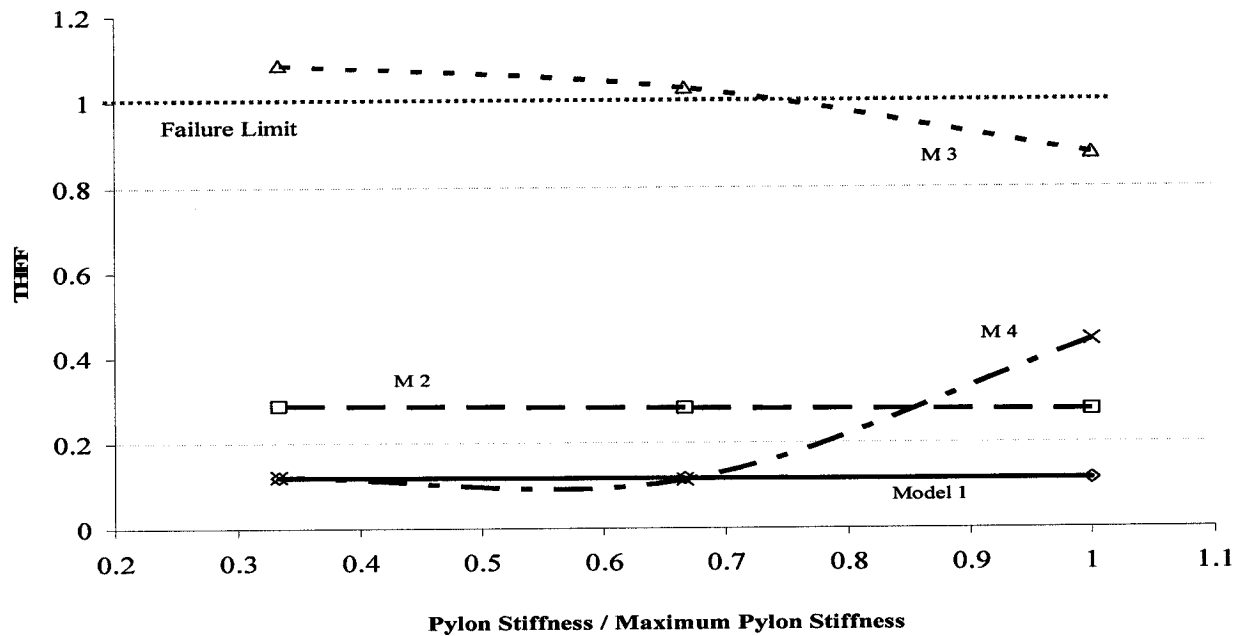


Figure 6.4.2.7 Pylon Stiffness effect on maximum THFF - Vertical Laminate of Models I, II & III and Right Tubular Laminate of Model IV- LDFAa, Tha, LLA, Annacis Island Bridge

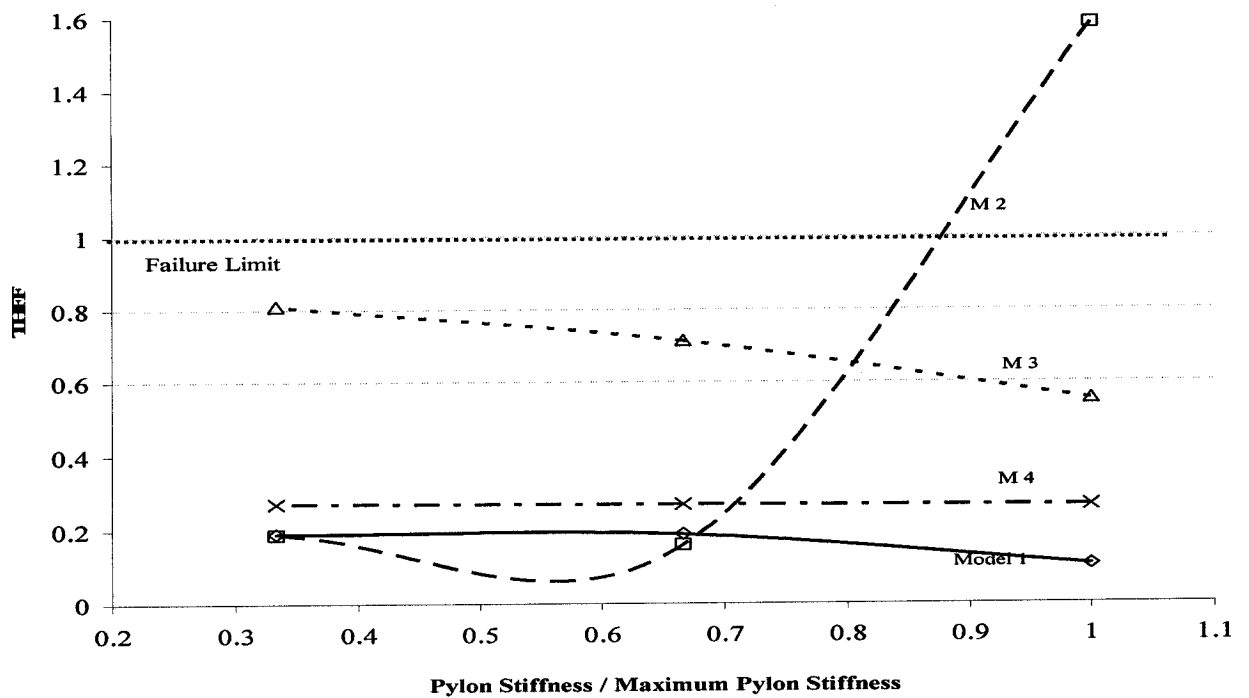


Figure 6.4.2.8 Pylon Stiffness Effect on maximum THFF- Vertical Laminates of Models I, II & III and Right Tubular Laminate of Model IV-LDFA-a, Tha, LLA, Second Chang River Bridge

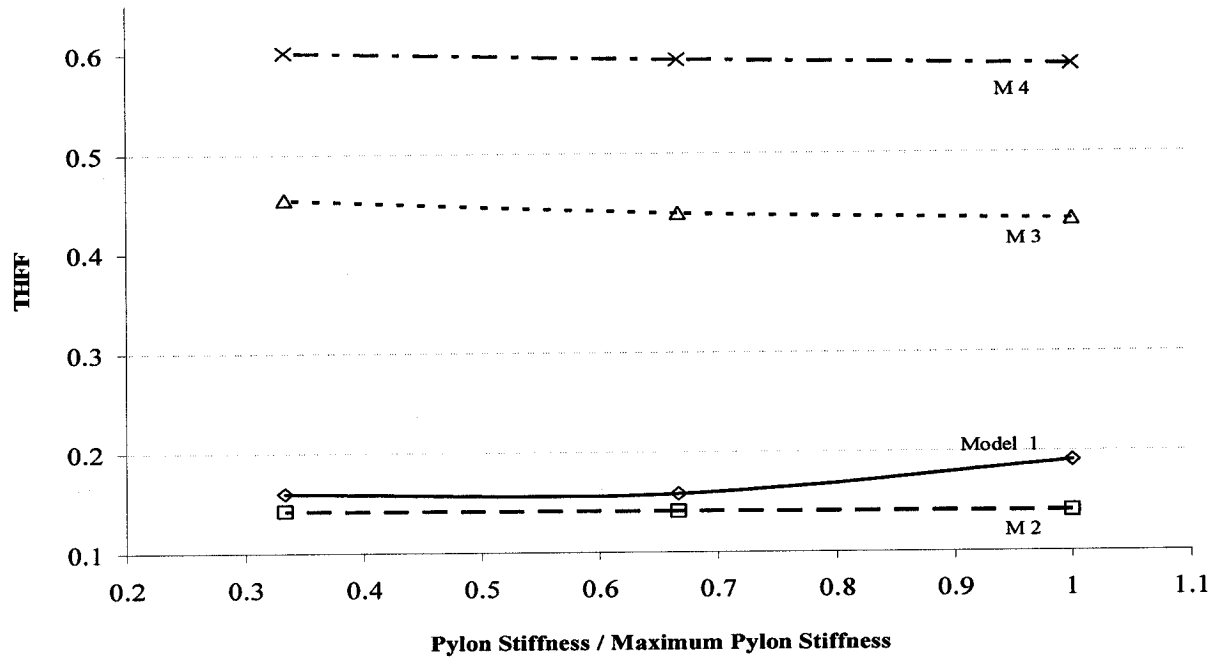


Figure 6.4.2.9 Pylon Stiffness Effect on maximum THFF- Vertical Laminate of Models I, II & III and Right Tubular Laminate of Model IV- LDFA-a, Tha, LLA, Tatara Bridge

### 6.4.3 Effect of Pylon stiffness on the Vertical Natural Frequencies:

The vertical natural frequencies increase in general with the increase of the pylon stiffness, with some exceptions. The rate of the increase is reduced as the pylon stiffness increases. The exceptions are: (i) with the longest central span, in Model I, the first and Second VNF increase as the pylon stiffness increases to stage II, then they reduce slightly as the pylon stiffness increases to stage III (Figures 6.4.3.3 and 6), (ii) with the medium central span, in Model III, the first and Second VNF increase as the pylon stiffness increases to stage II, then reduces greatly as the pylon stiffness increases to stage III (Figures 6.4.3.2 and 5), (iii) with the shortest central span, in Model IV, the first VNF increases as the pylon stiffness increases to stage II, then reduces greatly as the pylon stiffness increases to stage III (Figure 6.4.3.1).

The general increase of the VNF with the increase of the pylon stiffness, which is common behavior in around 85% of the examined cases, is due to the enhancement of the elastic support provided by the pylons through the stay system to the deck. However, the increase of the pylon stiffness significantly changes its mode shape of vibration and

results in significant changes in the stay system vibration mode shape, and the distribution of the pylons restraining effect on the deck through the stay-cable system. This would lead to a change in the deck mode shape of vibration. The deck mode shape is also a function of the deck stiffness and the cable-stayed bridge configuration and boundary conditions, particularly the ratio between the central span to the side span and the location of the deck external supports. Figure 6.4.3.10 shows such changes of different bridge components mode shapes of vibration due to increase of the pylon stiffness, for the Annacis Island Bridge, Model IV. The natural frequency reflects those changes; it always gives an increase when the pylon stiffness increases to the medium level or stage II, then according to the deck stiffness, it either continues increasing as for most of the cases or tends to decrease in some cases. The bridge models with low vertical flexural stiffness, Model II and IV, are affected by the pylon vibration and show the increase-decrease pattern for the shortest span length (Figure 6.4.3.1). Model III has a medium level (close to high) of vertical flexural stiffness, and shows the same variation pattern of its VNF for the medium bridge central span length (Figure 6.4.3.2). Model I has the highest vertical flexural stiffness and shows the same increase-decrease pattern of its VNF with less variation for the longest bridge central span length (Figure 6.4.3.3). On the other hand, the second VNF shows much less change (Figures 6.4.3.4 to 6), and the effect of the pylons' mode of vibration disappears with the third and higher VNF for all models and all bridge central span lengths (Figure 6.4.3.7 shows typical third VNF results for different models).

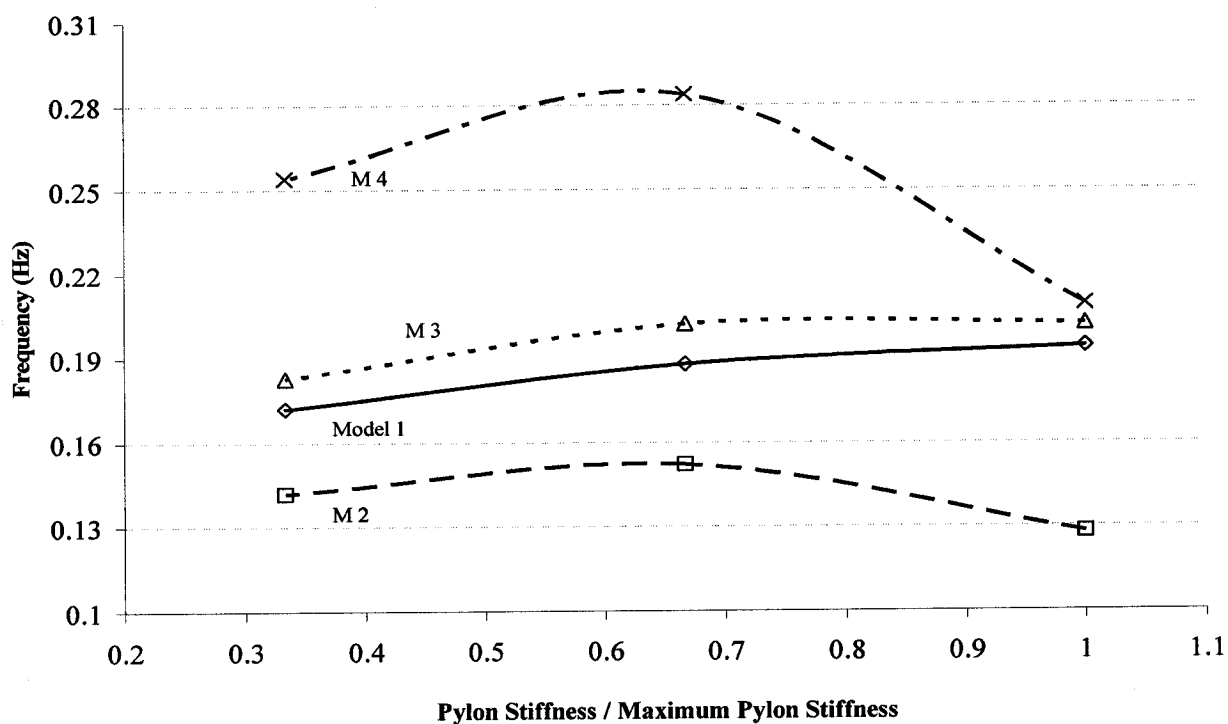


Figure 6.4.3.1 Pylon Stiffness effect on 1st Vertical Natural Frequencies, for different Models, LDFA-a, Tha, Annacis Island Bridge

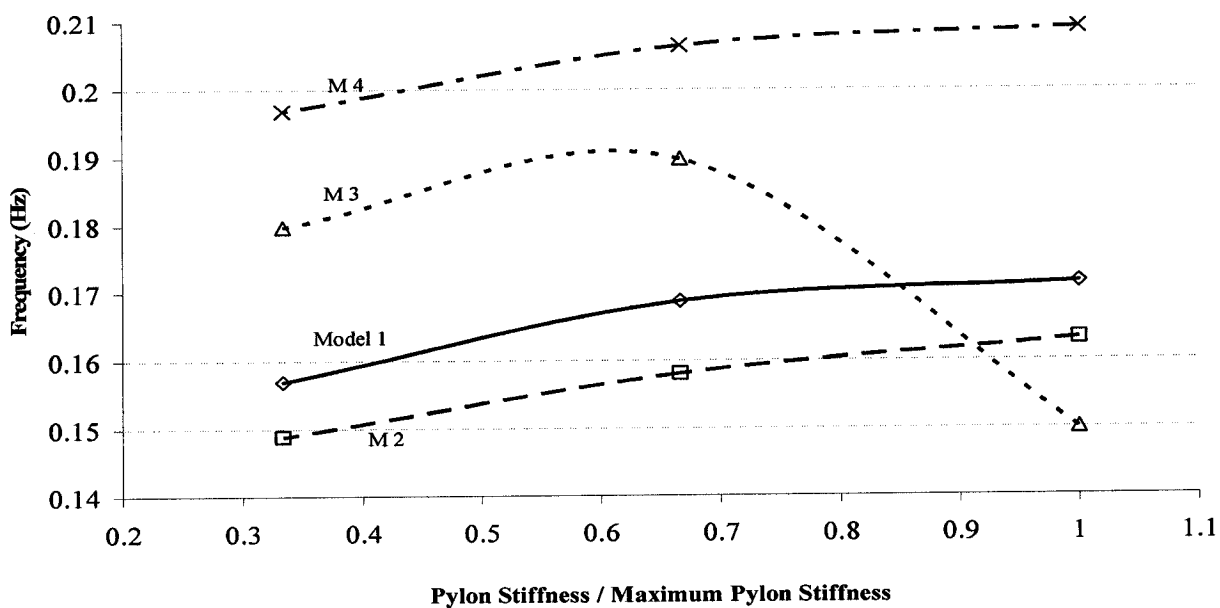


Figure 6.4.3.2 Pylon Stiffness effect on 1st Vertical Natural Frequencies, for different Models, LDFA-a, Tha, Second Chang River Bridge

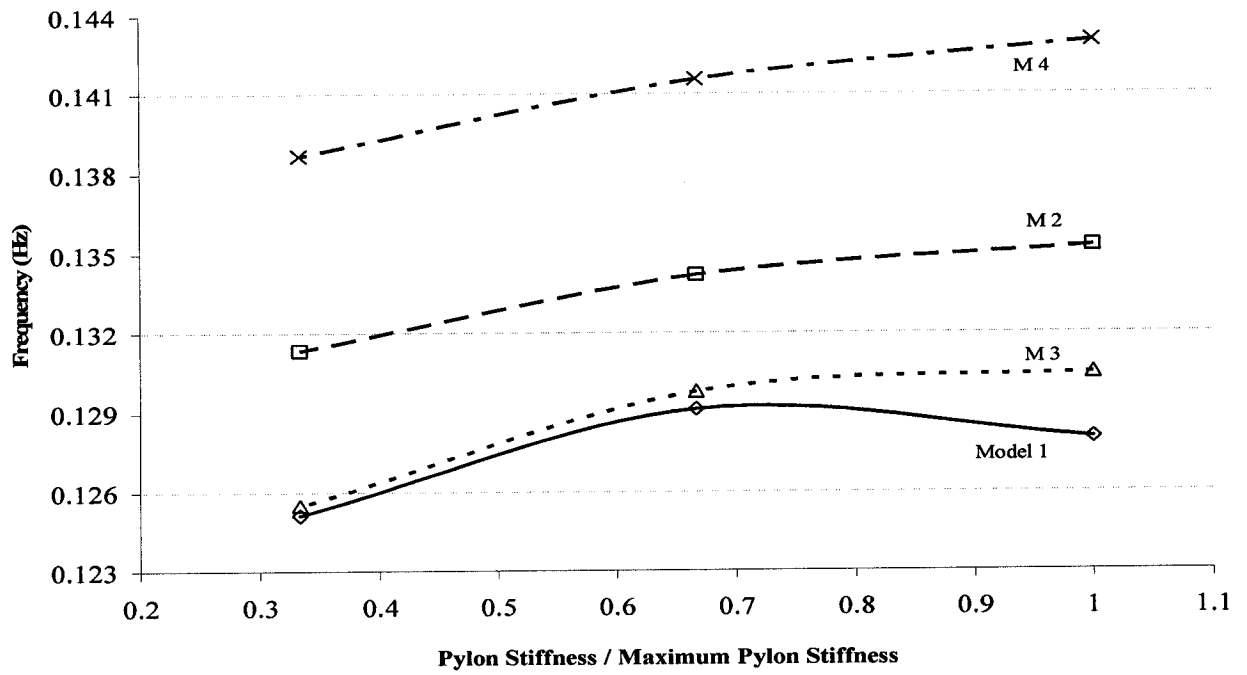


Figure 6.4.3.3 Pylon Stiffness effect on 1st Vertical Natural Frequencies, for different Models, LDFA-a, Tha, Tatara Bridge

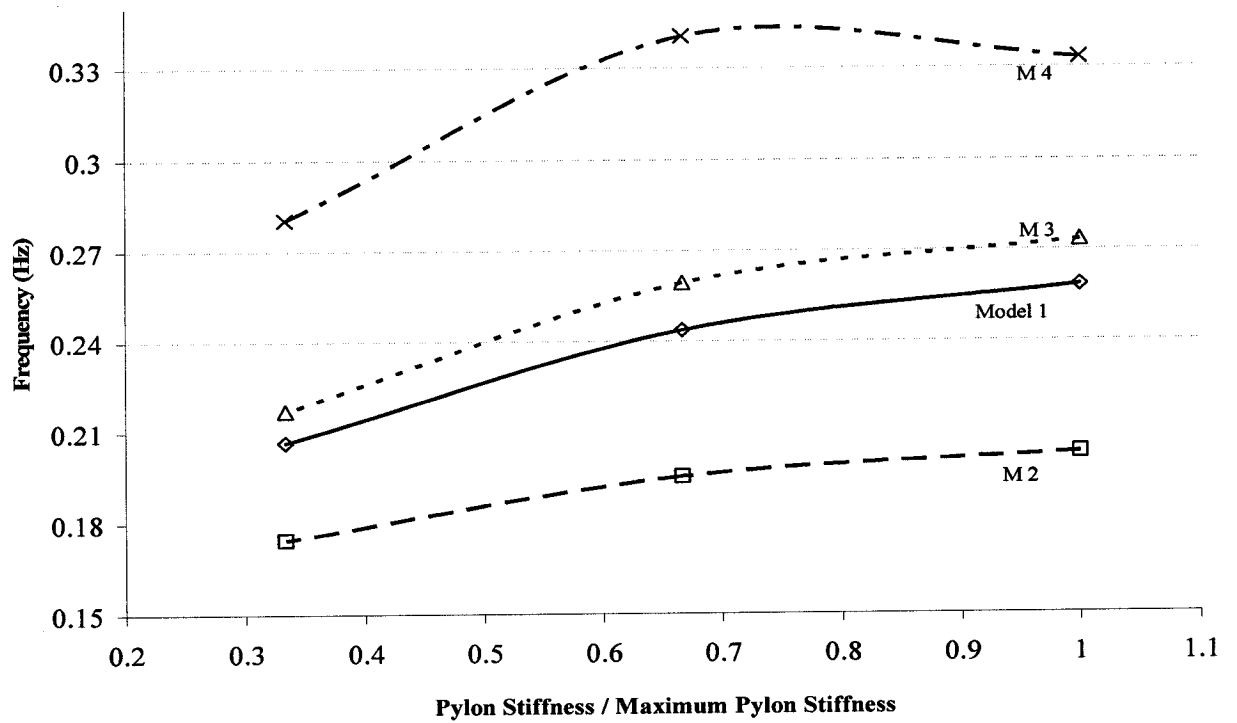


Figure 6.4.3.4 Pylon Stiffness effect on 2nd Vertical Natural Frequencies, for different Models, LDFA-a, Tha, Annacis Island Bridge

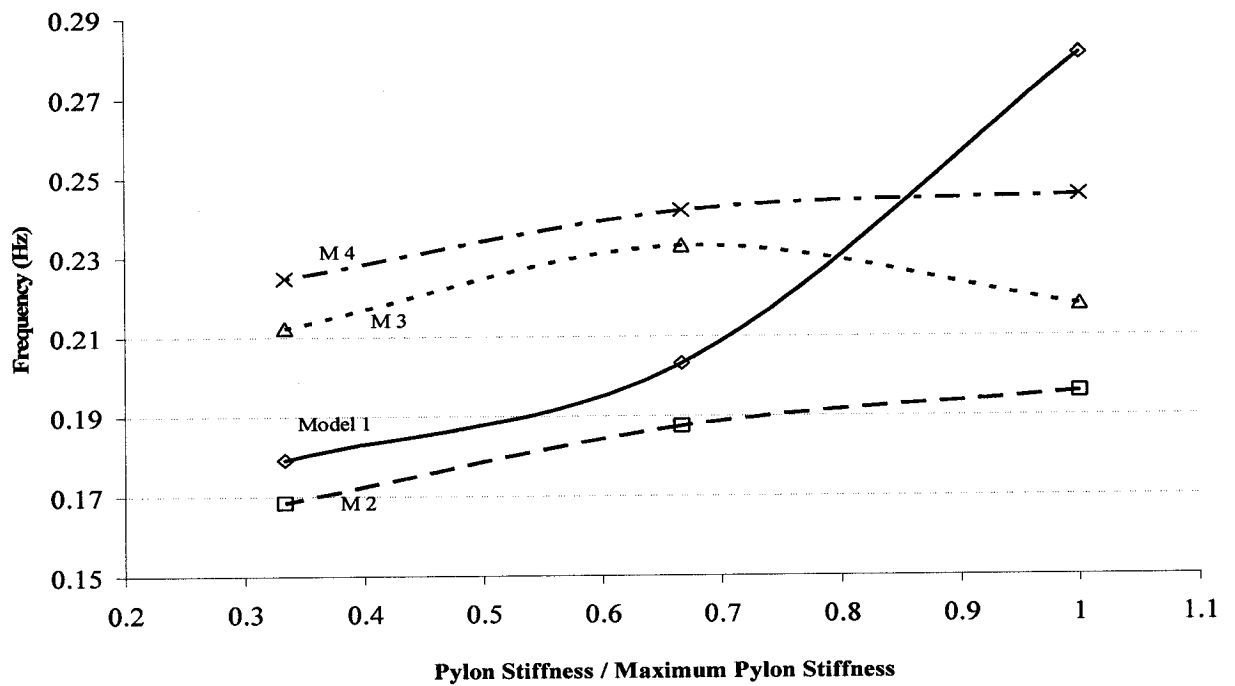


Figure 6.4.3.5 Pylon Stiffness Effect on Second Vertical Natural Frequencies, for different Models, LDFA-a, Tha, Second Chang River Bridge

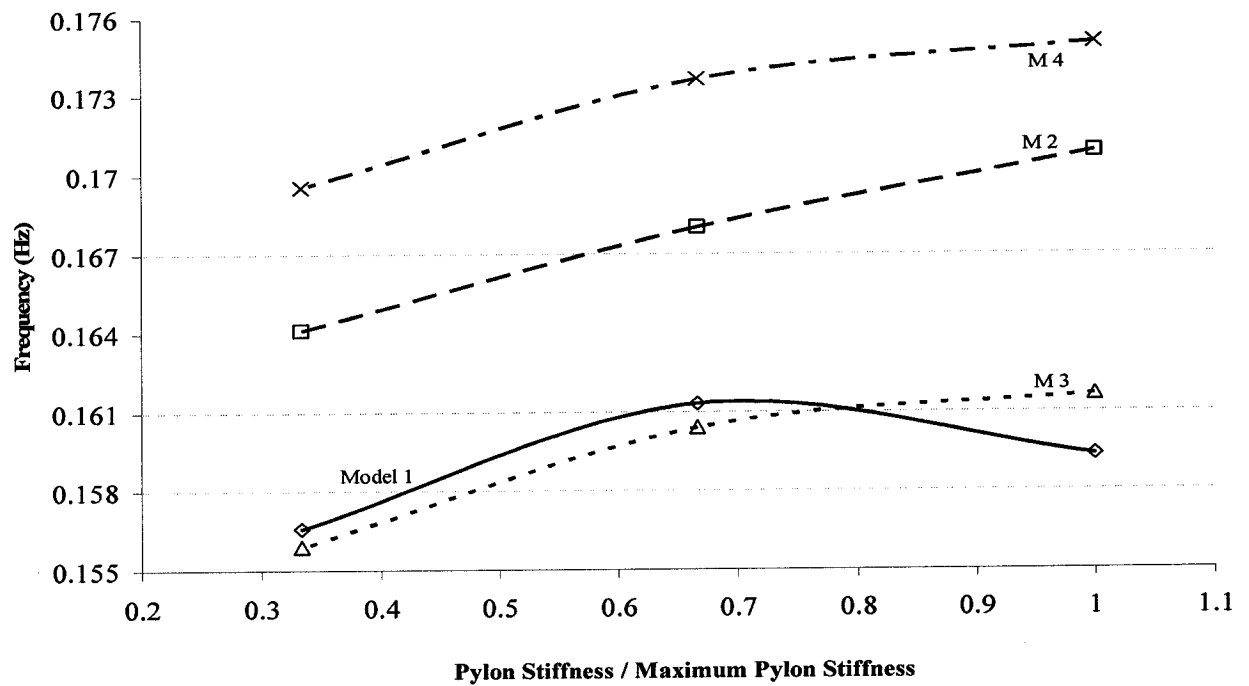


Figure 6.4.3.6 Pylon Stiffness effect on 2nd Vertical Natural Frequencies, for different Models, LDFA-a, Tha, Tatara Bridge

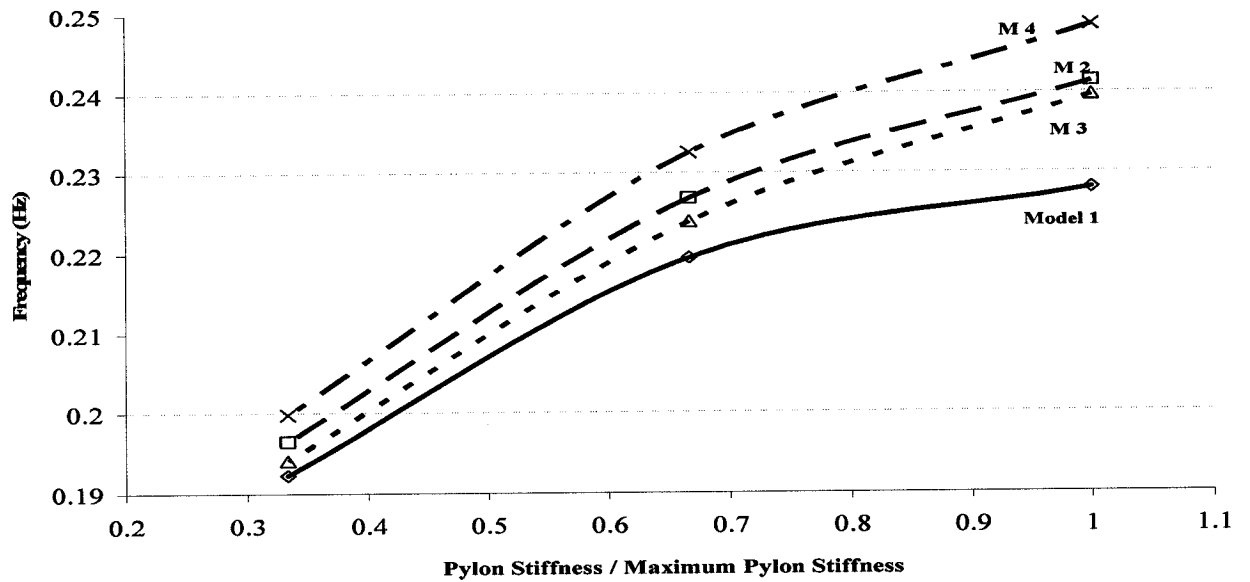


Figure 6.4.3.7 Pylon Stiffness Effect on 3rd Vertical Natural Frequencies, for different Models, LDFA-a, Tha, Tataru Bridge

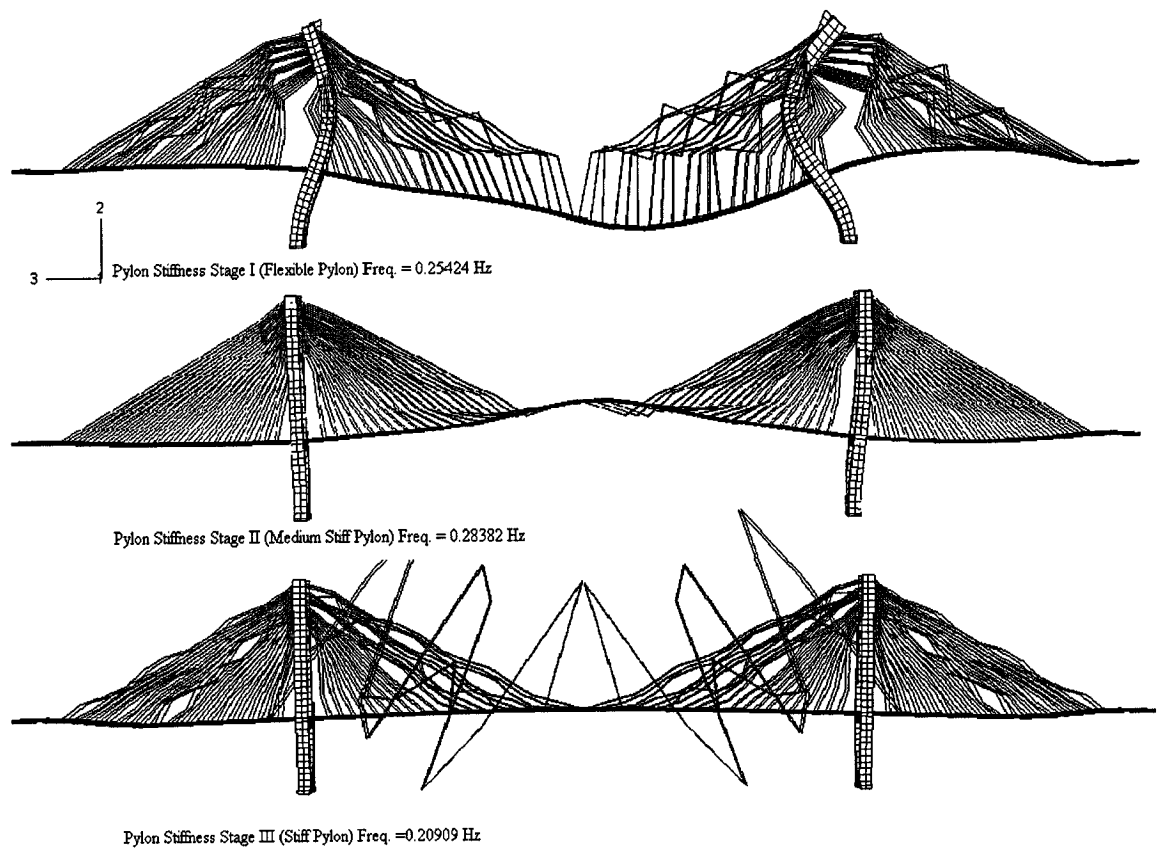


Figure 6.4.3.8 Pylon Stiffness Effect on the mode shape of the first Natural frequency of deck, pylons and cable of Model IV (side view), LDFA-a, Tha, the Annacis Island Bridge



#### **6.4.4 Effect of Pylon stiffness on the Torsional Natural Frequencies:**

All the Torsional Natural Frequencies (TNF) for Models I, II and III increase as the pylon stiffness increases; and sometimes the increase is very high. The high increase of the torsional natural frequencies and the smoother increase of the vertical natural frequencies (shown in the previous section) with the increase of the pylon stiffness, result in an increase of the TNF to VNF ratios. This would improve the dynamic performance and the aerodynamic response of different models.

The increase in the elastic supporting restraints of the cable stays due to the increase of the pylon stiffness explains the high increase in the TNF of all the models. The change of the pylon vibration mode shape does not appear to have any effect on the torsional natural frequencies. The cable stays work as supports to the two sides of the deck (in the transverse direction) and the enhancement of those elastic supports increases the deck TNF.

Different models show different responses depending on the ratio of their torsional stiffness to their mass for different bridge spans. While Model I, which has high torsional stiffness and highest mass, shows the best improvement with the shortest bridge central span (Figures 6.4.4.1 and 4), and it gives relatively less changes as the bridge central span length increases (Figures 6.4.4. 3 and 6). Model II has the lowest torsional stiffness and the lowest mass, and it gives the lowest improvement of its TNF with the shortest central span length (Figures 6.4.4.1 and 4). However, the increase of its TNF improved for the medium central span length (Figures 6.4.4.2 and 5), and further improved as the bridge central span became very long, with the Tatara Bridge (Figure 6.4.4.3 and 6). With the Second Chang River Bridge, where the deck is wider than other bridges, Model III appears to be more responsive to the increase of the pylon stiffness, and it shows high increase in its TNF (Figures 6.4.4.2 and 5).

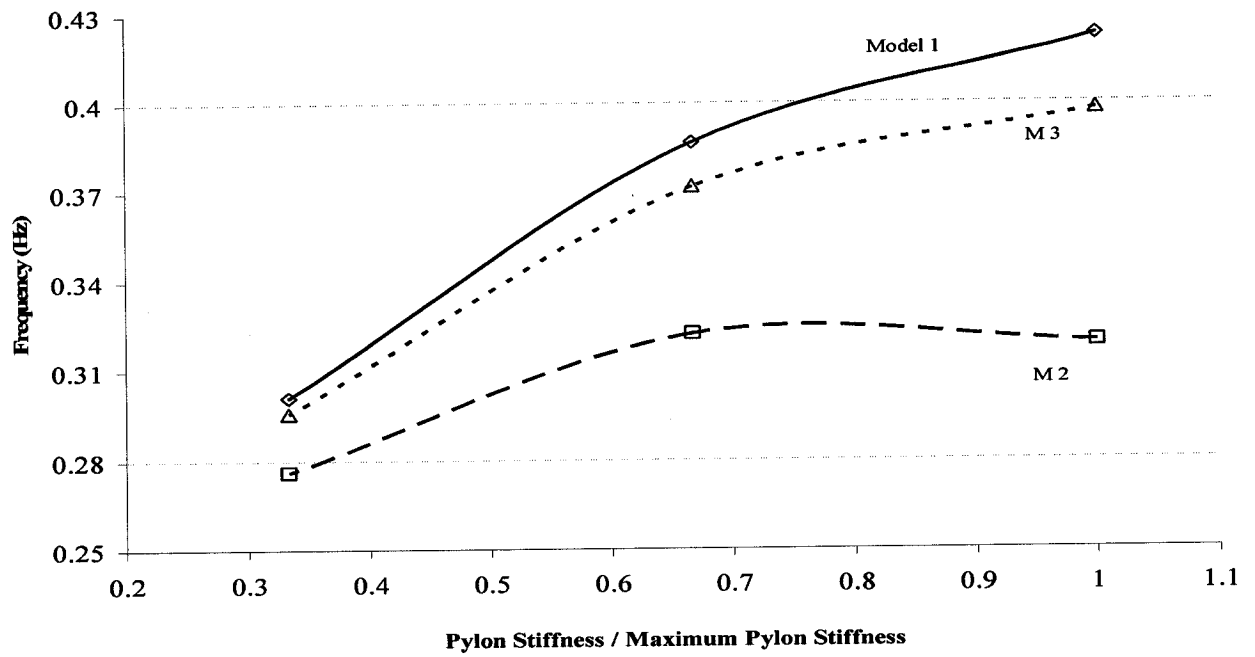


Figure 6.4.4.1 Pylon Stiffness Effect on 1st Torsional Natural Frequencies, for different Models, LDFA-a, Tha, Annacis Island Bridge

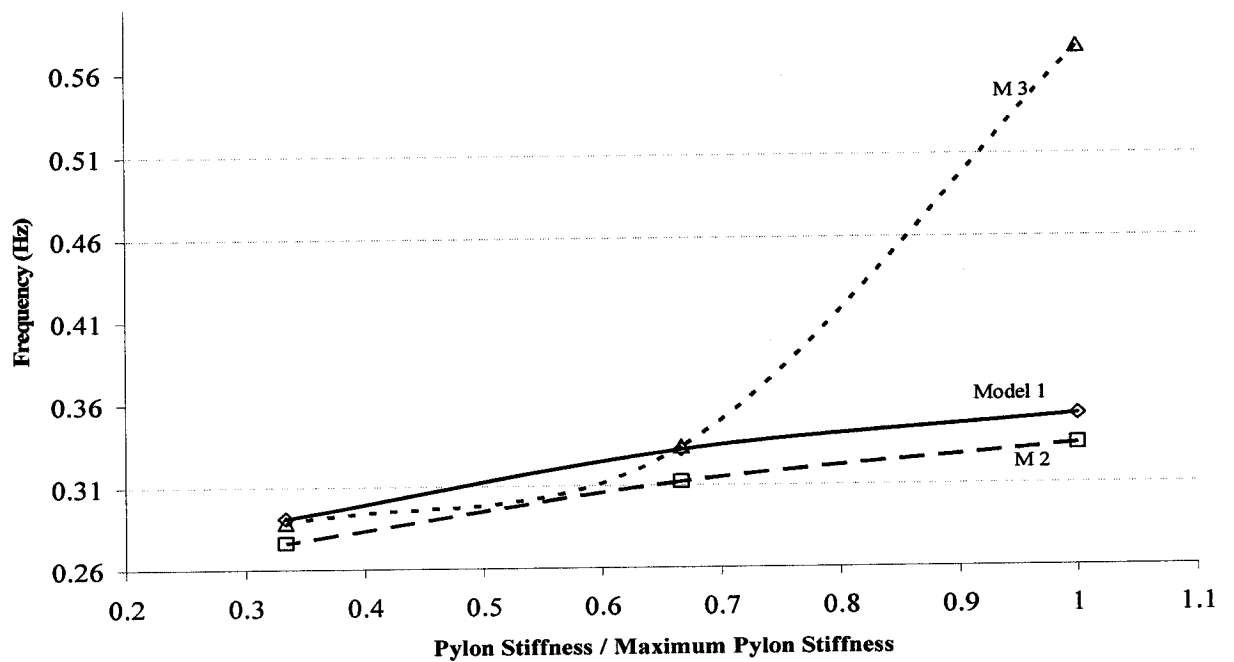


Figure 6.4.4.2 Pylon Stiffness effect on 1st Torsional Natural Frequencies, for different Models, LDFA-a, Tha, Second Chang River Bridge

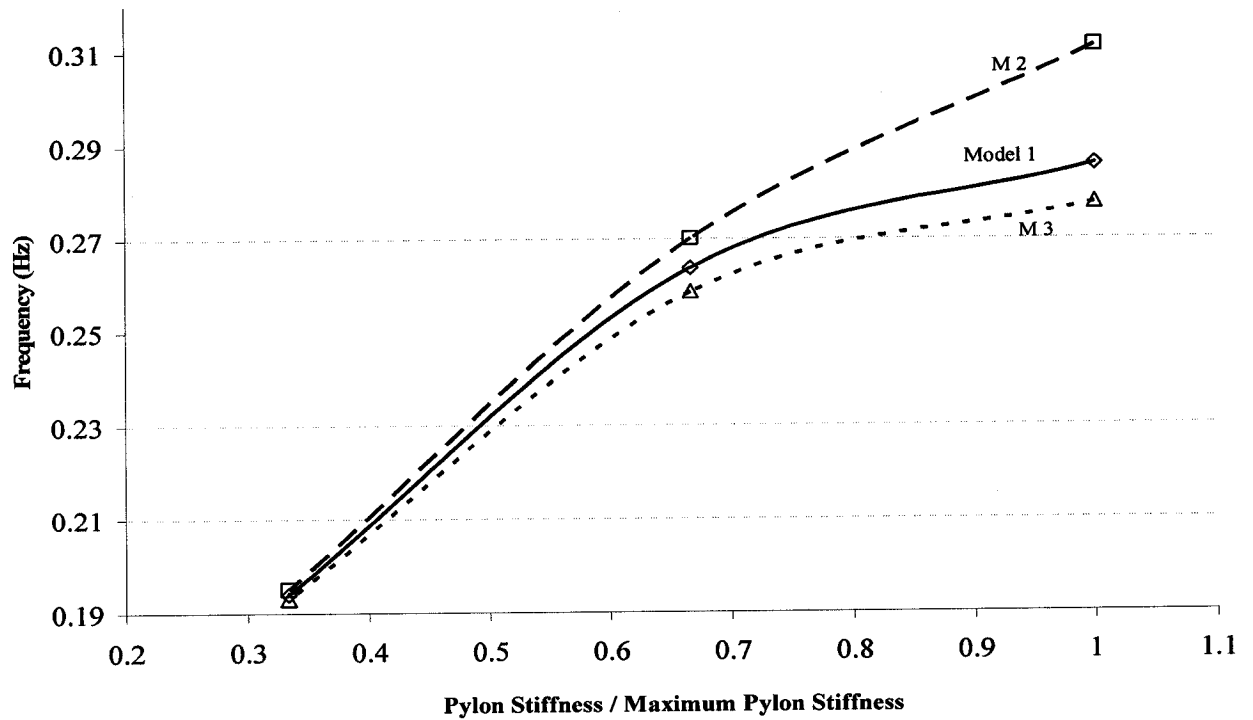


Figure 6.4.4.3 Pylon Stiffness effect on 1st Torsional Natural Frequencies, for different Models, LDFA-a, Tha, Tataru Bridge

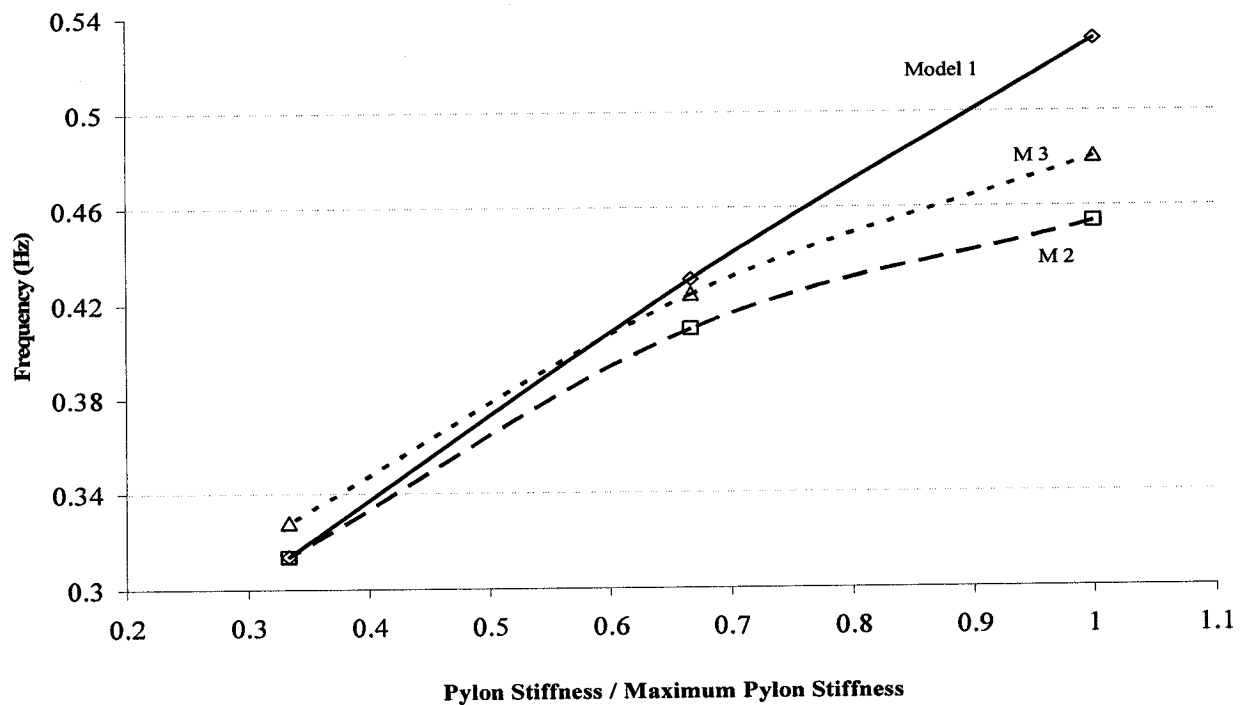


Figure 6.4.4.4 Pylon Stiffness effect on 2nd Torsional Natural Frequencies, for different Models, LDFA-a, Tha, Annacis Island Bridge

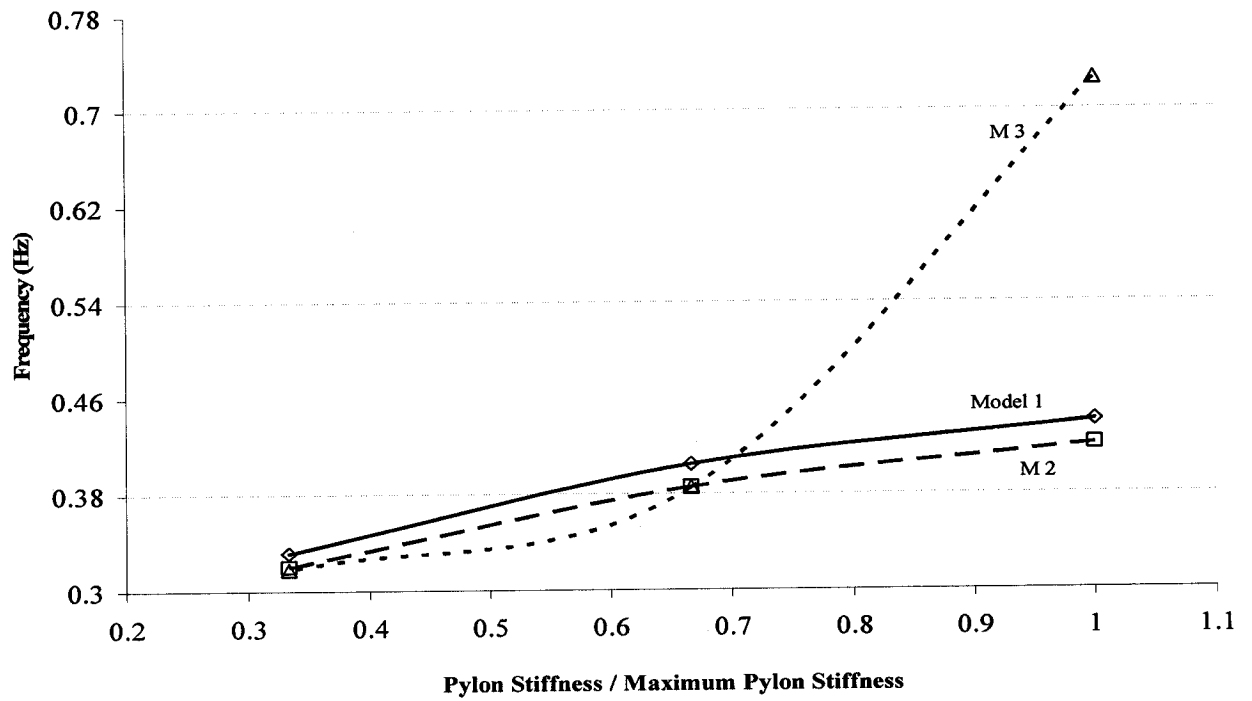


Figure 6.4.4.5 Pylon Stiffness Effect on 2nd Torsional Natural Frequencies, for different Models, LDFA-a, Tha, Second Chang River Bridge

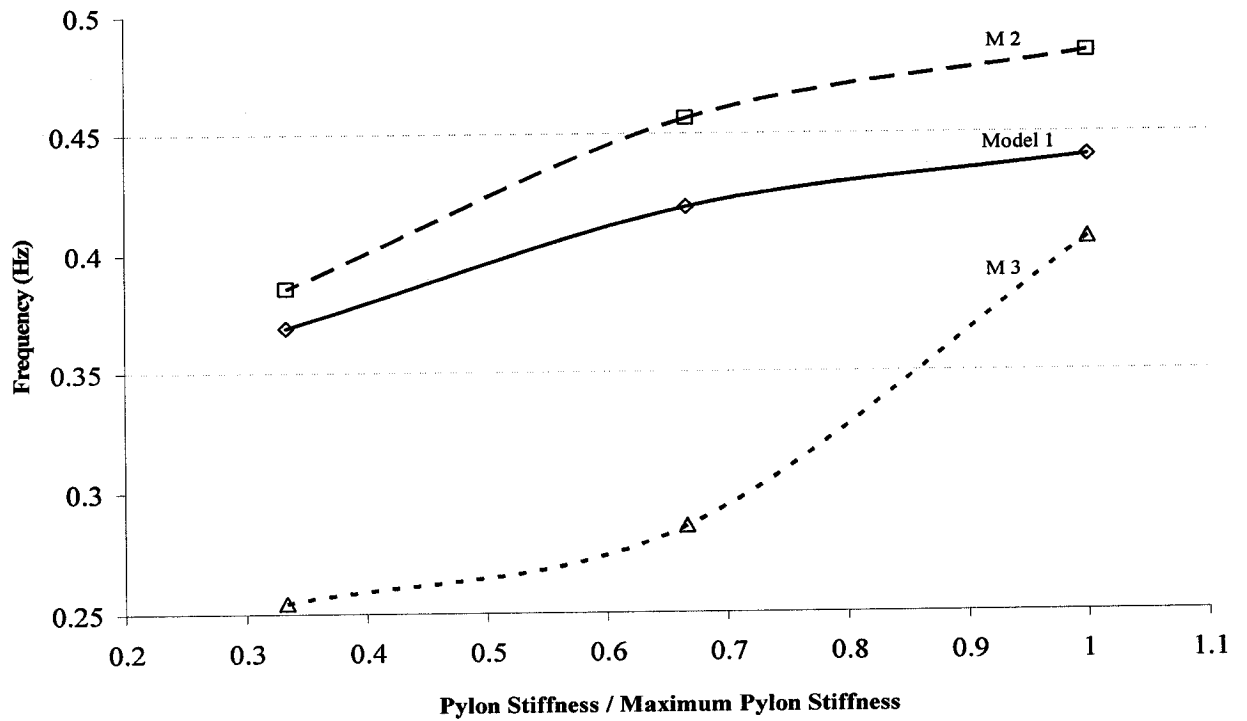


Figure 6.4.4.6 Pylon Stiffness effect on 2nd Torsional Natural Frequencies, for different Models, LDFA-a, Tha, Tatara Bridge

#### 6.4.5 Effect of Pylon stiffness on the Horizontal Natural Frequencies:

It has been found that the Horizontal Natural Frequencies are constant or show slight increases as the pylon stiffness increases (Figures 6.4.5.1 to 3). The horizontal (or lateral) flexural stiffness of all models are very high, and cause the low changes of the HNF due to the increase of the pylon stiffness.

In Model I; the 1stHNF shows a relatively higher increase with the medium and the longest central span lengths, as the pylon stiffness increases from stage II to stage III. The high mass of this model, in comparison with other models, makes the balance between its horizontal stiffness and mass contributions to the HNF more sensitive to the increase of the pylon stiffness (Figures 6.4.5.2 and 3). Model IV has low lateral stiffness and a low mass and its HNF shows a relative improvement for the shortest span length.

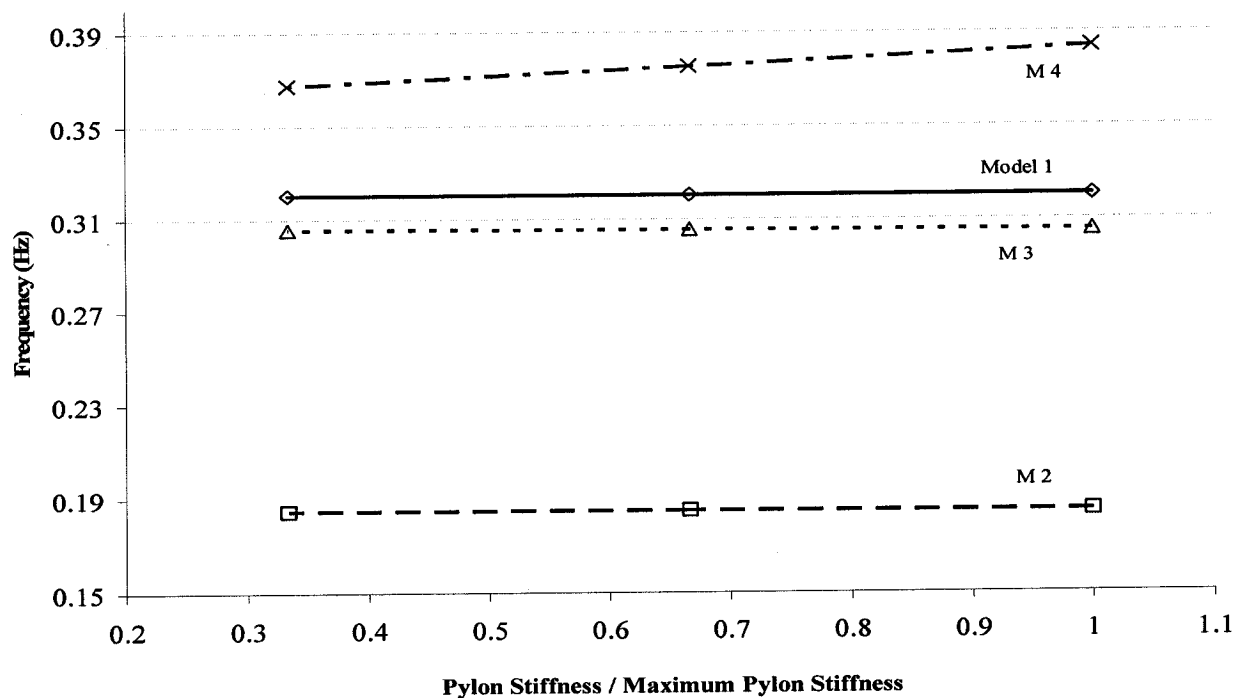


Figure 6.4.5.1 Pylon Stiffness effect on 1st Horizontal Natural Frequencies- for different models- LDFA-a- Tha- Annacis Island Bridge

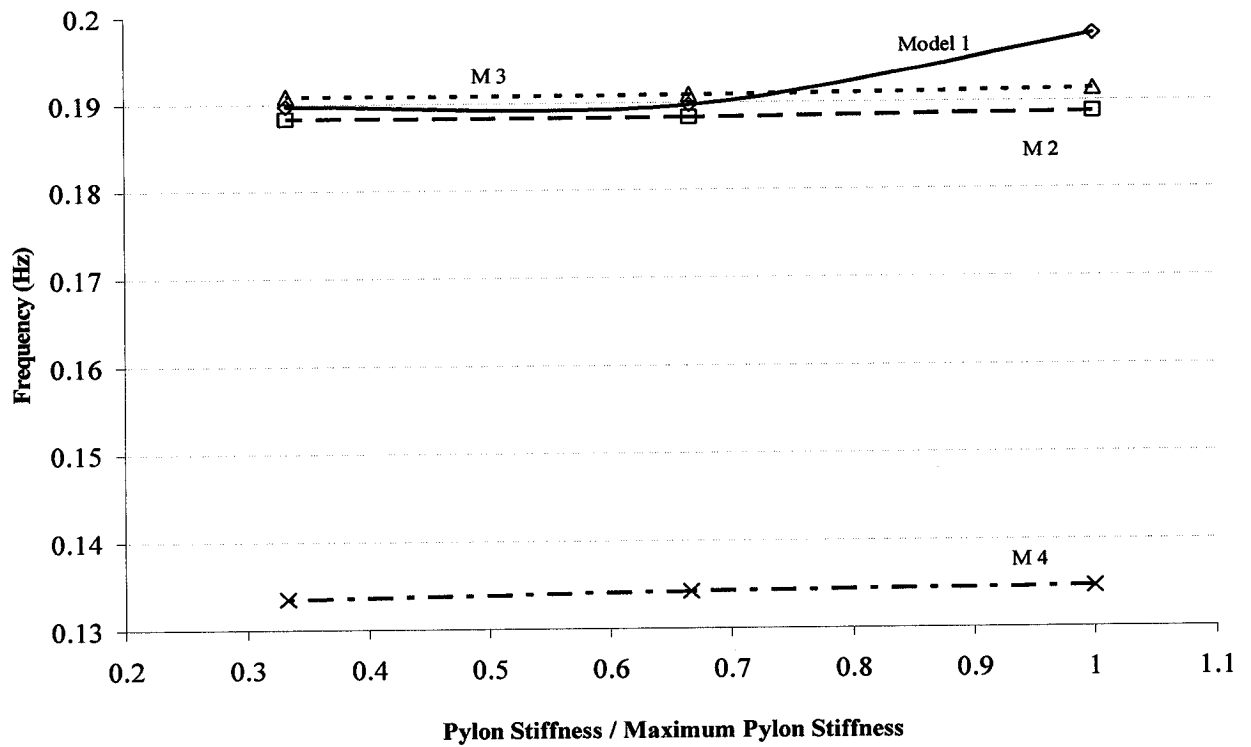


Figure 6.4.5.2 Pylon Stiffness Effect on 1st Horizontal Natural Frequencies- for different Models- LDFA-a- Tha- Second Chang River Bridge

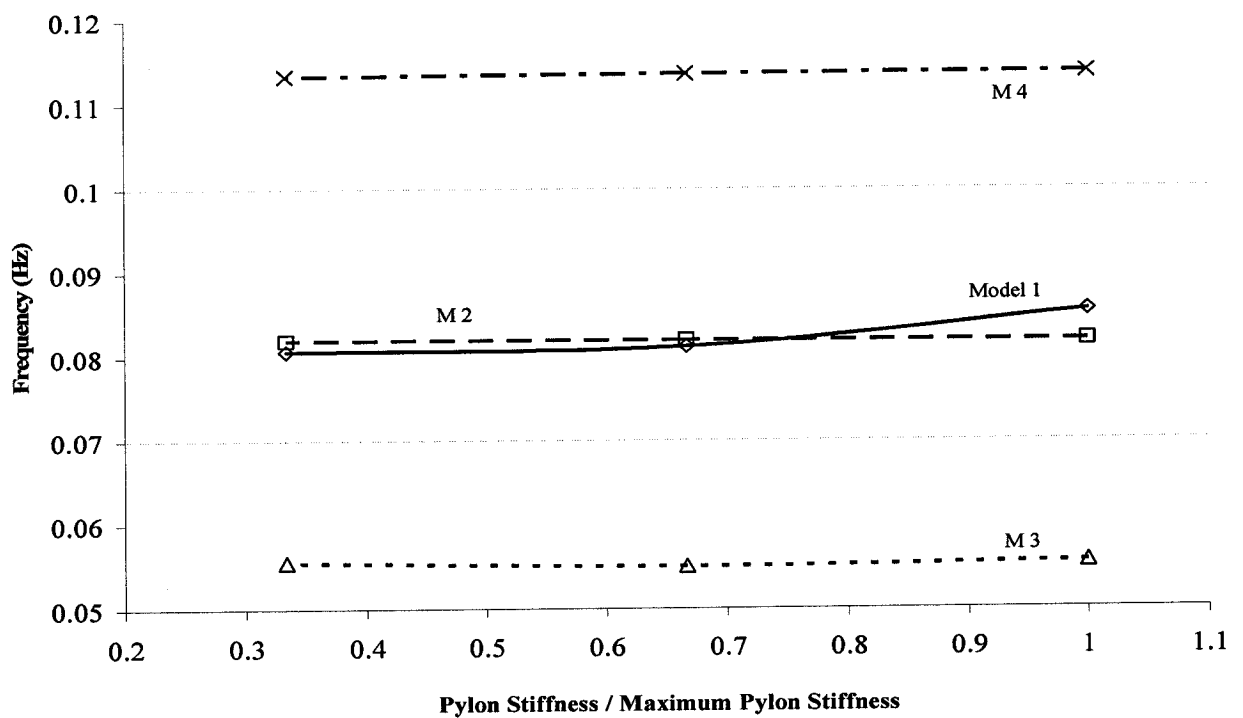


Figure 6.4.5.3 Pylon Stiffness Effect on 1st Horizontal Natural Frequencies- for different Models- LDFA-a- Tha- Tatara Bridge

## 6.5 Aerodynamic Stability:

Structural instability usually appears when the deformed shape of the structure produces an increase of the excitation forces acting on the structure. In the field of aeroelasticity, instability occurs when aerodynamic forces induced by wind are modified as the structure deforms. If the new values of the forces are greater, then the structural displacements are also increased, and this process leads to instable behavior of the structure.

Flutter is an aerodynamic phenomenon caused by the interaction of wind action and bridge vibration. When the input wind energy cannot be dissipated, vibration accumulates and the bridge becomes aerodynamically unstable [Cai et al 2000]. This class of instability can be produced by a single degree of freedom in vertical movements or torsional rotations, but it could happen in the coupled modes of both degrees of freedom [Scanlan 1978]. Figure 6.5.1 shows an example of a flutter phenomenon occurrence for Model III of the Second Chang River Bridge-Model III. Figure 6.5.2 shows an example of the time variation of the rotational degree of freedom of the top line mid point of the deck section at the bridge center.

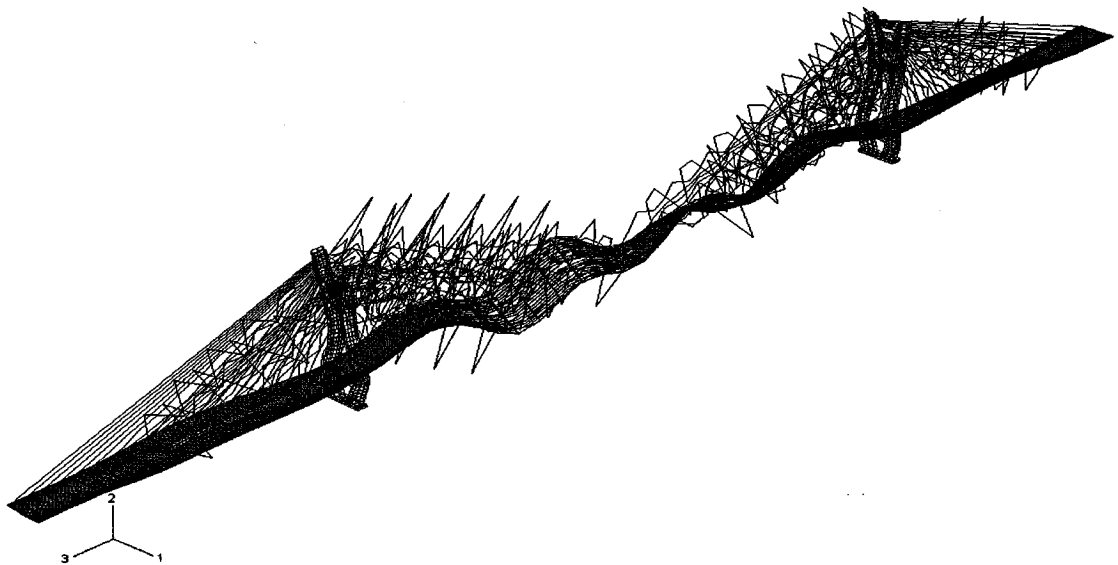


Figure 6.5.1 An example of 3D-deformation shape of the bridge due to the flutter phenomenon, Model III, LDFA-a, Th-a, CCR 0.125m, The Second Chang River Bridge

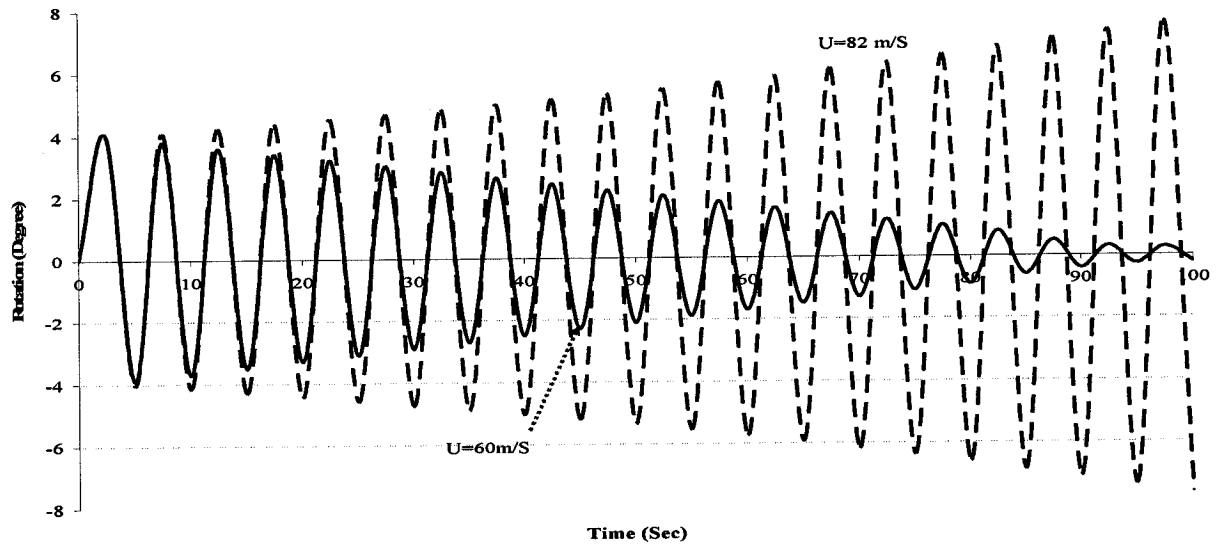


Figure 6.5.2 An example of time history of the rotational degree of freedom of the top mid point of the deck section vertical centerline at the of the bridge central span, for different wind speeds, Model III, LDFA-a, Th-a, CCR 0.125m, The Second Chang River Bridge.

The main purpose of this section is to evaluate the influence of different parameters on the critical flutter speeds of the proposed Hybrid Long-Span Cable-Stayed Bridge. The studied variables include the Laminas Dominant Fiber Alignment, LDFA, the Laminate thickness and CFRP Cable radius. The effects of those variables on the critical flutter speeds are investigated for different models and different bridge central spans.

The time domain flutter analysis is adopted in this research. The procedure is uneconomical due to the huge computational time required to perform it, however it can involve different nonlinearities of the structural system and/or aerodynamic nonlinearities [Chen et al. 2001]. The procedure is a coupled multimode flutter analysis, which introduces self-excited aerodynamic forces in terms of rational function approximations [Matsumoto et al. 1994, Chen et al. 2000]. This has led to a convenient transformation of the equations into a frequency independent state-space format as shown in Chapter 5. In this research all types of structural nonlinearity are involved while the nonlinearity of the wind is not.



The flutter derivatives collected from wind tunnel section tests for the Second Chang River Cable-Stayed Bridge [Tongji University State Key Laboratory Report, 1999] for the “completed condition” and  $0^\circ$  angle of attack are used in this research for Models I, III and IV, while the flutter derivatives of the very thin section (plate section) [Scanlan 1978] are used for Model II. In this investigation the frequency independent coefficients (i.e  $A_1$ ,  $A_2$ ,  $A_3$ , ...etc) are evaluated by means of least-squares method using the experimentally obtained flutter derivatives at different reduced frequencies. Consequently, the self-excited forces are then defined as frequency independent functions in the time domain finite element procedure.

The following assumptions are employed in the present investigation:

1. The damping ratio, which includes the structural and the aerodynamic damping, is assumed to be constant and equal 2% for all the four models and all the investigated spans. Although the aerospace structures made from advanced composite materials proved to have a high damping ratio, the damping properties are evaluated for limited size structures only. Hence, this conservative assumption is made because the proposed bridge system has an extremely huge size in comparison to the aerospace structures and, the proposed micro/macro arrangement is not used in the aerospace structures.
2. Because of the availability of wind tunnel test results, the flutter derivatives of Second Chang River Bridge are also used for the other two bridges (the Annacis Island Bridge and the Tatara Bridge).
3. The difference between the boundary of Model III deck section and the Second Chang River reference bridge deck section are ignored, specifically the rounded intersection of the bottom horizontal edge and the lower inclined side edges.
4. Although the Model IV deck section configuration is different from that of the reference Second Chang River bridge deck section, the boundary dimensions of both sections are very close. The effect of this difference on the flutter derivatives are disregarded in this investigation. The Model IV section has two side entities (or boxes) having a similar top and side boundary lines as the reference bridge deck section. However the circular sector (or the apparent lower part of the tubular laminate), and the thin intermediate “slab-and-grid” part represent major divergences of the Model IV deck section from the reference deck sections.

It has been found that the critical flutter speeds of the proposed hybrid long span bridge cable stayed bridge is lower than that of the reference bridge (the Second Chang River Bridge) for all the proposed models. Model IV, which has the best results, has a maximum flutter speed of approximately 111 m/s, which is only 63.4% of the wind tunnel critical flutter speed of the reference bridge (175 m/s) [Tongji University State Key Laboratory Report, 1999]. The best results are mostly related to low levels of laminas dominant fiber alignment (LDFA-a) and the medium laminate thickness (or Laminate Thickness-b). Model II gives the lowest and Model I gives the second lowest critical flutter speeds, while Model III gives the second highest critical flutter speeds. For all the examined cases, the pylon stiffness is assumed constant and equal to the stiff pylon situation.

Explaining the relation between the present numerically evaluated flutter speeds and the complicated dynamic behavior of different models of the proposed hybrid bridge is very complicated. However, the ratio between the torsional to the vertical flexural frequencies can be a key in this situation. On the other hand, for the proposed hybrid long span cable stayed bridge, it has been found that the coupling of different frequencies never happens with Model III. Model IV shows coupling of the HNF with the VNF for the high frequencies (e.g. Figure 6.2.5.5.5) and never occurs between the VNF and the TNF. Model I shows the coupling between the VNF and the TNF for high frequencies (sixth and higher), while Model II gives the coupling between its VNF and the TNF, starting from relatively lower frequencies (fourth and higher, e.g. Figure 6.2.5.4.12). In the time that the coupling between the VNF and the HNF does not affect the flutter instability, the coupling of the VNF and the TNF directly contributes in this instability. This could explain the low critical flutter speeds of Models II.

### **6.5.1 Effect of Laminas Dominant Fiber Alignment on the Critical Flutter Speed:**

It has been found that the increase of the Laminas Dominant Fiber Alignment results in slight changes of the Critical Flutter speeds (CFS) of different models. The increase of LDFA affects some high vertical flexural and torsional frequencies, while their effects on the first and second natural frequencies are very limited. Consequently, the ratio between

the first torsional to the first vertical flexural frequencies are not affected by LDFA increase. The effects on similar ratios of higher frequencies, which are basically very high, are limited. As a result, the flutter speeds give slight changes in general. The variation of CFS due to the increase of LDFA becomes even lower as the laminate thickness increases.

Models I, II and IV show slight decrease in their CFS as opposed to Model III that shows a slight increase, as LDFA increases (Figures 6.5.1.1 to 6.5.1.3 show the highest CFS with LTh-a). Model II appears to be the most sensitive compared to other models. The sensitivity of different models to LDFA increase is also reduced as the bridge span length is increased to the maximum examined length, with the Tataru Bridge.

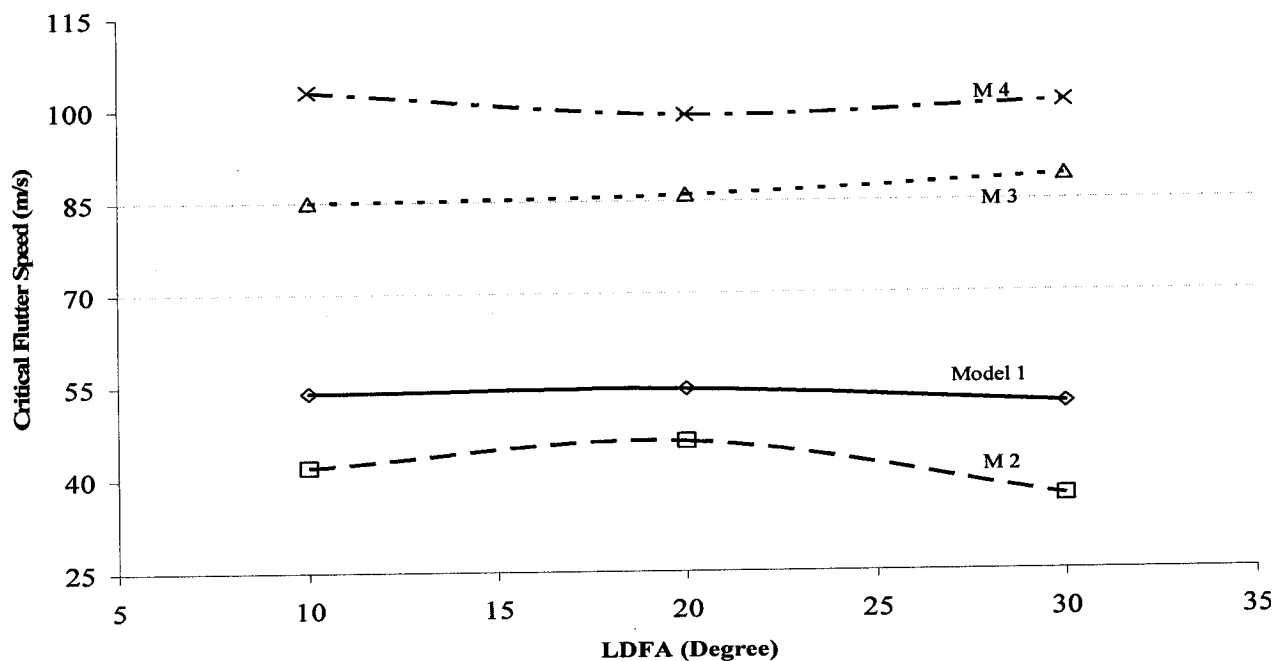


Figure 6.5.1.1 Laminas Dominant Fiber Alignment effect on Critical Flutter Speed, for different Models, Tha, CCR 0.100m, Annacis Island Bridge

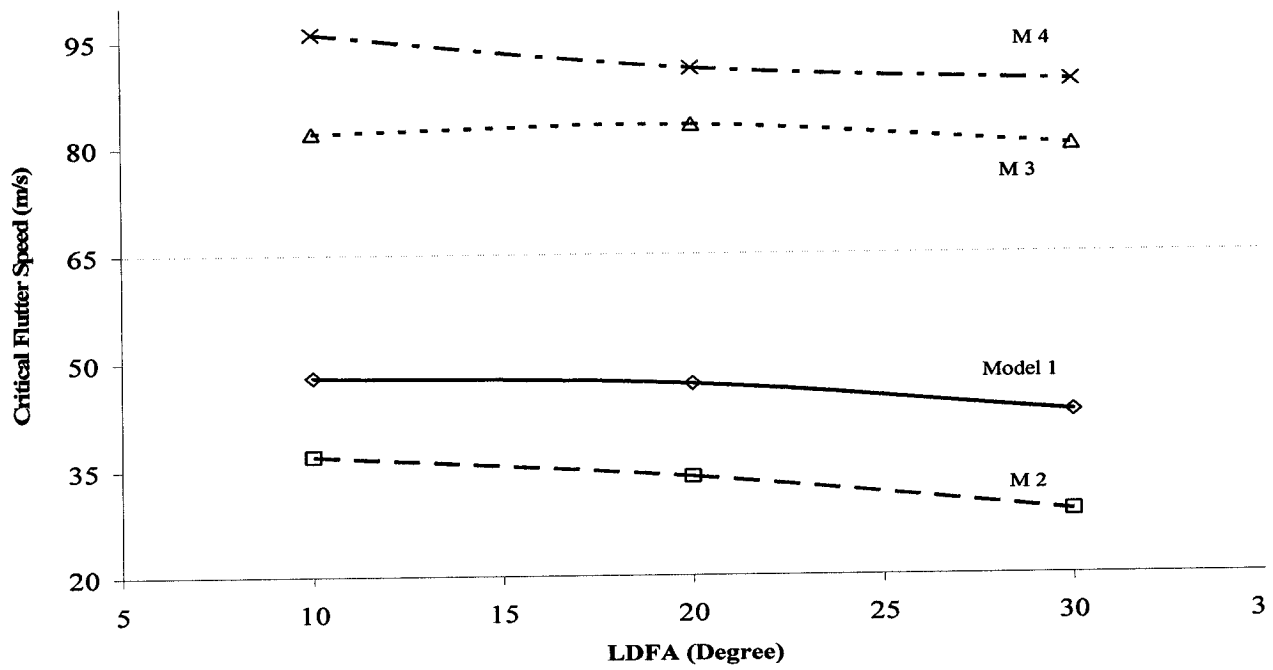


Figure 6.5.1.2 Laminas Dominant Fiber Alignment effect on Critical Flutter Speed, for different Models, Tha, CCR 0.125m, Second Chang River Bridge

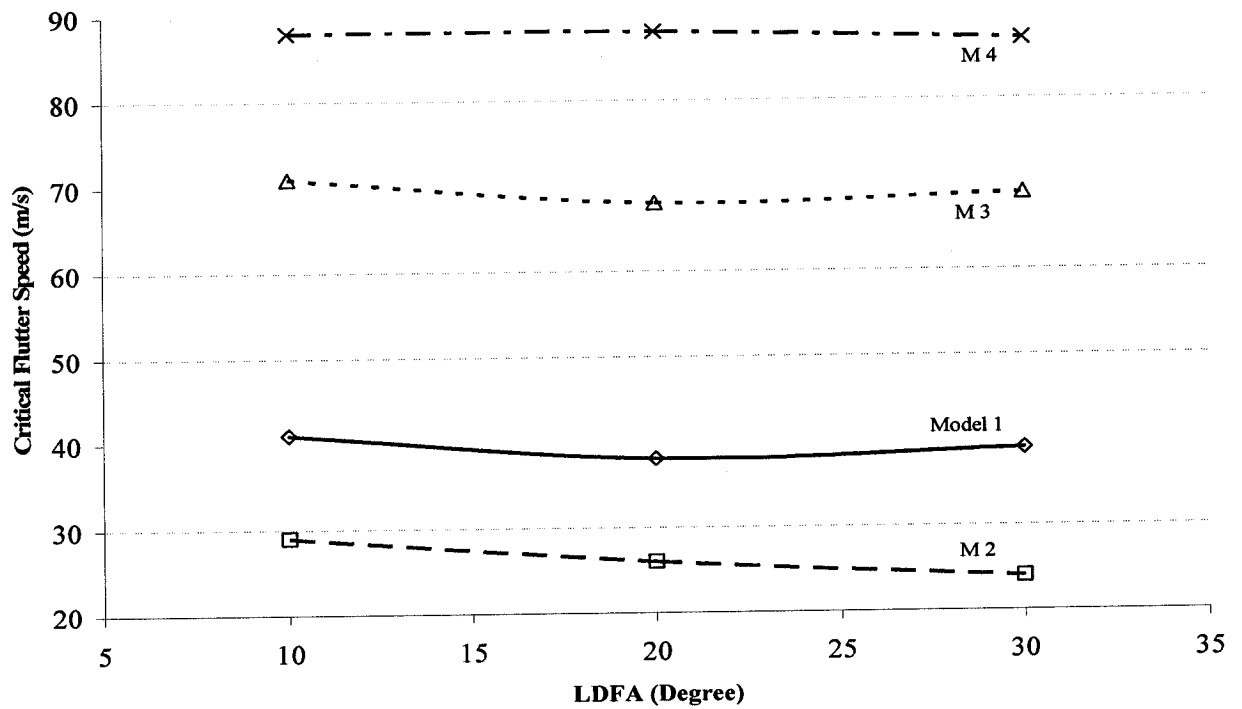


Figure 6.5.1.3 Laminas Dominant Fiber Alignment effect on Critical Flutter Speed, for different Models, Tha, CCR 0.150 m, Tatara Bridge

### 6.5.2 Effect of Laminate Thickness on the Critical Flutter Speed:

As the Laminate Thickness (LTh) increases, it has been found that the Critical Flutter Speeds (CFS) show almost the same pattern; increase in the CFS as LTh increases from the lowest examined thickness to the medium, then decreases with further increase in the laminate thickness. However, the CFS related to the maximum examined laminate thickness is usually higher than that related to the minimum laminate thickness.

As the laminate thickness increases, the vertical natural frequencies are decreased and the torsional natural frequencies are increased in general, as shown in sections 6.2.5.3 and 6.2.5.4. This leads to increase the TNF to VNF ratios, however, with further increase of the laminate thickness, the effect of increasing the mass reduces the changes in both frequencies and leads to reduce their ratio. The advanced composite material has relatively high stiffness to mass ratio, which reduces the rate of decrease of CFS, with further increase of the laminate thicknesses. Figures 6.5.2.1 to 6.5.2.3 show sample results of the effect of the laminate thickness on the critical flutter Speeds of different models and different bridges.

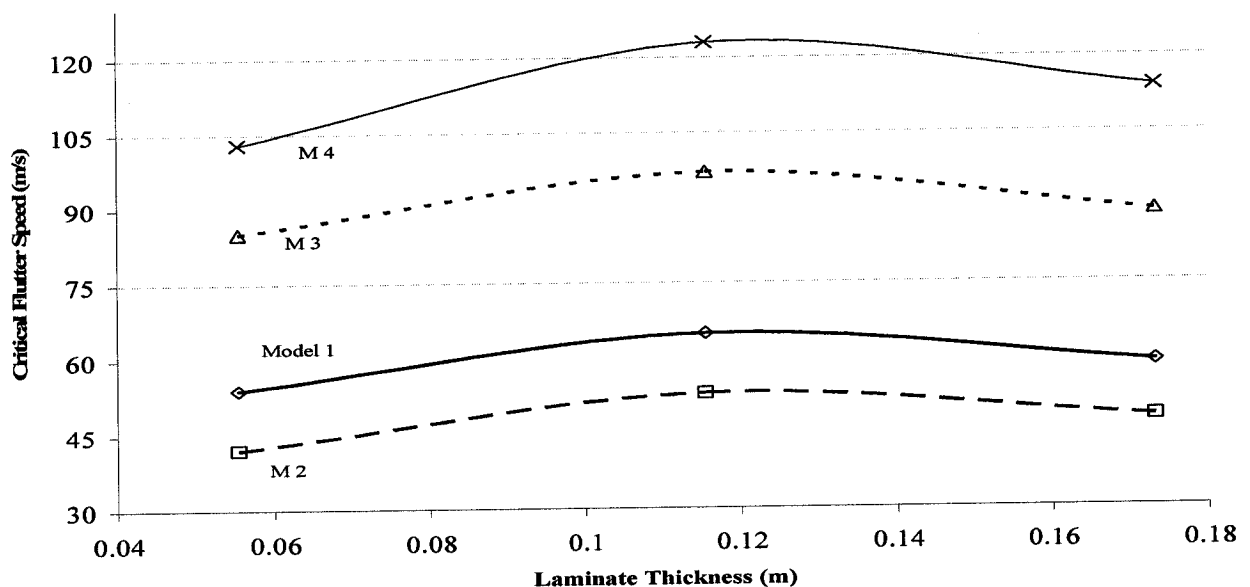


Figure 6.5.2.1 Laminate Thickness Effect on Critical Flutter Speed, for different Models, LDFA-a, CCR 0.100m, Annacis Island Bridge

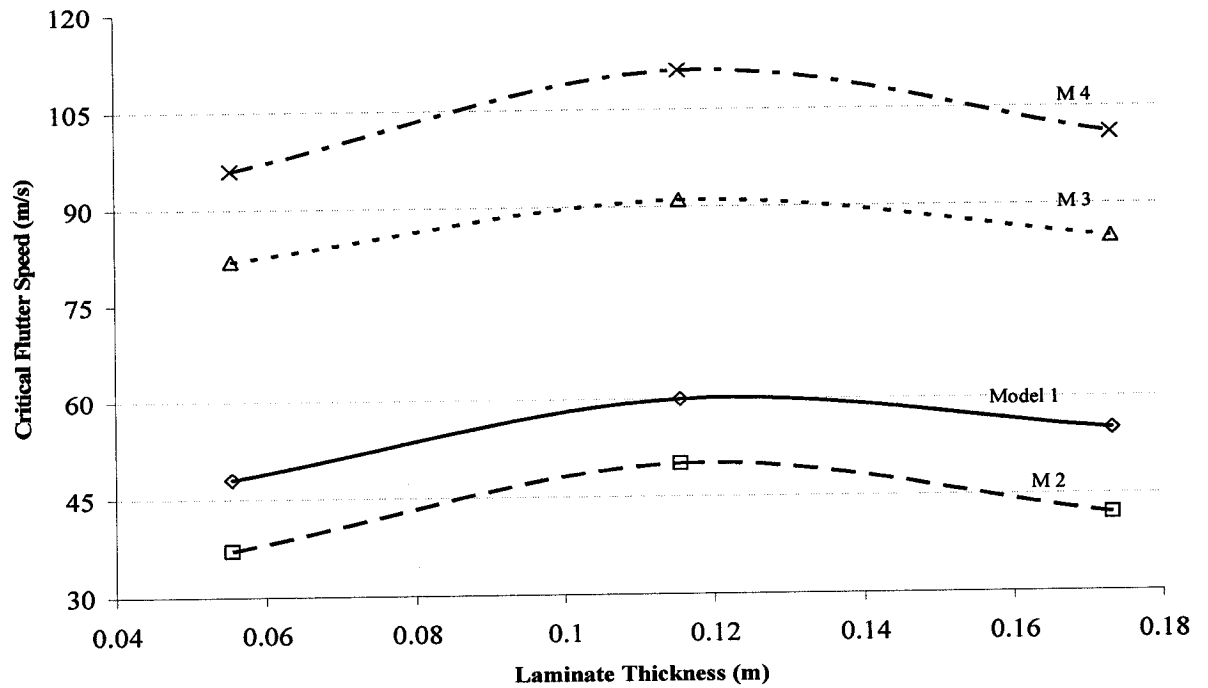


Figure 6.5.2.2 Laminate Thickness Effect on Critical Flutter Speed, for different Models, LDFA-a, CCR 0.125m, Second Chang River Bridge

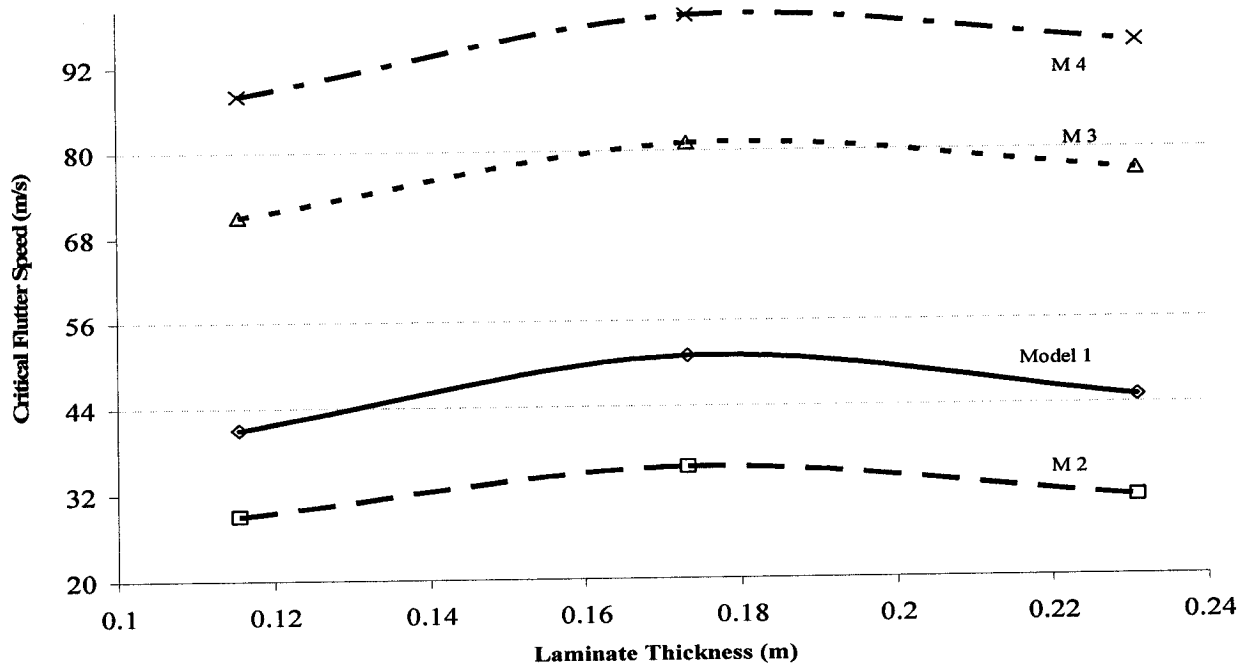


Figure 6.5.2.3 Laminate Thickness Effect on Critical Flutter Speed, for different Models, LDFA-a, CCR 125, Tataru Bridge

### 6.5.3 Effect of CFRP Cable Radius on the Critical Flutter Speed:

As the CFRP cable radius increases, the Critical Flutter Speeds of all models increases, however, Models I and II show higher sensitivity than other models. It is also important to note that the examined CFRP cable radii are basically increased as the bridge central span length increases.

The increase of the CFRP cable radius improves the restraining effect of the cable-stays system on the deck. Hence it improves the static behavior of the hybrid system as well as the natural frequencies. The increase of the deck restraint could change the mode shape of the different frequencies. It also diverges the different higher natural frequencies and contributes in releasing their possible coupling for some models. Ignoring the effect of the cable radius increase on increasing the aerodynamic loads on the cables themselves, the overall behavior of the bridge is improved since it is “better” supported. Different models show different rates of increase patterns for different bridge central span lengths (Figures 6.5.3.1 to 6.5.3.3), depending on their different stiffnesses, masses, and different natural frequencies.

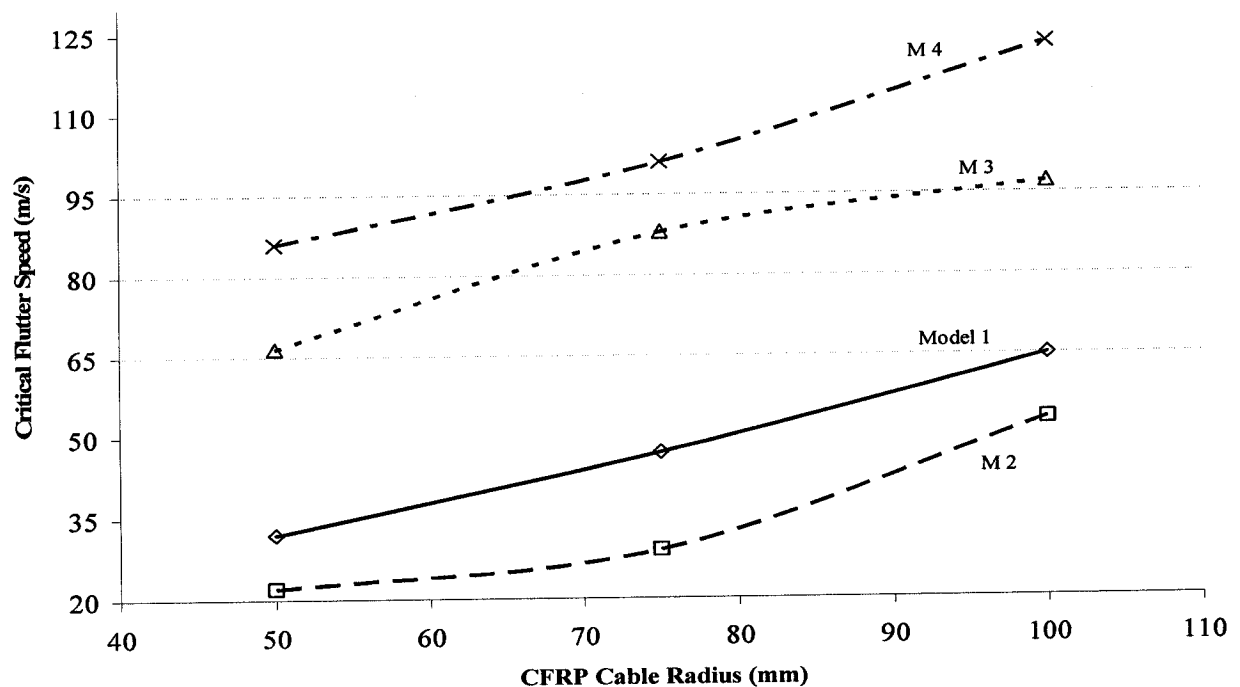


Figure 6.5.3.1 CFRP Cable Radius Effect on Critical Flutter Speed, for different models-  
- LDFA-a, Thb, Annacis Island Bridge

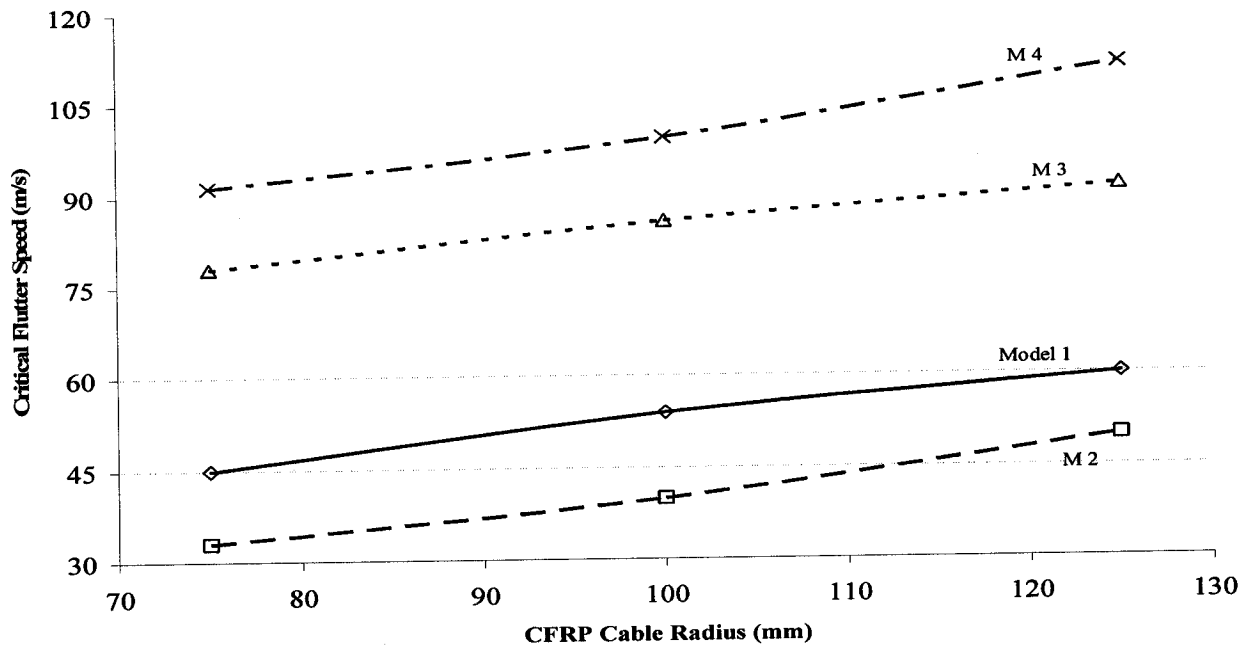


Figure 6.5.3.2 CFRP Cable Radius Effect on Critical Flutter Speed, for different models, LDFA-a, Thb, Second Chang River Bridge

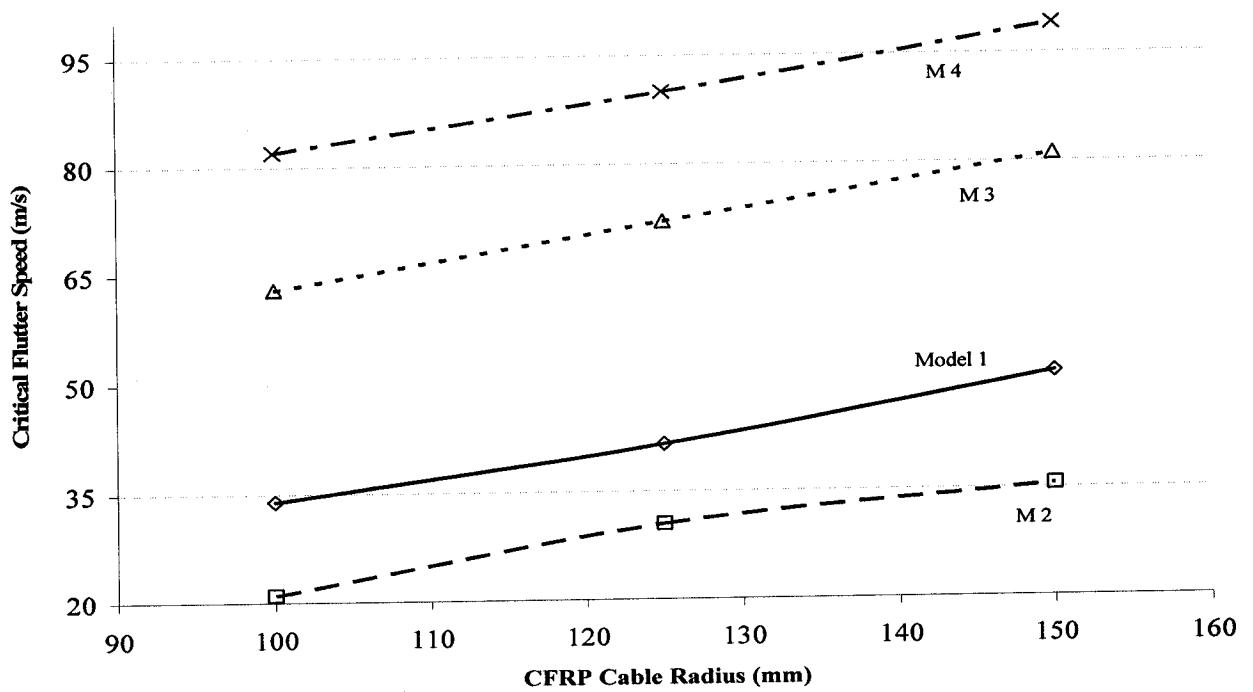


Figure 6.5.3.3 CFRP Cable Radius Effect on Critical Flutter Speed, for different models, LDFA-a, Thb, Tatara Bridge



## **Chapter 7**

### **Conclusions and Future Work**

# Chapter 7

## Conclusions and Future Work

### 7.1 Conclusions:

#### 7.1.1 The Hybrid Long Span Cable Stayed Bridge System:

- The Hybrid Long Span Cable Stayed Bridge is a competent system to safely bridge very long spans. With low LDFA and with proper laminate thickness and cable radius, the bridge system gives quite acceptable maximum deflections, ranging between  $1/600$  to  $1/400$  of the bridge central span length. The lateral and the longitudinal displacements are very small in general. For most of the cases, the Tsai-Hill Failure Function values are much lower than the criterion limit. The stresses in the stay cables and the chopped FRP parts are also far lower than their strengths. On the other hand, the natural frequencies for most of the cases are within the known ranges of this type of structure for the central span lengths considered.
- Considering the static and the aerodynamic results, it is apparent that Models III and IV are efficient and stable for extremely long spans (central spans of 900 m).

On the other hand, Model I can bridge very long spans (central spans of 600 m), while Model II is only able to bridge shorter spans of less than 400 m. However, by improving the dynamic damping characteristics of the four models and accurate evaluation of their structural and aerodynamic damping ratios, it is possible to extend the central span length limit of all the four models.

- The high stiffness and strength to weight ratios of the CFRP cables improve the bridge static and dynamic response. In addition, the use of longer cables results in only a slight reduction of their apparent stiffness. On the other hand, the high stiffness to weight ratio of the deck decreases the deck weight and the tensile stresses in the cables. Consequently, the hybrid bridge system can be constructed for even longer spans, since the critical stresses of the deck in the pylon zones are low in general and very long CFRP cables under high tension can be efficiently used.

#### **7.1.2 Analysis and Design Procedure:**

- To overcome the complications resulting from the need to integrate the advanced composite material micro/macro design, and the resulting mechanical properties, to the structural modeling, an analysis and design procedure is used. The procedure is iterative and employs a Multi-Scale Modeling Technique. It assumes that the FRP micro and macro parameters and architecture, as well as the structural parameters of the hybrid bridge system, are variables. The final values of all the parameters, micro, macro and structure, are decided during the procedure. Satisfying serviceability and ultimate limit states and the aerodynamic stability bound the structural behavior of the bridge, in the analysis and design procedure. It has been proved throughout this research that the procedure represents an appropriate, interactive and systematic process for analyzing and designing a new hybrid bridge system.
- The Stochastic Finite Element procedure appears to be a capable tool to evaluate the mechanical properties of the composite material with random oriented fibers. The numerical effort in the proposed method and the resulting accuracy are acceptable, with the use of a reasonably fine meso-mesh. The method provides an

integrated tool to the analysis and design procedure of the hybrid bridge system. This would enable an interactive process to decide the proper values of the material parameters for the best structural performance. The results obtained from the proposed method show a good match with the available industrial-experimental results.

### **7.1.3 The Advanced Composite Deck Models:**

- By comparing the analysis results of Models I and II, it can be concluded that reducing the height of the deck at the section sides would increase the Tsai-Hill Failure Function (THFF) in the vertical laminates, increasing the deflection at the central span and decreasing the critical flutter Speed.
- By a comparison of Models I and III analysis results, it is apparent that using the tubular inner cells in Model III leads to an increase in the level of the longitudinal displacement (W) of the deck, however it greatly improves the natural frequencies of the bridge system. In addition, the use of rounded edges to connect the two flat surfaces in the bottom outer laminate increases the THFF locally and the edge becomes sensitive to the change of different parameters.
- Most of the deck materials of Model IV are located in the two “boxes”, far from deck center of gravity. This gives high torsional stiffness, while the deck mass is relatively low, which greatly increases the torsional natural frequencies. Hence, the model critical flutter Speed is improved, but the lateral (or horizontal) and vertical natural frequencies are decreased.

### **7.1.4 Micro/Macro Design and Evaluation of Advanced Composite Materials:**

- The increase of the volumetric fiber fraction leads to a reduction in all the displacements and increases the natural frequencies. While these changes are small for the shortest bridge central span (or the Anacis Island Bridge central span), they are at their maximum level with the longest bridge span (or the Tatara Bridge central span).

- The laminate architecture, according to the proposed lay-up rule, which is based on the “Laminas Dominant Fiber Alignment (LDFA)” principle, gives better results than any general simple orientation arrangements of single, double or triple laminas. Under extreme static dead and traffic load conditions, it gives lower deformations, higher vertical natural frequencies and a lower level of Tsai-Hill Failure Function.

#### **7.1.5 Macro/Structure Parameters of Advanced Composite:**

- The increase of the Laminas Dominant Fiber Alignment (LDFA) leads to slight changes of all the deck displacements.
- The increase of LDFA results in a slight reduction of the Tsai-Hill Failure Function (THFF) over the top and the bottom laminates. On the other hand, the short vertical laminates of Model II and the rounded edges of the bottom laminate of Model III become sensitive and develop high levels of THFF.
- Following the slight change of the deformations and strains of the deck, the maximum and minimum principal stresses of the chopped FRP parts of different models are almost constant, due to the increase of LDFA angle. On the other hand, the maximum cable tension is slightly decreased by the LDFA increase.
- The deck stiffnesses are changed due to the change in all the laminates stiffness contributions, as the LDFA increase changes the moduli of every lamina in the laminate. This results in some changes of the natural frequencies of the bridge system. However, the influence on different natural frequencies of the bridge is limited and affects the high natural frequencies in general.
- All the displacements of the hybrid long span cable-stayed bridge deck are reduced with the increase of the Laminate Thickness (LTh). However, the rates of reduction of the displacements are different from model to model.
- The reduction of the stresses due to the increase in the sectional area results in a general reduction of Tsai Hill Failure Function (THFF) values of all laminates as the Laminate Thickness (LTh) increases for most of the examined cases. Due to the increase of the deck self-weight, that increases the deck stresses in the pylon

zones, the rate of change decreases as the laminate thickness increases. The short vertical laminates of Model II and the rounded connections of the bottom laminate of Model III appear to be sensitive to the laminate thickness increase and develop high levels of THFF.

- The laminate thickness increase results in slight reductions of the extreme principal stresses of the chopped FRP material parts of the deck. Similarly, the maximum cable tension stress is reduced as the laminate thickness increases.
- The laminate thickness increase results in an increase of the deck flexural and torsional stiffnesses, and an increase of the mass at relatively higher rates with the medium and the longest central spans. Consequently, the increase of the laminate thickness results in a general reduction of the vertical natural frequencies VNF, and a general increase in the Torsional Natural Frequencies TNF. Some fluctuations are observed with the high natural frequencies. However, the Horizontal Natural Frequencies HNF are constant or only affected very slightly.

#### **7.1.6 CFRP Cables:**

- It has been found that almost zero deflection along the bridge longitudinal centerline can be achieved using the proposed Cable Pre-Tension Formula, where the hybrid bridge is under self and the superimposed dead loads.
- Increasing the CFRP cable radius results in huge and dominant reductions of the maximum deflection of the deck for all models and all central span lengths. It also results in a reduction of the lateral displacement for most of the cases. However the maximum longitudinal displacements either remain constant or slightly increase.
- As the CFRP cable radius increases, a large decrease is evident in the maximum Tsai-Hill Failure Function in all laminates of all models and for all bridge spans.
- A general reduction of the principal stresses in the chopped FRP parts of the deck, with the increase of CFRP cable radius, is also evident. On the other hand, the stresses in the stay cables themselves are reduced as CCR increases. However, the resultant tensile forces in the stay cables are increased.

- The increase of the stay cables radius results in a general increase of the vertical and the torsional natural frequencies and slight changes of the horizontal natural frequencies, for most of the cases. However, the large effect of the bending-extension coupling for the cylindrical laminates works in opposition to the bridge system stiffness increase, and this causes a few irregular cases.

#### **7.1.7 Pylon Stiffness:**

- The increase of the pylon stiffness results in some reduction of the deck displacements in general, however the reduction is mostly small.
- In general, no effect, or a very slight decrease of THFF is dominant when the pylon stiffness is increased.
- As the pylon stiffness increases: (i) the vertical natural frequencies increase in general. (ii) All the Torsional Natural Frequencies (TNF) increase and sometimes the increase is very high. (iii) The Horizontal Natural Frequencies are constant or show a slight increase as the pylon stiffness increases. The high increase of the torsional natural frequencies and the smoother increase of the vertical natural frequencies result in an increase of the TNF to VNF ratios, in many cases, which improves the aerodynamic response.

#### **7.1.8 Aerodynamic Stability:**

- It has been found that the critical flutter Speeds of the proposed hybrid bridge are lower than those of the reference wind tunnel section model test of the Second Chang River Bridge, for all the proposed models. Model IV, which gives the best results, has a maximum flutter Speed of approximately 111 m/s, that is only 63.4% of the critical flutter Speed of the reference bridge (175 m/s).
- The best results are mostly related to a low level of lamina dominant fiber alignment (LDFA-a) and the medium laminate thickness (or Laminate Thickness-b). Model II gives the lowest and Model I gives the second lowest critical flutter Speeds, while Model III gives the second highest critical flutter speeds.

- It has been found that the increase of the Laminas Dominant Fiber Alignment result in slight changes of the Critical Flutter Speed of different models.
- As the Laminate Thickness increases, it has been found that the Critical Flutter Speeds (CFS) show almost the same pattern; increase in the CFS as LTh increases from the lowest examined thickness to the medium, then decreases with further increase in the laminate thickness. However, the CFS related to the maximum examined laminate thickness is usually higher than that related to the minimum laminate thickness.
- The Critical Flutter Speeds of all models increase with the increase of the CFRP cable radius.

## **7.2 Future Work:**

### **7.2.1 Theoretical Research:**

- The effects of further increase of the stiffness and strength to weight ratios for the deck on the static and dynamic performance of the system should be investigated. Consequently, other combinations of advanced composite material for the micro-design of the lamina, such as the Carbon fibers, with either Glass Epoxy or Kevlar<sup>®</sup> Epoxy-Matrix, should be investigated for the use in very long span Hybrid Cable Stayed Bridges.
- The chopped FRP parts of all four models are under low stresses in general. Hence, the possibility of replacing them by lightweight concrete as an economical alternative can be investigated.
- Further possible modifications of the Model IV deck section can be studied such as: (i) improve the stiffness of the intermediate “slab-and-grid” part of the section by adding aerodynamically shaped deep girders in the lateral direction, (ii) Reform the side boxes in order to minimize the vortex shedding, and improve the aerodynamic properties of the section, and (iii) investigate the effect of using higher Carbon and Steel Fibers fractions in the Chopped FRP, which is the dominant material of this model.



- Regarding the aerodynamic stability: (i) investigate the frequency domain flutter analysis as an economical alternative to the time domain analyses used in this research. The effect of neglecting the bridge system geometric nonlinearity in this type of analysis, on the accuracy of the results should be evaluated, (ii) investigate the effect of the structural and the aerodynamic damping on the aerodynamic stability, and (iii) investigate the buffeting and vortex shedding response of different models.
- Design and model advanced composite construction joints for the different deck models and evaluate the effect on the bridge performance and construction.

### **7.2.2 Experimental Investigations:**

- Experimental verifications of all the fiber and matrix combinations of both Chopped FRP and the advanced composite laminas and laminates need to be performed.
- Building simple-span scaled deck sections and testing them under static loads and dynamic reversal loads.
- Building scaled Hybrid cable-stayed bridges for all deck section models, then testing and monitoring their performance under static and dynamic loading conditions. The results would be very useful in simplifying the theoretical modeling procedures proposed in this research.
- Aerodynamic wind tunnel tests will be very important to: (i) evaluate the effect of different section shapes of different models on the flutter derivatives, (ii) evaluate the flutter speeds for different models and different angles of attack, and (iii) the effect of the variation of the deck stiffness and mass, due to the change of the micro/macro and structure parameters, on the flutter speed can be investigated experimentally.
- Experimentally investigate the possibility of involving lightweight concrete as an alternative to the Chopped FRP in a series of a simple span experiments. The concentration should be on the compatibility of deformations and the composite

action between the concrete and the advanced composite laminates/ laminated box girder.

- Experimentally evaluate the efficiency of the construction joints and their ability to transfer different stresses.

### **7.2.3 Practical Applications:**

- In the short term, the use of CFRP cable stays and partial-advanced composite decks (combination of traditional materials and advanced composite materials) section should be proposed.
- The design and construction of short or medium span advanced-composite bridge should be proposed as a successful option from the life cycle point of view, and as a capable structure from the theoretical modeling point of view.

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