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3D FE MODELLING OF CABLE-STAYED BRIDGE ACCORDING TO ICE CODE

M.Sc. THESIS

IN

CIVIL ENGINEERING

BY

MOHAMMED KHALEEL IBRAHIM

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Supervisor

Assoc. Prof. Dr. Mehmet Tolga GÖĞÜŞ

Co-Supervisor

Dr. Muthanna Adil ABBU

by

Mohammed Khaleel IBRAHIM

September 2021

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Mohammed Khaleel IBRAHIM

ABSTRACT

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IBRAHIM, Mohammed Khaleel
M.Sc. in Civil Engineering
Supervisor: Assoc. Prof. Dr. Mehmet Tolga GÖĞÜŞ
Co-Supervisor: Dr. Muthanna Adil ABBU
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This thesis presents the performance of cable-stayed bridge by representing two models using the ANSYS program under the influence of a concentrated load and then comparing them with the experimental results. two model used in the analytical, first one was (harp type1) used length of girder (4500mm), high of pylon (1480mm) and depth of flange for girder (40mm), while the second one was (harp type2) like the dimensions of the first model, but the difference is in the girder’s flange depth, which reaches in this model is (25mm). When cable-stayed bridges are built using the balanced cantilever method, the girder's stability problem is more acute during construction than in the initial state. In this study experimental and analytical experiments were done for two models: Harp type 1 and Harp type 2, to evaluate the ultimate behavior of cable-stayed bridges. Several plastic hinges formed at the limit state, and the girders buckled throughout the whole span. For each experimental model, a numerical analysis was performed, with the findings showing good agreement with the experimental data. The impact of such factors on cable-stayed bridge analysis and structural behavior has been thoroughly investigated in this study. The dimensions of the model were also compared with ICE Code in relation to the tower's height to the main span, and back span to main span ratio. It was within to standard specifications.

Key Words: Cable-stayed Bridge, Numerical Analysis, Ultimate Analysis, ICE Code.

ÖZET

KABLOLU KÖPRÜSÜN BUZ KODUNA GÖRE 3D SONLU ELEMAN MODELLEMESİ

IBRAHEEM, Mohammed Khaleel
Yüksek Lisans Tezi, İnşaat Mühendisligi
Danışman: Doç. Dr. Mehmet Tolga GÖĞÜŞ
İkinci Danışman: Dr. Muthanna Adil ABBU
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Bu tez, konsantre bir yükün etkisi altında ANSYS programını kullanarak iki modeli temsil ederek ve daha sonra bunları deneysel sonuçlarla karşılaştırarak kablolu köprünün performansını sunmaktadır. analitikte kullanılan iki model, birincisi kiriş uzunluğu (4500mm), direk yüksekliği (1480mm) ve kiriş için flanş derinliği (40mm) kullanılan (arp tip1), ikincisi ise (arp tipi2) boyutları gibi idi. İlk modelin farkı, ancak bu modelde ulaşılan kirişin flanş derinliğindeki fark (25mm). Dengeli konsol yöntemi kullanılarak kablolu köprüler inşa edildiğinde, kirişin stabilite sorunu inşaat sırasında ilk duruma göre daha şiddetlidir. Bu çalışmada, kablolu köprülerin nihai davranışını değerlendirmek için Harp tip 1 ve Harp tip 2 olmak üzere iki model için deneysel ve analitik deneyler yapılmıştır. Sınır durumunda birkaç plastik menteşe oluştu ve kirişler tüm açıklık boyunca büküldü. Her deneysel model için, deneysel verilerle iyi bir uyum gösteren bulgularla sayısal bir analiz yapıldı. Bu gibi faktörlerin kablolu köprü analizi ve yapısal davranış üzerindeki etkisi bu çalışmada kapsamlı bir şekilde incelenmiştir. Modelin boyutları ayrıca kulenin ana açıklığa olan yüksekliği ve arka açıklığın ana açıklığa oranı ile ilgili olarak ICE Kodu ile karşılaştırılmıştır.

Anahtar Kelimeler: Askılı Köprü, Sayısal Analiz, Nihai Analiz, ICE Kodu.

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To God be the Glory. First and foremost, I'd like to express my gratitude and appreciation to the Almighty God, who is the creator, sovereign, and sustainer of the universe and its beings. This endeavor could only be performed with his kindness and assistance, and I am hopeful that he will accept my small effort. I'd like to express my heartfelt appreciation to my supervisor, Asst. Prof. Dr. Mehmet Tolga GÖĞÜŞ, for recommending the research idea and for his constant advice and support during my work; without him, this study would not have been feasible to complete.

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A special thank is reserved for my parents and all my family members, they have given me an endless enthusiasm and encouragement.

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I would like to express my sincere gratitude to anyone who helped me throughout the preparation of the thesis.

At last but not least, the vast that will not happen twice, to her.

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LIST OF SYMBOLS

A	Sectional area of steel (mm2).
$\boldsymbol{I_x}$	The steel section's moment of inertia around the x-axis (mm4)
$\boldsymbol{I_y}$	The steel section's moment of inertia around the y-axis (mm4)
$\boldsymbol{P_y}$	Load about y direction (N).
$\boldsymbol{MP_X}$	Moment concerning x-axis (N. mm)
$\boldsymbol{MP_y}$	Moment concerning y-axis (N. mm)
α	Stay angle
H	Pylon height
L	Main span length
H\L	The ratio of pylon height to main span

CHAPTER I

INTRODUCTION

1.1 General

The cable-stayed bridge is considered one of the modern era's achievements, as its construction is considered one of the most advanced engineering and building successes, and most often the cable-stayed bridges span large waterways or between valleys to reach between two difficult-to-reach points or over residential areas where these bridges are the most impressive types of bridges. With its vast main space, and its wonderful appearance, as it is distinguished by a road carried on steel cables and both sides of the bridge are connected by two towers. Cable-stayed bridge are used to extend their spaces across vast areas, the space of one of them in most cases may exceed 300 m, and some of them have main spaces extending to more than 1,200 m. These bridges are also constructed to extend over deep water bodies or steep valleys, and are used in other locations where abutment construction is difficult and expensive. It does not need more than two pillars, and sometimes one pillar supports each of them on a tower. The main space of the cable-stayed bridge extends between these two towers. Each of the side spaces extends between a tower and anchorage. Most of these anchors are constructed from huge concrete blocks on either side of the bridge. The supporting cables for the towers are called main cables, and each suspension bridge has at least two sets of these cables, each extending between the ends of the bridge, where it is secured to be attached to an anchor. The main cables are also linked to the upper ends in the vertical suspended cables from which each cable is connected, at its lower end to the road spanning the bridge. Strong winds may cause vibrations of the suspended bridges, which leads to their closure in extreme emergency cases. Most of them are provided with thick fittings that support their perforations to reduce vibration, one of which is known as the stabilizer bar.

1.2 Types of Cable-Stayed Bridge

There are several classifications of cable-stayed bridge. According to the number of spaces, there are cable-stayed bridges with one space, two spaces, three spaces, and multi-spaces in case the spaces exceed the three. There is another classification for suspension bridges according to the type of suspension of the bridge, and this classification includes only two types, namely, the full suspension bridge and the cable-stayed bridge or the rest.

In this research, the Cable-Stayed Bridge will be used, which in turn is classified into several types according to the shape of the cables used in the suspension, namely:

1. Fixed Cable Fan Cable Stayed Bridge
2. Fixed Cable Harp Cable-Stayed Bridge
3. Radial Cable-Stayed Bridge

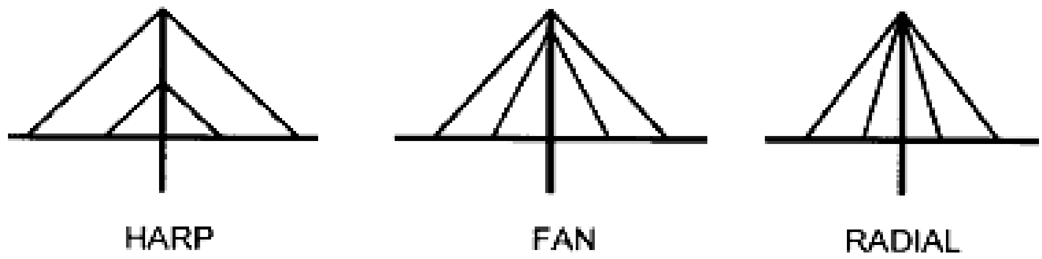

Figure 1.1 Type of cable stayed-bridges according to the shape of the cables

1.3 Parts of the Cable-Stayed Bridge

The components of a cable-stayed bridge are as follows:

1.3.1 Deck

It is the longitudinal structures that support and distribute the moving vehicles loads and ropes and ensure the stability of the structure. It has multiple sections and can be made of concrete or steel. The deck of a cable-stayed bridge is an essential element of the structure, resisting both bending and an axial force derived from the horizontal component of the stay force, rather than just being supported by the cable as in a suspension bridge. A minimum-weight deck section is the most cost-effective choice for the suspension bridge. The larger involvement of the deck in the overall structural behavior of the cable-stayed bridge allows for the consideration of alternate deck forms, especially since concrete is an effective material for compressive members. As a result, cable-stayed bridge deck forms have been constructed as follows: Link stayed connect deck structures have been developed as follows:

1. an orthotropic road deck incorporated into a steel section
2. a section of concrete
3. a concrete and steel composite section

- Section of steel deck

Early cable-stayed bridge designs had a limited number of stays, necessitating the use of a light yet stiff deck capable of spanning the broad stay anchor spacing. This structural configuration, which included an orthotropic road deck, was appropriate for the all-steel construction. For moderate spans, the steel deck solution has become less cost effective since the introduction of multi-stay cable systems. Steel construction has been retained for the longest spans where the reduction in deck weight is an economic imperative, such as the Normandy Bridge, Stonecutters Bridge, and Tatara Bridge.

- Section of composite deck

When compared to a comparable concrete slab road deck, the orthotropic deck bears a cost penalty because to its high manufacturing costs. The greater area of cable stays and the increased pylon and foundation expenses necessary to sustain the heavier dead loads balance the economic benefit of the composite concrete slab over the steel deck to a certain extent.

- Section of concrete deck

Concrete deck sections have gone through similar changes as steel deck sections. Torsion box deck sections are frequently combined with a single central plane of stays. The Brotonne Bridge, which spans the Seine near Rouen, France, was the first of these designs(Macdonald 2008).

1.3.2 Cables

It is a group of parallel cables bundled wires that support the bridge body by ropes and transfer the loads to the towers.

Basically, high-strength steel wires are arranged into cables. The wires are assembled to form a strand and later used to shape a cable on site or in the shop.

The seven-wire strand is the simplest form to be found in a cable supported bridge. The wires have, in general, 5mm diameter and tensile strength between 1770 and 1860 MPa. They are normally arranged with a single straight core confined by one level of six wires placed in a helicoidal course (Campus 2009).

The multi-wire helical strand, also called spiral strand, is conceived in a similar way to the seven-wire strand. However, this type has multiple wire layers with opposite course directions. As a side effect, when the wires are twisted the total strength drops when compared to the straight element. Approximately. A helicoidal strand's final strength is 10% lower than the total of all parallel wires. The helicoidal shape of the top layers has a 15-25 percent lower nominal modulus of elasticity than straight wires. This phenomenon also occurs on the seven-wire strand but with less magnitude as an alternative to the normal strands there is the locked-coil system. An array of round wires is surrounded by a particular Z-shape wires on the outside of this sort of cable. The Z-shape wires ensure a tight surface that prevents corrosion in the interior. The final protection against corrosion used to be a surface treatment with painting, nowadays all the wires are galvanized instead. Strands at this system are manufactured in full length and with a diameter range from 40mm to 180mm large strands are typically utilized in multi-cable cable-stayed bridges where each stay is a mono-strand cable(Campus 2009).

However, difficulties in the design of anchorages and protection against corrosion restricted the usage of this types of systems. Today, the main option adopted in several countries is the parallel-strand cable. In this system seven-wire strands with a protection layer of high-density polyethylene (HDPE) are arranged together in parallel to form the cable. In many cases, a cylindrical pipe surrounds the stay to reduce the drag coefficient caused by the grooved surface. Cables of this type can have between 18 and 90 strands with breaking load up to 24 MN (Campus 2009).

1.3.3 Pylon

They are the vertical intermediate structures that support the main cables with the bridge body. The pylon is the major element that conveys the visual shape of every cable-stayed bridge, allowing for a unique style to be included into the design. The pylon's design must also handle the different stay cable configurations, as well as the geography and geology of the bridge site, and transport the stresses efficiently. The pylon's principal role is to transfer the forces generated by anchoring the stays, and these forces will dictate the pylon's design. Wherever feasible, the pylon should bear these forces through axial compression, minimizing any eccentricity of loading.

- Pylons of steel

Early cable-stay pylons were mostly made of steel boxes, and bridges like the Stromsmund Bridge used a steel portal frame to give transverse constraint to the stay system. However, because adequate transverse restraint may be given within the stay system itself, this restraint is generally unneeded.

- Pylons of concrete

When sustaining loads under axial compression, concrete is extremely efficient. Despite its significantly higher self-weight than a steel equivalent, advances in concrete construction and contemporary formwork technology have made the use of concrete for pylon building increasingly competitive. Concrete has shown to be particularly adaptive to increasingly complicated pylon designs. To accommodate both vertical and inclined stay configurations, several different types of pylons have been created (Macdonald 2008).

1.3.4 Anchors

They are massive concrete blocks that hold the main cables and act as an end to support the bridge and the towers. Anchorage systems are crucial for the structure, this elements are responsible to transfer the actions that came from the deck and transfer them to the cables. Special requirements are needed to design this system, especially them to have appropriated mechanical resistance and fatigue resistance.

A bridge anchor is composed by two main parts: the anchorage head, that is responsible for the load transmission; and the transition zone, where the cable arrangement changes to fit the anchorage system.

A unique socket was created in the 1970s to anchor parallel-wire strands, in which the wires are displaced through holes in a locking plate at the far end of the socket and furnished with button heads to improve resistance to individual wires sliding. To decrease fatigue effects, the concavity inside the socket is generally filled with cold cast materials such as epoxy resin, zinc dust, or tiny, hardened steel balls.

The principal functions that an anchorage must accomplish are listed below

- Properly transfer the loads

- Filtering out angular deviation
- Possibility of adjustment
- Corrosion protection
- Removability

Besides, in order to prevent cable vibration internal dampers can be added close by the anchorage element. A variety of anchor types are available, depending on the solution adopted for the structure. In past solutions, using eye-bars and strand shoes were very common, according to the trends of bridge constructions of the time. Recently, the use is focused on standard solutions based on wire cables that allow the maintenance and implementation of devices such as dehumidifiers.

1.4 The Working Principle of the Cable-Stayed Bridge

Figure 1.2, explanation of the mechanism of transmission of loads for cable-stayed bridges, in which the load is transmitted from the girder to the tower, which then transmits the weights to the ground under compression. This mechanism is considered the basic idea in the work of cable-stayed bridges, which gives a comprehensive visualization method for the way this type of bridge works.

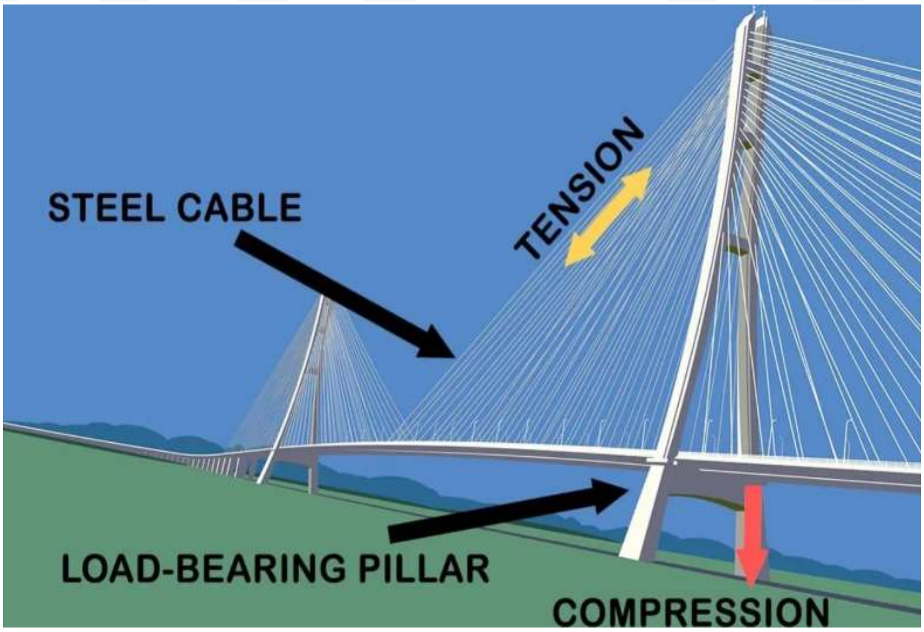

Figure 1.2 Explanation of the mechanism of transmission of loads

The concept of suspended bridge work is based on a simple principle, as the bridge carries vertical loads that interrupt the bridge body. The suspension bridge completely differs from the fixed cable bridge in that the first is a full suspension in which the bridge body is suspended by the girder by vertical cables that divide the space of the long bridge body into short spaces and these secondary cables in turn are attached to a main cable that is installed on the bridge towers. The bridge body's weight is transferred to the towers, which then transfer it to the earth through the cables.

Whereas, the fixed cable bridge depends mainly on the towers, and the cables carry the bridge body directly to the towers by connecting them to the towers without a main cable connecting each of the two towers. The cable distribution divides the wide space of the bridge body into small spaces, which allows increasing the length of the main space of the bridge body without any embarrassment. Triangular shapes are formed by connecting the cable to the tower with the bridge body (road) structural as in the Figure 1.3.

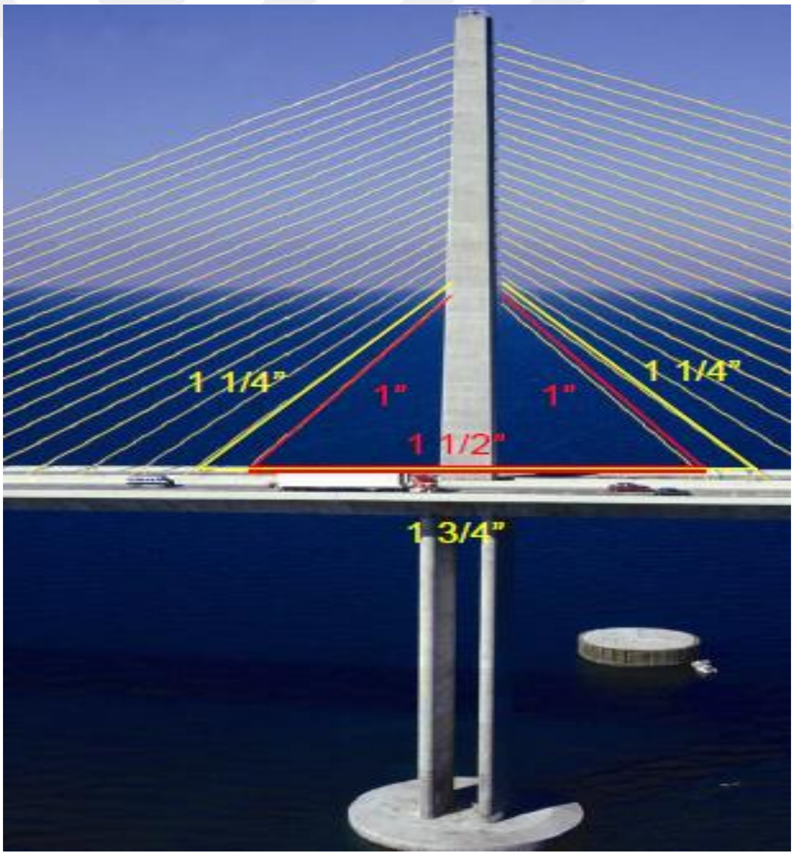

Figure 1.3 Dividing the bridge into smaller spaces in the form of triangles

We note in the red triangle that the lengths of two sides are 1, for example, so the base is 1.5, as a result, the total of the two sides' lengths is more than the base's length, and the same is the case for the yellow triangle, and thus triangles are formed close to the equilateral triangle, which represents the strongest triangle.

The loads are distributed according to this arrangement, so that the cables bear the tensile forces in the bridge, while the pylon towers and the body of the bridge girder bear the pressure forces, and thus all parts of the bridge bear axial loads as in Figure 1.4, which gives efficiency in structural performance greater than the parts that bear flexural loads, which makes this type of bridges are more economical.

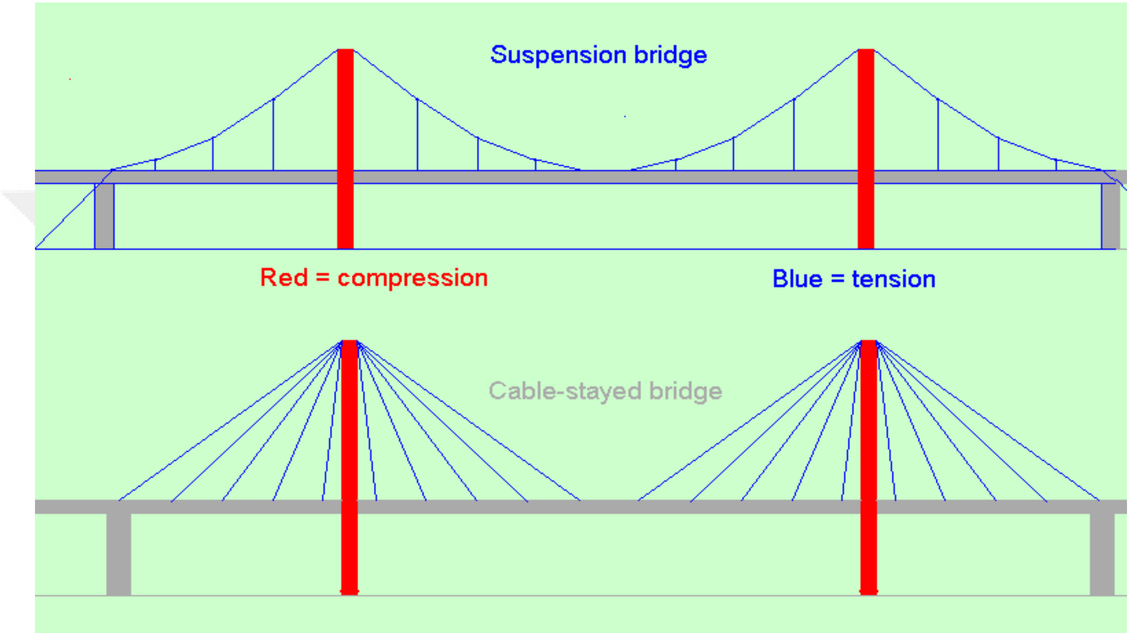

Figure 1.4 Distribution of axial loads on the bridge sections

1.5 Analysis Using (ANSYS) Program

The ANSYS program is considered one of the strongest and most famous programs in the world in the field of advanced studies and research. Marketing of the program began in 1970 and is now used all over the world in civil engineering sciences, space sciences, autonomous devices, industry, nuclear research, energy services and engineering. Electricity and electron. In addition, the software is used by hundreds of consulting firms and advanced institutions across the world for education, research, and phenomenon interpretation. The program is based on the principle of the finite elements. The process of dividing the elements is a mandatory process in the ANSYS program. The program has great capabilities in this field. The user can choose the size of the resulting elements, and it is possible to choose the shape of the division (triangular or quadruple in the binary, hierarchical or prismatic case. In static case), as well as choosing the type of element (linear or square), as well as free or withdrawal division

1.6 Finite Element Method (FEM)

The numerical solution of field issues is done using the finite element method (FE). The geographical distribution of one or more dependent variables must be determined for a field problem. Differential equations or an integral expression are used to describe field problems mathematically. Finite elements can be expressed using either description. Individual finite elements can be thought of as tiny components of a larger structure. The assemblage of components is termed a finite element structure because the elements are linked at nodes. The real variation in the region encompassed by one element is almost probably more complex, thus a finite element analysis can only offer a rough estimate. We are increasingly finding that approximate numerical solutions to issues, rather than accurate closed-form answers, are required in engineering scenarios. In practice, there are a variety of approaches for analysis with varying degrees of rigor (Abbu, Ekmekyapar, and Özakça 2013).

These vary from simple beam theory to sophisticated shell finite element studies; additional analytical methods include folding plate and finite strip approaches. While some of the latter approaches are generic and accurate, they may not be appropriate for use in the early stages of design.

This is especially true with the FEM, which necessitates a significant amount of work and produces an excessive quantity of data, which may be time consuming to understand. The use of more complex techniques also obscures the importance of specific structural activities in determining the overall resistance of a cable-stayed bridge. For example, it’s difficult to separate the effects of torsional and distortional warping on normal stresses from those of extension and bending in a finite element analysis. This is also true for finite strip and folding plate studies.

1.6.1 Finite Element Modelling

The finite element technique is the most powerful, adaptable, and appropriate numerical instrument for solving a complicated continuum issue out of all the current analysis methods. The technique has evolved into a critical and frequently required component of engineering analysis and design.

Finite element computer programs may now be used virtually in all fields of engineering thanks to recent advancements in computer technology. The finite element method can easily simulate a complicated geometry like that of a cable-stayed bridge. The technique may also cope with a variety of material characteristics, structural component connections, boundary conditions, and statically or dynamically applied loads.

This approach can predict the linear and nonlinear structural responses of such bridges with high accuracy. For predicting and modeling the physical behavior of complex engineering systems, the (FEM) has become a standard. Engineers in business, as well as academics at universities and government laboratories, have embraced commercial Finite Element Analysis (FEA) packages. As a result, graduate or undergraduate senior-level courses in engineering departments teach not only the theory of (FEM) but also its applications utilizing commercially accessible (FEA) tools.

1.7 Types of Structural Analysis Used for Cable-Stayed Bridge

1.7.1 Linear Structural Analysis

Depending on the nature of the bridge load, one of the kinds of analysis offered in ANSYS, either static (static) or kinematic, may be used to estimate deformation and load bearing capability. This kind is further subdivided into the following categories:

- Analysis of static data

Different forms of finite element within can be used to evaluate the behavior of structures under constant stress. The kind of components utilized in the study is dictated by ANSYS and the nature of the bridge construction. This type is considered one of the simplest types of analysis to deal with.

- Dynamic Analysis

This type is considered more complicated than the first type, due to the nature of the loads causing such type of analysis, which are important loads and should be considered, suspension bridges, in particular, are a good example due to traffic from transport vehicles and pedestrians, as well as the wind movement that greatly affects such type of bridges. As well as seismic loads, if any. This type, in turn, is divided into three more common types:

- Modal analysis

This sort of analysis is utilized in structural engineering to figure out the form of the natural phase and the frequency phases of the structure, i.e. the bridge, in this case, during the exposure of the bridge to various vibrations, especially those generated by the wind movement. It is possible to perform this analysis manually, but it is common to perform it using the finite element method and in our study this method will be dealt with through the ANSYS program.

- Harmonic Analysis

When a structure is subjected to a cyclic load and the response is predicted to be cyclic as well, this form of analysis is utilized. The combinational analysis tool in ANSYS allows the user to tackle these types of issues. It's also a crucial cable-stayed bridge study.

- Transient Analysis

When non-periodic loads have a temporary effect on the facility, this analysis is employed. The most important example of the effect of this type of load on the bridge is the movement of moving vehicles. ANSYS independently handles this type of analysis and provides realistic modeling of such cases.

1.7.2 Non-linear Structural Analysis

Because nonlinearity occurs in real life, nonlinear analysis of cable-stayed bridge systems is essential. Only a linear analysis can be used to estimate the performance of a bridge. The behavior of the cables, stiffening girder, and pylons all influence the nonlinear performance of a cable-stayed bridge. The cables' nonlinearity was caused by the loading they were subjected to, which resulted in cable elongation and axial tension. In the majority of situations, the equivalent modulus of elasticity is employed in each stage of the analytical computation.

Nonlinear behavior was also caused by the interplay of loads and axial forces in the pylons and girder. The extent of deflection induced by bending and the intensity of the compressive force relative to the bucking load determine the degree of nonlinearity. Nonlinear behavior is also caused by the deformation of the structure, which is constantly changing.

The second order theory is utilized to tackle nonlinearity problems by taking into account the entire structure's deflection.

1.8 Objectives and Aims of the Study

One of the most effective structural methods for large span bridges is the cable stayed bridge. Due to the horizontal components of cable force, the girders of cable-stayed bridges are subjected not only to the bending moment but also to the compression force. The compressive force has an influence on the girder's transverse stiffness, which is known as the P-effect. This effect can result in significant geometrical nonlinearity throughout the bridge system. The compressive axial pressures operating on the middle of the girder in the mid-span may be cancelled out and the P-effect ignored in the case of the cable stayed bridge with two pylons and three spans examined in this study. When the bridge is built utilizing the balanced cantilever approach, however, all of the horizontal components of the cable forces are applied to the girder. The compressive pressures reach their peak just before the critical segment is installed. As a result, the girder must be able to withstand not just bending moments caused by self-weight and the weight of the derrick crane, but also compression pressures caused by cable forces. The girder near the pylons develops the greatest compressive pressures and bending moments, and the plastic hinges may be created there. As a result, both geometrical and material nonlinearity should be considered when assessing the stability of cable-stayed bridges.

The goal of this thesis is to use FEM to assess and design the performance of cable-stayed bridges. The thesis's ultimate objective is to obtain a working understanding of the bridge system in question. When the following objectives were fulfilled, this was regarded a success:

- The imperfect interaction between steel pylon and girder in cable-stayed bridges has been the subject of few theoretical research. The goal of this theoretical research is to look at how cable-stayed bridges behave amongst components.
- Create a three-dimensional finite element model capable of forecasting cable-stayed bridge structural reaction.
- This research looks into the capabilities and limitations of analysis system for static issues (ANSYS), a commercial finite element analysis tool for modeling cable-stayed bridges.
- Determine the global reaction and behavior of cable-stayed bridge bridges with complete and partial load interactions systems.

- Determine the system's localized behavior of joints and connections.
- Examine the major geometric characteristics of the bridge superstructure that influence service load behavior.

1.9 Layout of Dissertation

To achieve the aforementioned objectives, the first step toward completing the study was to compile and study existing literature on related topics, as described in the Contents and Arrangement of this Study. This review of the literature can be found in Chapter 2 of the thesis. This chapter also includes a literature assessment of previous cable-stayed bridge analytical and experimental work. In Chapter 3, we will learn how to create three-dimensional finite element cable-stayed bridge models with complete interaction theory using the commercially available finite element computer application "ANSYS".

To verify the finite element model and provide information about the linear and response of cable-stayed bridges, Chapter 3 compares the finite element model predictions with the experimental findings of the laboratory tested bridge models for various load cases. Once the literature review was performed the possible variations of the model to be studied were pared down using a rating model.

Section details, analytic tool selection, parametric studies, and thorough analyses of joints and connections are all covered in Chapter 4. Lastly, Chapter 5 contains a summary and conclusions.

CHAPTER II

LITERATURE REVIEW

2.1 Introduction

This part offers a previous review of a cable stayed bridges, and how previous researchers have understood their behavior under loads that contain different units of static and kinetic loads. The development and characteristics of modern high performance cable-stayed bridge. How these cable-stayed bridge studied in composite material is also discussed.

One type of bridge is the cable-stayed bridge that are utilized frequently in all types of constructions due to its many advantages like high strength/hardness, attractive appearance, high energy absorption. The building of a suspension bridge is regarded one of the most sophisticated engineering and construction feats of the modern age, much as cable-stayed bridges are. In addition to suspension bridges, a cable-stayed bridge is another suspension structural system.

The stay cables, main girder, and towers are the primary components of a cable-stayed bridge. Stay cables and towers can be arranged in a variety of ways to satisfy transportation, financial, and other requirements. The lateral bending and torsional deformation of the bridge are mutually linked as a result of efficient stay cables. The dynamic characteristics of cable-stayed bridges are more difficult than those of conventional bridge designs due to the structure's thin and flexible qualities (Feng 2015).

2.2 Simulations Numerical (Finite Element Method)

Bridge structural design, particularly for long-span bridges, is difficult at every stage: design, building, and completion, since there are so many variables to consider, such as weather, time, cost, and the structure's characteristics (Feng 2015).

In addition to cable-stayed bridge analysis, we need many complex and long equations to be used in the analysis, where many numerical analysis methods have been developed for the analysis of cable-stayed bridges during the last few decades The planar grid technique, space frame method, finite strip method, finite difference method, finite element method, and closed form method are some of these approaches. It has been credited as a powerful tool for presenting solutions to complex structures.

H. Wang et al. (2014) throughout his study, he used ANSYS to create a 3D finite element (FE) simulation for the Sutong cable-stayed bridge (SCB). The dynamic characteristics of the bridge are investigated using a subspace iteration approach. On the basis of recorded wind data, the observed spectra expression is given using the non - linearity least-squares regression approach. To model turbulent winds at the bridge location, the spectrum representation technique and the FFT methodology are employed. The effects of several main structural factors and measures on the dynamic characteristics of the bridge are investigated. Among these are the dead load intensity, as well as the vertical, lateral, and torsional stiffness of the steel box girder. The vibration mode of the steel box girder is also investigated in relation to the elastic stiffness of the connecting device used between the towers and the girder. The vertical, lateral, and torsional buffeting displacement responses all gradually decrease as the dead load intensity increases. The dynamic characteristics and structural buffeting displacement reaction of the steel box girder are mostly influenced by its vertical and torsional stiffness. As the lateral stiffness increases, the lateral and torsional buffeting displacement responses decrease. These data can be used to assist the design and study of super-long-span cable-stayed bridges. A finite element model of the Sutong Bridge was created using ANSYS software. The FE model is created in compliance with the design drawings to a tee. A backbone model is used to represent the bridge in this spatial model, and two types of elements are used to replicate varied structural members. Temporal beam elements (BEAM 4), a uniaxial element with tension, compression, torsion, and bending characteristics, are used to model steel girders, cross beams, towers, and piers. At each node, each element has six degrees of freedom: transcriptions in the x, y, and z orientation, as well as rotations around the x, y, and z axes. The steel girders are meshing at the cable points. The stay cables are represented by LINK10. LINK10 is a bilinear stiffness matrix-based

3D spar element that creates a uniaxial tension-only element. The stiffness is removed if the element is compressed while using the tension-only option.

ERNST and H. (1965) provides a simple and effective method of simulating equal cable stiffness. Due to the fact that Ernst's technique improves cable modeling efficiency, it is limited to extracting the long-span bridge structure's response. It is impossible to ascertain the natural frequencies and vibration mode forms. However, when the deck girders are only concentrated in a long bridge construction, this is still a highly typical strategy.

Odden, Skyvulstad, and Sigbjörnsson (2012) on the finite element model, a method for simulating the static wind load was presented. In their study, the Sogne Suspension Bridge in Norway was chosen, and the model was created using ABAQUS software. To compute the static response of the bridge models under the mean wind load, the wind loads are exerted to the models in ABAQUS. The wind load is assumed to be applied to the whole bridge span. Drag and lift loads on the girder are imposed as concentrated loads on the girder's mass center every 20 meters. The bending moment is implemented as a focused moment at the center of the girder every 20 meters. The drag load is applied as concentrated forces where the hangers are attached to the cables. The static load given to the model is calculated using the mean wind load equation. And, as one might anticipate, the variance increases as the mean wind speed rises. The horizontal variance at mid-span is between 5 and 30 m/s with average wind speeds from 5 and 30 m/s. At a mean wind speed of 30 m/s, the horizontal displacement begins at a low level and increases to about 0.5 m. They also said that wind speeds from 0 and 30 m/s are considered low in wind engineering, and that vortex shedding tends to exacerbate variability at low mean wind velocities.

Øiseth, Rönnquist, and Sigbjörnsson (2012) the Hardanger Suspension Bridge was submitted to a finite element study. At a mean wind speed of 40 m/s, the horizontal, vertically, and torsional buffeting action at the mid-span of the beam even though the reaction is obviously narrow banded, there is a substantial background component. The spectral densities calculated as the mean value of the spectral density of 20 time series of the horizontal, vertical, and torsional responses at the mid-span at a mean flow speed of 40 m/s were coupled with the multi-mode frequency domain results. The graph depicts the frequency ranges in which the spectral densities deviate considerably from zero. The spectral density for all response components corresponds exceptionally well with the spectral density obtained in the frequency domain throughout the whole range of frequencies, as shown by the findings.

2.3 Structural Analysis of Cable-Stayed Bridge

2.3.1 Development of Finite Element Model

Computers have become an effective tool for analyzing the stability of complicated structures as technology advances. Numerical approaches have the advantages of saving time and money, not being limited by space or equipment, and gathering data quickly as compared to physical tests. In the analysis of complex structural problems, the finite element approach is commonly utilized. Each component of a bridge can be simplified as beam or truss elements using finite element software, and the material characteristics and connections can also be established using the software. The model may generate potential bridge responses matching to various conditions after applying various load. The issue in modeling cable-stayed bridges with finite elements is determining the type of element used to represent the cables. Stay cables are tension elements that can only withstand axial tensile stresses and have no bending resistance. In finite element models, the 2-node truss element, which has the same behavior, is extensively utilized. The effect of cable sag caused by its own weight, on the other hand, causes geometric nonlinearity in the bridge structure and must be taken into account in the numerical analysis.

Cheng and Lau (2002) looked explored the effect of employing three-node and multi-link cables instead of a single truss cable on the bridge's dynamic behavior, with the goal of compensating for the cable's nonlinearity.

The multi-link cable model reflected the transverse motion of cables due to vibrations better than the single truss cable model, according to the findings. It was possible to see how the stay cables and deck moved together. The bridge model with multi-link cables also had similar initial few frequencies to the analytical, demonstrating that the cable vibration model has little effect on other portions of the bridge.

2.3.2 Bridge Models' Dynamic Analysis and Natural Frequencies

The equation of motion, which employs the principle of a system vibration, describes the dynamic motion of the bridge.

Criag and Kurdila (2006) the three basic structural features of the structure that govern its dynamic response are mass, stiffness, and damping qualities, as described. Vibration frequency can be used to define the structure's dynamic response. The imposed dynamic load must be compared to the structure's natural frequency in the dynamic analysis of the bridge model to avoid resonance when they both reach the same value. Otherwise, resonance will cause the dynamic response to be amplified until the bridge fails.

2.3.3 Wind Load Simulation

Because long-span bridges are thin structures, they are vulnerable to wind-induced aerodynamic instabilities. which has been studied for decades since the Tacoma Narrows bridge collapsed in 1940 (Billah and Scanlan 1991). The interaction between the wind load and the structure causes the bridge's aerodynamic response, which is a mix of the mean wind load and dynamic wind load. As per the effect of pressure distribution, the mean wind load can be divided into three components: lift force, drag force, and torsional moment.

Scanlan (1996) the mean wind load formulae for the three components were obtained. Wind force coefficients are dimensionless and determined by wind tunnel testing. The size of the model, air density, wind attack angle, and wind velocity are all controlled in a wind tunnel experiment. To find the unknown coefficients, all known values are substituted.

Diana et al. (2013) on the Messina bridge model, wind tunnel tests were conducted to investigate the wind force coefficients. The authors discovered that the values obtained for a smooth deck fluctuate with wind attack angles, while wind velocity has little effect on the coefficients values. When compared to a smooth surface, roughness can affect the lift and moment coefficients when changing wind velocity. Furthermore, the values of the three coefficients corresponding to a smooth surface are similar to those obtained when a rough surface is assumed at low wind speeds.

2.3.4 Fourier Analysis

Fourier analysis is used in dynamic analysis to convert a time-domain response to a frequency-domain response. The dynamic response is described using the Fourier series, which divides it into frequency components with separate frequencies, phases, and amplitudes. The Fourier series technique can then be extended to a Fourier Transform in terms of infinite-period signals (Reisham and Heerah 2009).

The Fast Fourier transform (FFT) was introduced by Newland (2012) which is a computer algorithm for performing the DFT or inverse discrete Fourier transform. The FFT is more efficient since it needs N to be an integral power of two, which cuts down on the time it takes to calculate the DFT of the response time history (Reisham and Heerah 2009). The time history signal can be converted into the frequency domain with amplitude fluctuation using the FFT. The signal function for the random signal is considered to be white noise, which includes all of the frequencies in the frequency spectrum.

2.4 Structural Behavior of Bridge with Damaged Cables

The structural performance of cable-stayed bridges is increasingly being studied using finite element models. Unless specific parameters must be determined by physical tests, the rest of the study can be done using numerical models. The results of the model can be a good predictor of the actual reaction .Damage to the stay cables in these skinny bridges should be investigated to see if they pose a safety risk.

The impact of ruptured cables on the structural performance for stayed-cable bridges were explored by Kao and Kou (2010). The authors performed a numerical study on a three-span cable-stayed bridge with a main span of 600 m, removing cables one by one in order to analyze the resulting remaining cable forces, the tower's horizontal displacement, and the drooping of the main span's center. The cables adjacent to ruptured cables suffered an increase in cable forces, according to the authors. The tower and the center of the main span both experienced severe displacement when the outermost cable ruptured. Cable ruptures have also been found to cause uplifts in other areas of the deck.

Vikas et al. (2013) investigated the influence of cable deterioration on the structural behavior of cable-stayed bridges using a numerical study of a bridge with a main deck of 220 m. To simulate the damaged cables, the authors utilized the loss of elastic of rigidity and cross-sectional area as a function of corrosion level. In their model, corrosion is inversely related to modulus and cross-sectional area. The authors ran static and dynamic assessments on the bridge model, with one wire reaching and collapsing at its ultimate strength. The second wire on the left side, close to the mid-span, was the one that failed. The internal tensions of cables around the burst cable were raised, whereas those of cables away from the damaged cable were reduced, according to the authors. Around the place where the cable failed, the deflection of half deck increased by 6.9%. The dynamic study indicated the progressive drop in natural frequency even as corrosion level rose.

A progressive collapse study was proposed by Das et al. (2016) for the cable-stayed bridge with a 457.2 m main span and cable failures in the side span. The response was extracted in crucial points such as the center of the span, the pylon's top, and the end cables using a nonlinear dynamic approach. This analysis took into account multiple cable failures. According to the findings, the bridge's end cables on both sides were the most susceptible, increasing the likelihood of a failure over the entire structure. The failure of the wires caused more deflection on the pylon top and deck.

Olamigoke (2018) investigated the load redistribution along a three-plane cable-stayed bridge after a sudden cable failure. It's fascinating to learn that a cable loss in one plane might lead to an increase in tension in another plane. Previous study looked at how corrosion or unforeseen occurrences affected the structural response of cable-stayed bridges. Only a few studies have looked into the structural response of a cable-stayed bridge that has been corroded over time.

2.5 Bridges with Cable-Stayed Spans that have been subjected to Blast Loads

The damage criteria and dynamic behavior of cable-stayed bridges subjected to blast loads are discussed in this section. The outcomes of this field's research are confined to full-scale cable-stayed bridges that have been simulated and analyzed by many scholars (Hao and Tang 2010).

Two kinds of pylons are used in a cable-stayed bridge (a hollow steel box and a solid steel box).

The issue of blast load on a concrete-filled composite pylon was examined (Jin Son and Lee 2011). Using dynamic-nonlinear analysis and a mixed Lagrangian and Eulerian model, a vehicle bomb explosion scenario was investigated and simulated. The explicit numerical program used in this work to model the spatial and time fluctuation of the blast load and shock wave was (Jin Son and Lee 2011). Under the same quantity of explosive materials, a concrete filled pylon may survive, but a hollow steel box section would collapse owing to a strong P-effect, according to the numerical model's findings (Jin Son and Lee 2011).

The influence of blast loads was investigated using a numerical model (Xianlong et al. 2009). The Shanghai Minpu II Bridge was chosen as the model for the standard. An 800 kilogram TNT explosive charge was assumed to be placed on top the deck of a bridge. For blast load simulation, the ANSYS AUTODYN computer program (2007) was utilized. The stress concentration caused by the blast load was found to be limited to the explosion location, and cracks were found to be scattered in tiny areas.

Hao and Tang (2010) studied two different kind of decks: a concrete back span and a steel composite mid span. The steel deck suffered less damage than the deck of concrete due to differences in material qualities. The damaged section deck of concrete measures 30 meters in length and 20 meters in breadth, while the damaged zone of the steel deck measures 11.5 meters in width and 10 meters in length.

The behavior and failure mode of a bridge deck subjected to blast loads were investigated by J. Son and Astaneh-Asl (2012). A nonlinear finite-element simulation was used to build a typical orthotropic deck for cable-stayed bridges in order to model blast loadings. Furthermore, J. Son and Astaneh-Asl (2012) adopted a novel blast-resistant technology (named the "fuse system") that may be used in the design and construction of progressive-collapse bridges. Vertical buckling failure (similar to a hollow steel tube pylon) is the primary collapse mechanism in this conventional orthotropic deck due to significant P-effects. This disaster, however, did not result in the bridge's full collapse.

According to experts, the cable anchoring zone of a cable-stayed bridge is one of the most critical places under blast impact (Williamson and Winget 2005). Direct blast load contact seldom damages the cable, but damage near the anchoring zone causes the cable to lose support and, as a result, the stay. A few cables in the bridge model, for example, lost their support owing to damage to the anchoring location in the concrete decking (Hao and Tang 2010). 300 kg of TNT thrown 1 m above this concrete deck might result in concrete spalling on the bottom side and damage to the anchoring zone. In the instance of 1,000 kg of TNT, three cables were ripped, resulting in a maximum bridge span displacement of 4.54 m and a lateral displacement of the tower of 2.14 m. Despite the loss of some cables due to concrete deck damage, the overall impact on the bridge is minimal. The damage to the steel deck for the similar bridge models was not substantial due to the durability of structural components made of steel.

2.6 In Various Other Studies

Freire, Negrão, and Lopes (2006) studied geometrical nonlinearities such as the sag effect, large displacement, and beam-column effects were taken into account when analyzing a cable-stayed bridge. They did not, however, include material nonlinearity.

The significance of geometrically nonlinear effects in the structural static analysis of steel cable-stayed bridges is explained. Linear, pseudo-linear, and nonlinear techniques are used to evaluate a finite element model. In the pseudo-linear technique, the modified elastic modulus is used. The nonlinear analysis includes cable sag, significant displacement, and beam-column effects. Both cable sag and substantial displacement generate the most severe nonlinear effects in those systems, according to the research. Beam-column effects are unimportant for service loads. Both the modified modulus element and the pseudo-linear method look to be very restrictive, if not outright incorrect.

Xi and Kuang (1999) proposed a fully nonlinear analysis method based on the energy method of analysis for the in-plane ultimate load capacity of cable-stayed bridges. The potential energy of the entire bridge, including the bridge deck, stayed cables, and pylons, as well as the work done by external loads, is factored into the bridge energy calculation. Both geometric and material nonlinearities are included in the study. The method is simple to implement and has a high rate of convergence. The predictions of the proposed approach are in good agreement with experimental data.

Using an Eigen-solver, and by Tang, Shu, and Wang (2001) the stability of cable-stayed bridges was examined. The purpose of this research is to compare the buckling loads predicted using finite element methods with those computed using an Eigen-solver to assess the stability of steel cable-stayed bridges. Engineers have recently become interested in cable-stayed bridges because of its structural characteristics and aesthetic appeal. This research looked at a conventional three-span steel cable-stayed bridge with a range of design features. The numerical findings demonstrate that structural behavior is influenced by design factors. Cable configurations and pylon height are another significant factor in buckling estimates for this sort of bridge. According to numerical studies, bridges supported by a pylon with a harp-type cable arrangement have higher critical loads than bridges supported by a pylon with a fan-type cable arrangement. The form of the pylon, on the other hand, has no bearing on the critical load of this type of bridge. All numerical data was provided in tabular and graphical formats and was non-dimensionalized.

Ermopoulos, Vlahinos, and Wang (1992) they also looked at the stability of cable-stayed bridges, but they concentrated on the Eigen value problem rather than the nonlinear one. The feasibility of two-pylon cable-stayed bridges is examined. The location of the cables, the height of the pylons, and the pylon to deck moment of inertia ratio are all things to think about. An evenly distributed load and a moving concentrated load are the two sorts of loads to consider. The position and size of the minimal elastic critical loads are estimated for various values of the aforementioned parameters and load combinations. In the meanwhile, the effect of the supports' fixation on critical loads is being studied. Designers may find the results valuable because they are presented in a graphical manner.

W.-X. Ren (1999) was examined the long-term behavior of cable-stayed bridges. The research investigates the nonlinear static and ultimate behavior of a long-span cable-stayed bridge, as well as the bridge's overall safety, up to failure. Both geometric and material nonlinearities are included in the study. Geometric nonlinearities are caused by the cable sag effect, axial force-bending interaction, and significant displacement effect. Material nonlinearities occur when one or more bridge elements exceed their unique elastic limits.

L. Wang and Liu (2011) the effect of the bridge reaching its limit condition on the cables was examined. In the context of a long-span composite girder cable-stayed bridge with three pylons that is under construction for research, this study builds two models of the entire bridge by addressing structural geometric nonlinearity. The failure loads and structural internal forces of the two models are compared to determine the effects of a stable cable on the cable-stayed bridge's ultimate load-carrying capacity. According to this study, the internal forces in the main girder, middle pylon, and stayed cables in the bridge with stable cables are smaller and their distributions are more reasonable under the failure load than in the bridge without stable cables. As a result, owing to the stable cables, long-span composite girder cable-stayed bridges with three pylons may effectively enhance their ultimate load-carrying capacity.

Kyung (2006) was examined the effective buckling length of a girder x using elastic and inelastic buckling analysis, and a design approach for structural components was developed. A practical stability design technique for major members of cable-stayed bridges is created and addressed using a design example. For this reason, initial tensions of stay cables and axial forces of major components are estimated using initial shape analysis of bridges under dead loads. Using system elastic/inelastic buckling analysis and second-order elastic analysis, the effective buckling length and bending moments for major girder and pylon components exposed to both axial forces and moments are calculated. The effects of live loads on effective buckling lengths are investigated using three load combinations of dead and live loads, resulting in the maximum load effects due to live loads.

Kim et al. (2011) built a cable-stayed bridge analysis software and conducted a case study on cable-stayed bridge limit states. This study uses geometric nonlinear finite-element analysis to evaluate the structural stability of cable-stayed bridges, taking into consideration different geometric nonlinearities such as cable sag, girder and mast beam-column effect, and large displacement impact. The nature of this study is analytic. A nonlinear frame element and a nonlinear equivalent truss element were used to simulate the girder, mast, and cable member. The traffic loads were employed to generate the live-load instances used in this research. In order to conduct an acceptable analytic research, first shape studies in the dead-load condition were done before live-load analysis.

This study compared the geometric nonlinear reactions of cable-stayed bridges with various cable arrangement types. The effect of geometric shapes on the structural stability of cable-stayed bridges was investigated in this parametric study, which used various geometric parameters such as the cable arrangement type, the girder and mast stiffness ratios, the area of the cables, and the number of cables to conduct parametric studies on the features of structural stability in critical live-load instances.

Lozano-Galant et al. (2012) presented a cantilever-based analysis technique for cable-stayed bridges with temporary supports; however, they did not investigate the long-term behavior of cable-stayed bridges. On the other hand, a number of experimental investigations have been carried out.

A field test was conducted by W. X. Ren, Peng, and Lin (2005) to determine the dynamic characteristics of cable stayed bridges. The Qingzhou cable-stayed bridge in Fuzhou, China, was subjected to an analytical and experimental modal study. With a main span of 605 meters, it is the world's longest finished composite-deck cable-stayed bridge. To obtain the analytical frequencies and mode shapes, an analytical modal analysis is performed on the produced three-dimensional finite element model, starting with the deformed configuration. Field ambient vibration testing on the bridge surface and all stay cables was completed just before to the bridge's opening. The output-only modal parameter identification is then performed using peak picking of the average normalized power spectral densities in the frequency domain and stochastic subspace identification in the time domain. The findings of the finite element and ambient vibration tests reveal a significant link. The results of the analytical and experimental modal investigations are presented to provide a complete picture of the bridge's dynamic characteristics. The primary modes of this sort of large span cable-stayed bridge may be determined via ambient vibration testing at frequencies below 1.0 Hz. Based on the results of ambient vibration tests, the validated finite element model can be used as a baseline for more precise dynamic response prediction and long-term health monitoring of the bridge.

To demonstrate the effect of active damping, El Ouni, Ben Kahla, and Preumont (2012) conducted a modal analysis experiment These were, however, experimental investigations of the dynamic behavior of cable-stayed bridges. This work presents a nonlinear dynamic analysis of a three-dimensional cable-stayed bridge in the building phase with parametric stimulation. To account for quadratic and cubic nonlinear couplings between in-plane and out-of-plane motion, a nonlinear inclined cable with low sag is coupled with a finite element model of a cable stayed bridge.

The structure was successfully included active damping using collocated displacement actuator–force sensor pairs located on each wire, as well as a robust control approach based on decentralized collocated Integral Force Feedback. Under parametric stimulation, the impact of excitation amplitude and active damping on the steady state response of the stay cable is studied computationally and empirically.

Between the cable and the deck, there is an energy transfer mechanism. Quantitatively, the numerical and experimental results are in good accord.

Furthermore, Shao et al. (2005) they looked into the local stability of the main girder, but their research was limited to a single bridge model. The researchers also discussed the bridge's main components' design approach. The three points below will be emphasized. To begin, the pylon's weight, as well as all of the dead loads and a portion of the active loads on the main girder, must be balanced. Second, the primary girder should be an orthotropic steel-concrete composite box girder due to the increased safety and weight reduction of this type of construction. Third, the cable stays should be anchored along the pylon's neutral axis to minimize the formation of excessive secondary moments caused by different anchor approaches.

Also, Song and Kim (2007) utilized a variety of cables to illustrate how cable stayed bridges fall. Although their research concentrated on the behavior of cable-stayed bridges in their initial condition, they took into account both material and geometrical nonlinearity. As previously stated, the axial forces acting on the girder are greatest when the model is in the construction stage. This study conducts an experimental and analytical investigation into the ultimate behavior of two different types of cable stayed bridges during the construction stage when the balanced cantilever method is used.

2.7 Summary of Literature Survey

Many references about cable-stayed bridge and the interface between it components are submitted. As noted above there is a gap in the static analysis even some researchers worked in this line but no one of them deal with the problem directly. Some of them used dynamic analysis instead of static analysis for cable-stayed bridge. We note that most of the studies were directed towards analyzing the loads in stayed-bridges as a dynamic analysis, and that a few researchers moved towards analyzing the loads by static analysis, so the researcher chose to move towards the static analysis of the loads affecting the cable-stayed bridge. In recent years, serious issues have arisen in the decks of each bridge, as well as in the cables. When vehicles travel through the structure, inspectors have noticed movement in the deck panels. A precise computer model of the part of the composite cable-stayed bridge was created to answer the question of possible causes of the bridge damage. The slip calculations between cable-stayed bridge components are estimated. This will allow you to determine whether the bridge is performing to the standards for which it was built. The computer model will allow for a more in-depth examination of the many situations that the bridge may face. Furthermore, there is no published information on the appropriate method for performing static analysis in cable-stayed bridges.

CHAPTER III

BRIDGE FINITE ELEMENT MODELLING

3.1 General

It is preferable to limit the number of columns used in the design and construction of a bridge for long span crossing. Cable-stayed and suspension bridges have the ability to span a long distance with the least amount of column support. The extra space created by removing the columns under the bridge can be used for a wide highway, a broad vassal cleared way, or a public park, among other things. Over the last century, cable-stayed and suspension bridges have become increasingly popular. Cable-stayed bridges, on the other hand, have become increasingly popular in recent years, particularly for bridges with a primary span of less than 800 meters. The installation methods, economics, and aerodynamic stability of cable-stayed bridges are three important factors that have increased their applicability.

This section will go over the benefits and drawbacks of some of the many different components and combinations that can be used in cable-stayed bridges.

- Layout for a Longitudinal Stay

The cable steel's weight used, the force in the axial direction introduced into the stiffening girder, the axial forces and moments of bending imposed a pylon, the stiffness of the bridge system, the ease of anchorage or saddle design, and the aesthetic qualities of the stay layout are just a few of the many factors to consider when evaluating the longitudinal stay layout of a bridge.

- Type of Pylon

Compatibility with the chosen stay layout, efficiency in resisting transverse loads, size of the space occupied by the pylon's base, needed median width, and constructability are all factors to consider while choosing a pylon type for a bridge. Tower with a cantilever.

- Layout of the Transverse Stay

When examining the transverse stay layout, look at how well it resists unbalanced loads on the road and how much vertical clearance there is above the road.

- Type of Girder for Stiffening

Flexural stiffness, torsional stiffness, aerodynamic stability, dead load, and constructability are all factors to consider when deciding which type of stiffening girder to utilize.

- Techniques for Anchoring

The usual forces acting on the stiffening girder and the forces that must be withstood by the foundations at the abutments are the two key factors to consider when choosing an anchoring method.

- Pylon Fixity in the Longitudinal Direction

A major consideration when investigating the longitudinal fixity of a pylon is its effective length in the longitudinal direction. If buckling is the controlling factor in the pylon design, then a pylon with a longer effective length will require a larger cross section to carry a given load than a pylon with a shorter effective length.

- System Options for Cable-Stayed Bridges

The preceding sections introduced some of the advantages and disadvantages of cable-stayed bridge system components. Selecting the most efficient combination of system components for a cable-stayed bridge for a given situation requires consideration of the issues presented, as well as many more which were not discussed in this document. There are numerous methods that can be used, and evaluating which one is ideal for a specific case is a fascinating and difficult challenge (Stuffle 2006).

3.2 Cable-Stayed Bridge

A cable-stayed bridge, being a type of cable-stayed system, has a higher crossover potential than other types of bridges. The tower, cables, and girder make up a cable-stayed bridge. In cable-stayed bridges, the most common section is a box-shaped pre-stressed concrete section. Because the beam and girder are supported by stayed-cables fixed on the tower, the cable-stayed bridge is part of the composite structure system. The stayed-cables are directly and under considerable pressure fixed in the girder. The cable's horizontal force causes the girder to be pressed; as a result, the girder and the tower are compressed, the fundamental distinction between a suspension bridge and a cable-stayed bridge is this. Because the cable-stayed bridge's girder is directly linked to the tower by pre-tensioned cables, the main girder's bending and torsional rigidity improve, resulting in superior dynamic characteristics than a suspension bridge.

The maximum value of the internal force and deformation of the structure with the closest connection to the natural frequency of vibration and mode of vibration, are intrinsic dynamic characteristics of the vibration system. The experimental model of this work was chosen as a reduction model based on Wang's model (P. H. Wang and Yang 1996) to replicate a genuine cabled bridge, The experimental model is built at a scale of 1/68 and has two pylons (61 m per pylon) and three spans (total length of 610 m). The girder's length and pylon's height are 4500 and 1480 mm, respectively, according with law of similarity.

To determine the maximum forces with moments at the crucial components of the bridge under a variety of loading conditions, a global analysis is required. The deck's local analysis is generally handled separately from the global analysis. Computer analysis is required for an accurate and effective evaluation of bending and torsion effects. Programs come in a variety of levels of intricacy and capacity, and the designer's choice will typically be based on his or her computer capabilities. However, quite simple programs will usually suffice for global analysis of what is fundamentally a simple structure. More detailed analysis, possibly involving the use of finite element programs, may be required if a cable-stayed bridge with very little distortion restraint is needed.

3.3 Analysis Approach

Many of the problems experienced in steel structures and other structural elements are combined in cable-stayed bridge modeling and analysis, as well as particular concerns arising from the interaction and force sharing between structural steel plus other structural components. The selection of the mathematical model and associated analysis method is an important aspect of the structural analysis process. There are few definitive rules for selecting an analysis method. The number of permutations that may be created by combining various complex physical characteristics and mathematical models is nearly infinite. This selection should be based on an assessment of the structure's nature and complexity, a comprehensive comprehension of the predicted behavior, and a complete grasp of the capabilities and limits of the different analysis alternatives. The choice of an appropriate analysis technique is critical to obtaining accurate and comprehensive analysis findings that have been refined to the desired level of refinement.

3.3.1 Analysis Dimensionality

Bridges are all three-dimensional structures. The real system, depending on its complexity, might be theoretically reduced to a two-dimensional or one-dimensional model. Some bridges, such as a straight I-girder bridge with moderate or no skew, may be efficiently represented directly using a one-dimensional model. A one-dimensional model analyzes each individual girder outside of the context of the system, and it uses an experimentally based technique to describe the transverse distribution of live load. In some situations, a three-dimensional (3D) grillage (grid) model can be reduced to a two-dimensional (2D) grillage (grid) model. In an indirect and/or approximate manner, this model can include the effect of girder eccentricity and cross frame stiffness. In the remaining cases, a 3D analysis approach is appropriate, which includes the depth of the system in the model as well as associated cross frames, etc. A suitable structural model must first be created in order to assess a cable-stayed bridge system. Before beginning the analysis, important details such as the stiffness or flexibility of each member, as well as the type of connection between the members, must be determined and idealized. A two-dimensional plane frame may be used to evaluate a single-plane system, whereas a three-dimensional plane framework can be used to investigate a double-plane system (Abbu, Ekmekyapar, and Özakça 2013).

3.3.2 Categories of Cable-Stayed Bridge

The structure's nature and complexity, as well as the research goals and resources available, all influence the analysis approach chosen. In broad terms. The cables are one-dimensional elements with a material behavior that is essentially linear elastic, but have a global behavior that is somewhat more complicated due to their construction (parallel wires, strands, or closed cables), the effect of their own weight, and their small bending stiffness. As a result, several mechanical and geometric non-linarites, rheological effects, and execution faults will arise. This complex behavior frequently indicates the need for more thorough investigation. The analysis of cable-stayed bridges as line components requires numerous simplifying assumptions and extra computations at the conclusion of the study to compute all of the individual member load effects (Carlos, Bengoechea, and Stráský 2016).

3.3.3 Complicating Factors

In addition to the inherent degrees of complexity associated with different construction styles, other variables might complicate the analysis of any steel cable-stayed bridge. These challenging aspects can sometimes be addressed outside of the analysis. Nonlinear behavior was also caused by in the pylons and girders, the interplay of loads and axial forces. The magnitude of deflection generated by bending and the compressive force's intensity relative to the buckling load determine the degree of nonlinearity. Nonlinear behavior is also caused by the deformation of the structure, which is constantly changing. The theory of second order is utilized to handle nonlinearity problems by taking into account the entire structure's deflection.

With the use of numerous computer applications, the tiresome work of addressing nonlinear problems can now be avoided. The transfer matrix method is utilized to create the majority of computer programs used in the business today. Engineers who thoroughly comprehend the fundamentals of the bridge system must nonetheless enter the input parameter appropriately into the software. The stiffness and flexibility approaches are available in these programs, such as FRAN, STRESS, or STRUDL.

3.4 Finite Element Approach

The finite element method is considered the most powerful and flexible, and appropriate numerical instrument for resolving a complicated ongoing problem of all the existing analysis methods. The approach has evolved into a critical and frequently required component of engineering analysis and intent. Programs that use finite elements can now be used in all disciplines of engineering thanks to recent advancements in computer technology. The finite element method can easily represent a complex geometry such as that of cable-stayed bridges. The approach may also cope with a variety of material properties, structural component interactions, boundary conditions, and statically or dynamically applied loads. This technique can predict the structural responses of such bridges, both linear and nonlinear, with high accuracy. A Finite Element Method (FEM) has become a standard to predicting & modeling the physical behavior of complicated engineering applications. Engineers in industry, as well as academics at universities and government labs, have embraced commercial Finite Element Analysis (FEA) packages. As a result. There are senior-level engineering programs that include more than just the theory of FEM., and its applications that make use of commonly available FEA tools. The finite element technique, often known as finite element analysis, is a numerical approach for addressing field problems. In a field problem, the geographical distribution of one or more selected variables must be determined. Differential equations or perhaps an analytic expression are used to describe a field situation mathematically. Either description can be used to formulate finite elements. Several finite elements could be viewed as small elements inside a bigger framework. Because the components are connected at nodes, the assemblage is termed a finite element structure.

3.4.1 Finite Element Modelling Methods

Despite the fact that 3D Finite Element Analysis (FEA) modeling is most time-consuming plus demanding. For dynamic and static investigations, it is still the most general and complete approach, covering all aspects impacting structural response. The other approaches were adequate, but their breadth and applicability were limited. The approach has grown in importance aspect of engineering design and analysis as a result of recent advancements in computer technology. Until further notice,, FE computer application are being employed in almost every discipline of engineering.
In any finite element program for solid modeling, there are three techniques to create a model.

The first way is created directly (manually); in this approach, users specify the location for nodes as well as which nodes make up each element. This approach is suitable for simple issues involving line elements (link, beam, pipe) and items with low complexity (rectangles), but it is not recommended for complex solid structures.
The second approach is importing geometry, which involves creating geometry in a CAD system such as Autodesk Inventor and saving it as an import file such as an IGES file. However, mistakes can occur during the import, and the model may not import correctly.

The final option is the solid modeling approach, in which the model is built from simple primitives (rectangles, circles, polygons, blocks, cylinders, and so on) and primitives are combined using Boolean operations.

3.4.2 Three Dimensional Finite Element (3D FE) Modelling

3D analysis is typically the most analytically difficult level of study. A 3D analysis is a finite element modeling approach, similar to a grid analysis. A 3D analysis analyzes the superstructure in three dimensions (flanges, webs, deck, bracing members, etc.) rather than confining the model to a 2D grid of nodes and line elements. Parts or all of a 3D analysis can be performed using commercial software. There are software that can create a 3D model of a cable-stayed bridge and do a live load analysis using influence surface or live load generator approaches. The three-dimensional finite-element method is the most time-consuming and involved, but it really is the most general and accurate technique for dynamic and static analyses, capturing all aspects affecting structural behavior; the other methods were sufficient but limited and applicability.

Because all of the bridge's components are explicitly modelled, a 3D analysis has the benefit of being precise & internally consistent among all regions of the structure. The stiffness characteristics are immediately modeled in a 3D analysis, all aspects of the structure are subjected to direct analysis, with findings provided for each. Rather of approximating the total stiffness characteristics with estimated single values, complex structural configurations are simulated in detail.

For example, in a 3D analysis of the cable-stayed bridge, girder, channel section, two flanges, and one web are all modelled separately, as are the two pylons, each of which has a flange and web, and the cables. A grid analysis, on the other hand, models the complete cable-stayed bridge (flanges, webs, and cables) as a single line element, with the related complicated frames' stiffness determined using simplified formulas or empirical predictions. This level of depth and rigor in a 3D model should presumably result in more accurate analysis. Many designers find the results of 3-D analysis to be less intuitive and difficult to envision and comprehend. As a result, there's a higher chance of making an error or misinterpreting analysis results.

3.4.3 Finite Element Modelling with ANSYS

ANSYS is a finite element analysis tool for solid modeling with substantial structural analysis capabilities. The solid model is made up of important points/nodes, lines, regions, and volumes, in that sequence, with increasing complexity. Before constructing the full model, careful thought must be given to it.

If volumes, regions, or lines are related to existing meshed elements, they cannot be erased after the model is mesh. Depend on size and shape of the entire solid model, the aspect ratio and mesh type must also be determined. There are several solid finite elements to pick from in ANSYS, each with its own specialization.

The type of analysis must first be determined, which might range from structural analysis to thermal analysis to fluid analysis. After determining the type of analysis, select an element type from beam, plate, shell, 2D solid, 3D solid, contact, couple-field, Specialty, and explicit dynamics. In both 2D and 3D analysis, each element has its own set of capabilities and is made up of tetrahedral, triangle, brick, 10 nodes, or 20 node finite elements (Madenci and Guven 2015).

From simple linear static analyses which provide stresses or deformations to modal analysis that determines vibration characteristics but also advanced transient nonlinear phenomena involving dynamic effects and complex behavior, ANSYS structural mechanics solutions can simulate each structural aspect of a product.

All users, from designers to advanced experts, may benefit from ANSYS structural mechanics solutions. The ability to model any product, from single parts to the really complex assemblies with hundreds of components interacting through contacts or relative motions, as well as the large number of material models available and the high quality of the elementary library.

The authenticity of the findings is enhanced by the resilience of the solution methods and the capacity to represent any product, from single parts to highly complex assemblies with hundreds of components interacting through contacts or relative movements.

The ease of use of ANSYS structural mechanics solutions allows product developers to focus on the very important element of the simulation process: comprehending the findings and the influence of design adjustments on the model. ANSYS is a software suite for finite element analysis (FEA). It creates geometry with the help of a preprocessing software engine.

The meshed geometry is then loaded using a solution routine. Finally, in post-processing, it produces the required outcomes (Madenci and Guven 2015).

3.5 Description of Collecting Experimental Bridge Models

This study's experimental model is a reduction model based on Wang's model (P. H. Wang and Yang 1996), which is a two-dimensional model with two pylons (61 m per pylon) and three spans (total length of 610 m); the experimental model is built at a scale of (1/68). The length of both the girder and the height of the pylon are 4500 mm and 1480 mm, respectively, according with law of similarity, as shown in Figure 3.1.

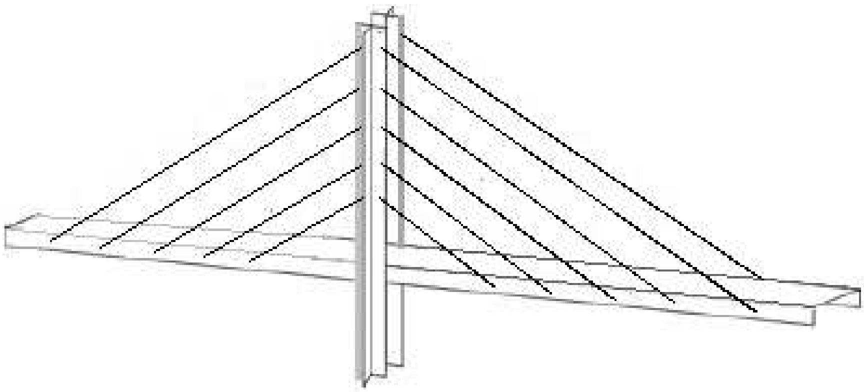

Figure 3.1 The concept of the model

- STAYS

Arranged in a fan system, the cables are made of steel wires with, every model has 20 cables, which are made of 7-strand wires with a diameter of 6 mm. Every cable is anchored end to end at the deck

- DECK

The deck For the Harp type1 models, the smallest size available ready-made channel section (C75×40) is chosen where it consists of two flange. Each one has dimensions (40mm * 8mm) and one web has dimensions (75mm * 5mm) In addition to the total length of the girder is 4500mm. Cables withstand all loads operating on the girder and carry them into the ground via the pylons.

- PYLON

The bridge has two steel pylons with high 1480mm to support the vertical reactions. Every pylon consist of T-section (T50×100) where the flange has dimensions (40mm* 8mm) and web has dimensions (75mm * 5mm). At the base level, the towers are supported by fixed with wall. Due to the obvious loads acting on the girder and tried to resist by cables as well as carried into the ground via the pylons, the towers are subjected to high compressive forces and also bending moments.

(Lee et al. 2015) detailed the test findings of two cable-stayed bridge models. The geometrical arrangement of the bridge model in combination with boundary conditions is shown in Figure 3.2. In this work, the numerical evaluation is used to replicate the behavior of giving a model. The steel pylon's height was 1480 mm, as well as the web and flange thicknesses were 6 mm and 8 mm, respectively. The girder's length is 4500mm, and his width is 40mm.

He also laid out the method in which the two models were formed in terms of the shape of the cable system and the horizontal and vertical distances between each cable and another, as well as the number of cables used, their quality and the cross-sectional area of each. He also showed how the cables were connected to the tower and the girder.

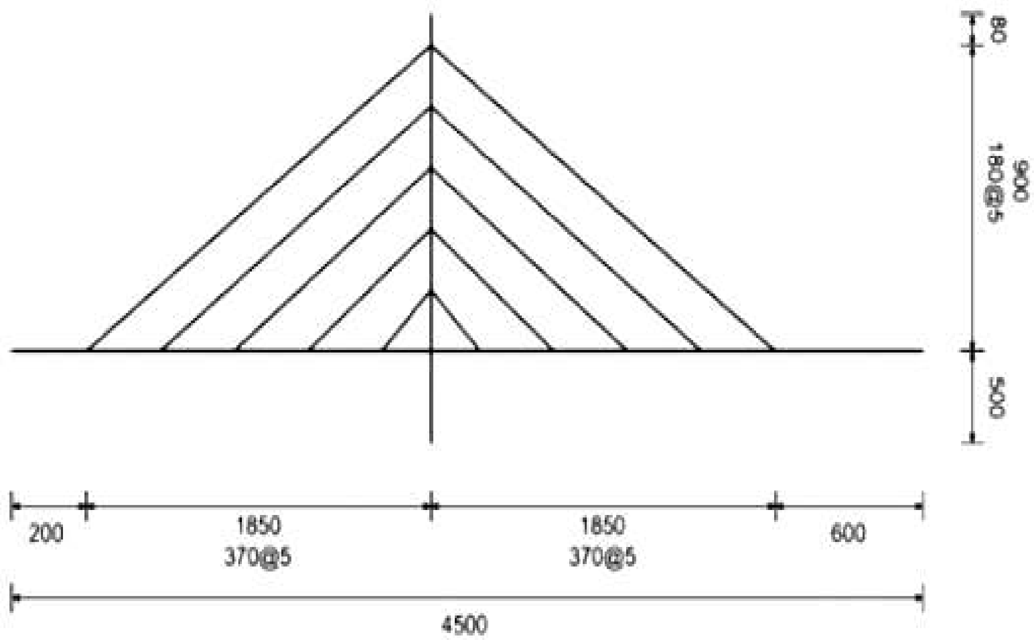

Figure 3.2 The geometrical configuration of the bridge model

3.5.1 Material Properties

Yield stress and the tensile strength of the steel used in construction the two pylon and girder portion are 240MPa and 500MPa, respectively. Steel's elastic modulus is 200GPa. Cable elastic modulus used to build the stays is 205GPa and the cross section area for the strand that used to build the stays is 197.82mm^2. As shown below where Table 3.1 represent. The girder's sectional characteristics and Table 3.2, represent Pylons' sectional characteristics

Table 3.1 The girder's sectional characteristics

	Harp type 1	Harp type 2
A	88.1 mm2	655 mm2
Ix	122,000 mm4	33,404 mm4
Iy	753,000 mm4	475,574 mm4
Py	352.2 KN	262 KN
MPx	4,465.13 KN.mm	1,173.47 KN.mm
Mpy	9,660.55 KN.mm	5,310.37 KN.mm

Table 3.2 Pylons' sectional characteristics

A	1,095 mm2
Ix	161,000 mm4
Iy	669,000 mm4
Py	438 KN
MPx	6,570 KN.mm
Mpy	13,140 KN.mm

A preparatory simulation shows that perhaps the cable force may reach as much more than 800 kgf (8 kN) for an ultimate limit state but such a large tension might well potentially cause deformation at just the hook of the turnbuckle, that also subsequently caused a sudden changes in the cable forces as well as the entire structural behavior.

As a result, the hooks are welded together to form a closed ring to reduce distortion. The installation of a steel bar in the center of the cable allows for the measuring of cable forces. At the steel bar, strain gages are connected.

The stiffness of the structure is affected by cable forces, thus it's critical to compute the precise starting cable forces. The cable forces derived from the initial state analysis are insufficient since the experimental model is meant to replicate a bridge under construction.

3.5.2 Experimental Model Loading

An actuator applied one concentrated load at the girder bridge's mid-span at a distance of 300mm from the termination point. Figure 3.3 shows a testing system with a capacity of 5330 N. The mid-span was subjected to static tests in order to observe the model's elastic behavior. The displacements of mid-span point were measured using linear variable differential transformers (LVDTs).

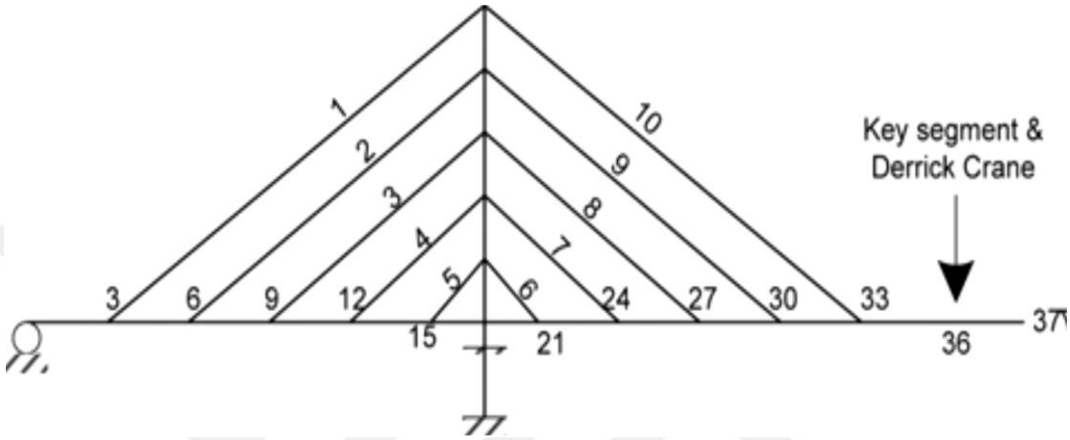

Figure 3.3 Bridge loading

3.6 Description of Numerical Bridge Models

3.6.1 Loading Test for Finite Elements Models

In some engineering issues, the performance of some of the unknown degrees-of-freedom may be known. Several points (nodes), for example, may be expected to have the same displacement in a particular direction. This property may be taken advantage of and enforced in to achieve a precise response with little processing resources. These degrees - of - freedoms can be connected if a given degrees-of-freedom at numerous nodes is predicted to have the same unknown value. Coupling is a technique for making a group of nodes share like D-O-F value. In contrast to the constraint, D-O-F value is normally determined by the solver rather than being supplied by the user. A linked set collection of nodes that are connected in one direction (i.e, one degrees-of-freedom). The coupling is accomplished using the CP command, and the degrees-of-freedom value for all the points to be coupled is the same. For the finite element models of cable-stayed bridge, a single point load was applied, and point loading was simulated by applying a concentrated load at a total of line nodes that are connected restricted on the side span.

3.6.2 Description about the Elements Used in the Model

This study's finite element modeling and analysis were carried out with ANSYS, a general-purpose, multi-discipline finite-element tool with a large library of links and solid elements. The following is a brief summary of the model's components:

Description of the Link180 Element: LINK180 as shown in Figure 3.4 is a three-dimensional spar that may be used in a wide range of engineering programs. Trusses, sagging cables, linkages, springs, and other structures can all be modelled with this element. Each element is a three-dimensional uniaxial compression-tension element with nodal x, y, and z translations as well as three degrees of freedom at each node. Tension-only (cable) and compression-only (gap) options are available. No bending of the element is taken into account, as it is in a pin-jointed structure.

The capabilities of plasticity, creeps, rotation, enormous deflection, and big strain are all covered. LINK180 automatically includes stress-stiffness variables in any research with large-deflection effects. Elasticity, isotropic hardened plasticity, kinematic hardening plasticity, Hill anisotropic plasticity. Chaboche nonlinear hardening plasticity, and creep may all be supported in this way. To simulate the tension/compression-only options, a non-linear iterative solution approach is necessary; as a result, large-deflection effects must be engaged (NLGEOM, ON) prior to the solution phase of the study. Increased mass, hydrodynamic additional mass and loading, and buoyant loading are all choices (ANSYS mechanical Manual (2009)).

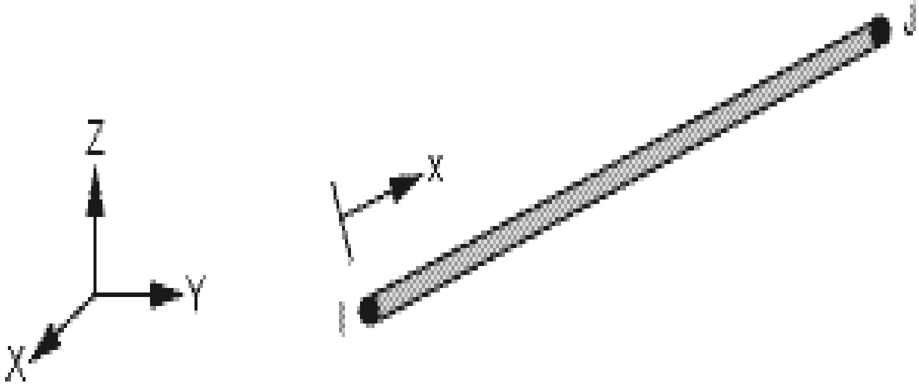

Figure 3.4 link180 Geometry

Description of the SOLID185 element: SOLID185 is a three-dimensional modeling software for solid structures. It is made up of eight nodes, and each has three degrees of freedom: nodal x, y, and z translations. The element's characteristics include plasticity, hyperelasticity, stress stiffening, creep, high deflection, and enormous strain capacities. It also allows you to use combined formulations to model deformations of practically incompressible lastoplastic materials and completely incompressible hyperelastic materials. Figure 3.5 depicts the form, node locations, and element coordinate system for this element (ANSYS mechanical Manual (2009))

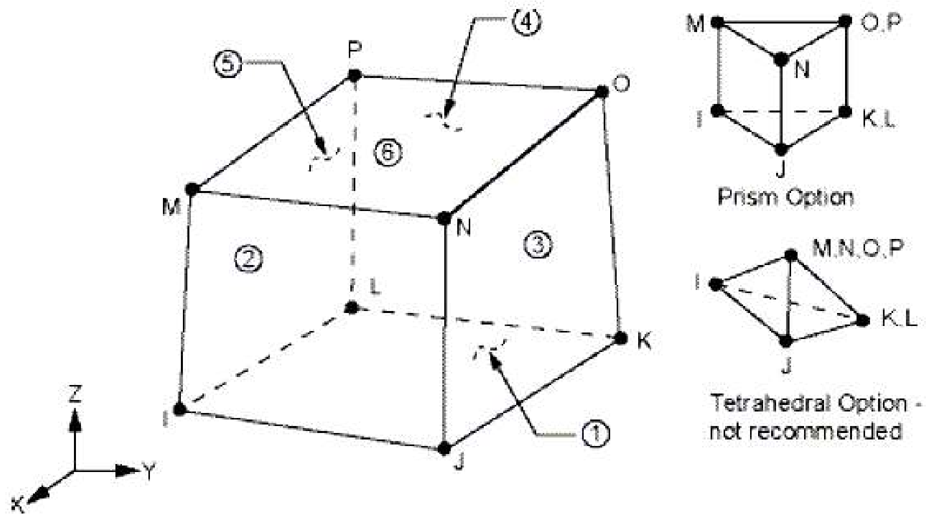

Figure 3.5 Solid185 Geometry

3.6.3 FE MODELS

Pylon or tower, deck, and stay cables are the three primary components of cable-stayed bridges, as discussed in Chapter 1. Three-dimensional line elements are used to model these three components. A three-dimensional linear frame element is used to simulate both the deck and the pylon. The cables are also modelled using a three-dimensional non-linear cable element. The modeling of these components is detailed in the subsections that follow.

A commercial FE program called ANSYS was used to simulate cable-stayed bridge models. Assumedly, the pylon and the girder were connected by full interaction in the middle, and each section of the pylon was linked to the girder. Figure 3.6 presents FE model that is developed using solid elements for both pylon and girder sections and link element for cables. In this model, constrained load is used, as illustrated in Figure 3.7. As illustrated in Figure 3.8, the nodes' vertical translation degrees-of-freedom are linked over the deck's breadth. Coupling is a method of assigning the same D-O-F value to a collection of nodes.

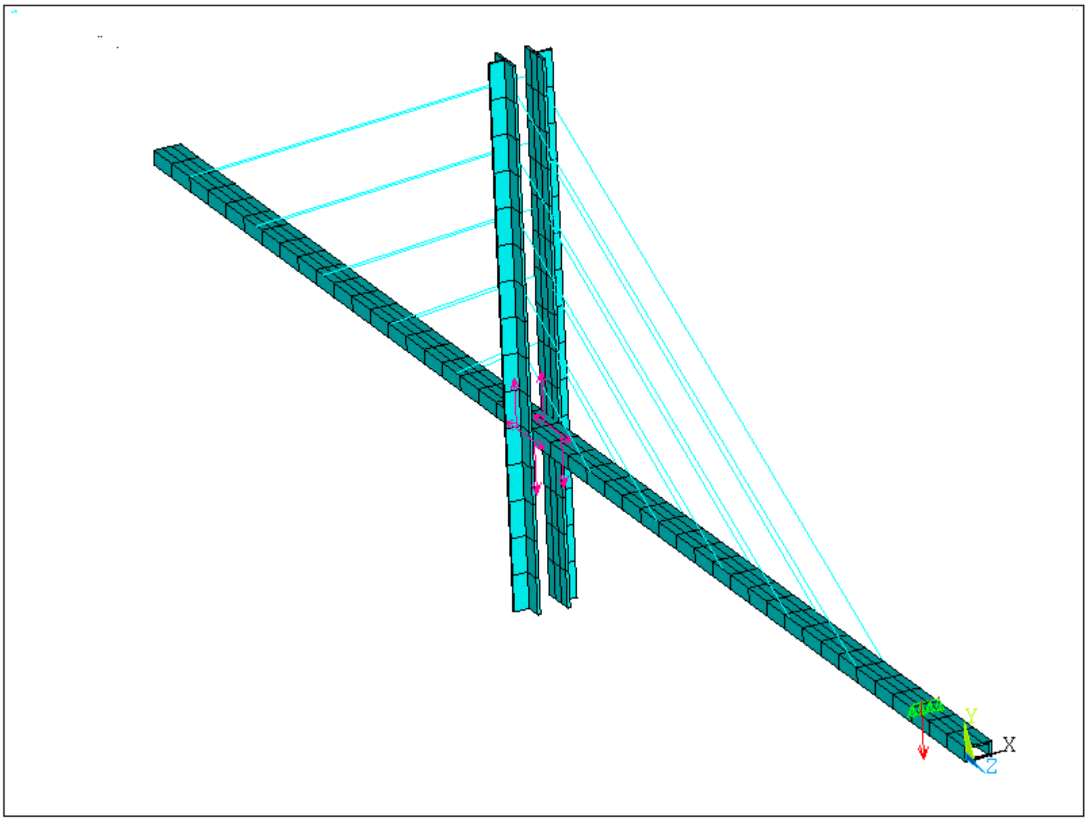

Figure 3.6 FE model by ANSYS program

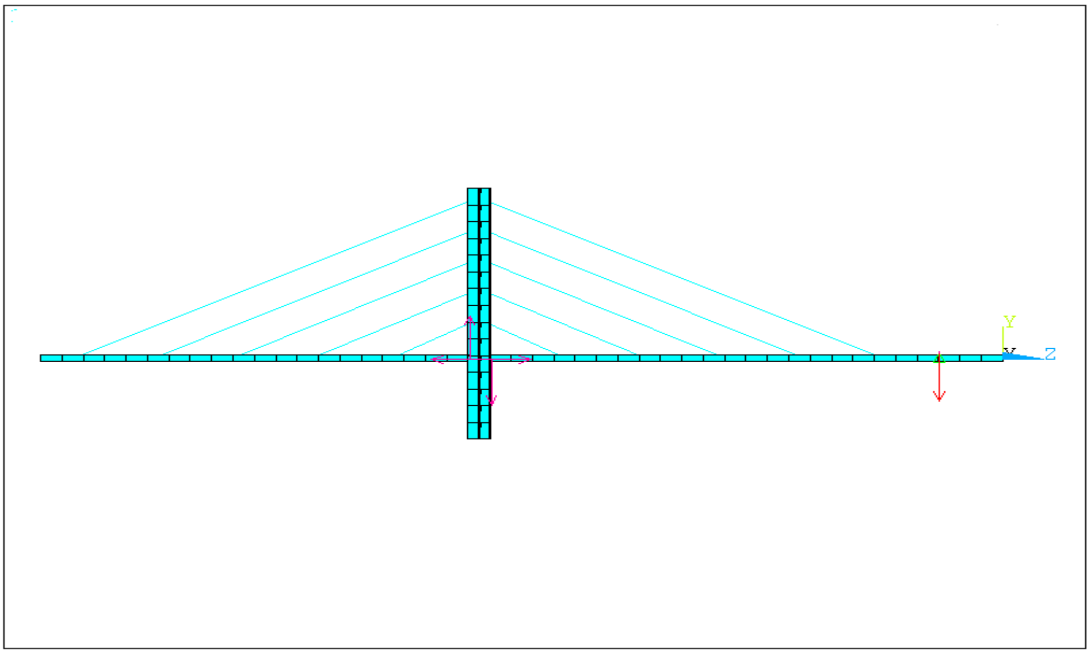

Figure 3.7 Constrained load is applied in FE model

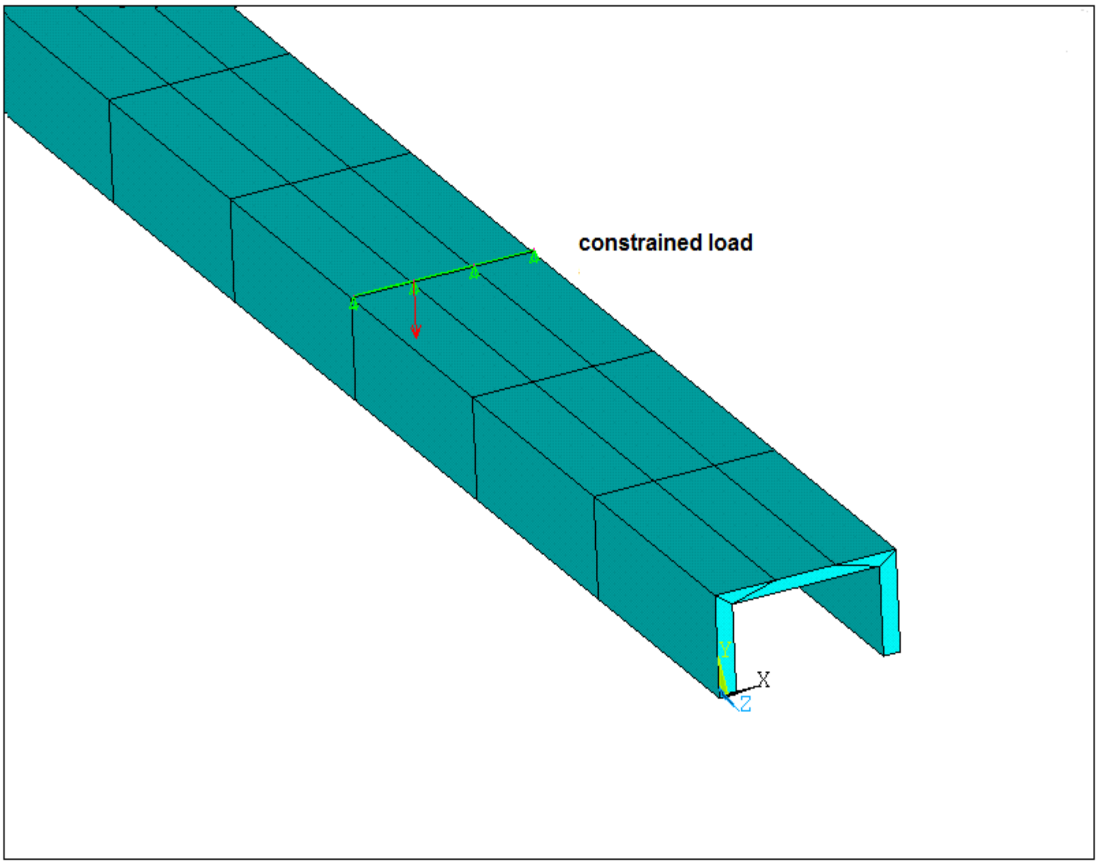

Figure 3.8 Coupling technique for making a group of nodes

Since the model of bridge consists of two sections. Using 3D brick elements would not necessitate a time-consuming modeling process. Models 1 and 2 are built with eight nodes brick elements to evaluate the performance of this modeling technique against solid models and test results. Solid components still depict the steel pylon and girder part of the bridge as in models 1 and 2. Brick elements are only utilized to portray the steel pylon and deck portion of the bridge.

In this study, each flange of the girder is divided into forty six element and the web divided into one hundred thirty-eight rectangle element, while each web was divided into fifteen element in every pylon and every flange in every pylon was divided into forty five element, Also, each cable was considered as one element, as the total cable became twenty elements, solid 185 elements are used to model the pylon and girder in three dimensions. They are made up of eight nodes, each with three degrees-of-freedom. The link180 element is used to simulate a cable constraint between pylons and girders, which are utilized to transfer forces and moments in all of the previous models. The boundary conditions are handled in a way that they replicate the test specimen's simple supported circumstances.

The following assumptions guided the research:

1. The cables Works as a structural member exposed to tension only
2. All materials are elastic and homogeneous.
3. The steel deck and pylons is considered as same material

3.7 ICE Code

In the nineteenth century, using inclined stays as a tension support for a bridge deck was a well-known concept, and there are many examples, especially when the inclined stay is used to give stiffness to the suspension bridge's principal draped cables. Unfortunately, the concept was not well understood at the time. Because the stays could not be tensioned, they became slack under varying load circumstances. The structures were frequently unable to withstand wind-induced oscillations. There have been some major bridge failures, such as the bridge across the Tweed River in Dryburgh, built in 1817, it was just six months after it was built in 1818 that it was destroyed by a storm. As a result, the stay concept has been phased out in England (Macdonald 2008).

Nonetheless, Roebling, an American bridge engineer, expanded on these concepts by designing bridges with cable stays and draped suspension cable.

The most well-known of Roebling's bridges is the Brooklyn Bridge, which he built in 1883. The modern cable-stayed bridge was initially proposed for the repair of numerous bridges spanning the Rhine River in postwar Germany in the early 1950s. These bridges proven to be more cost-effective than suspension or arch bridges for modest spans (Macdonald 2008)**.**

3.7.1 Ratio of Back Span to Main Span

To give the main span a distinct visual emphasis, when designing the bridge's conceptual arrangement, the ratio between both the back span and the main span must be less than 0.5. This ratio, which is equally important technically, influences both the uplift pressures at the anchor pier as well as the limit of load inside the back stay cables supporting the top of the pylon. In the case of the two models that were represented, the ratio of the back space to the main space is 0.418 which is considered less than 0.5 Therefore, the two models are considered within the limits of the specification set by the ICE code

3.7.2 Pylon Height

The overall rigidity of the structure is determined by pylon height. The needed stay size decreases as the stay angle (α) rises, but the pylon height rises. The deflection of the deck, on the other hand, will rise as the stay lengthens. Both the weight of the stay as well as the deflection of the deck are the least when the equation $1/(\sin\alpha \times \cos\alpha)$ is likewise a minimum. As a result, a stay inclination of 458 is the most efficient. When the stay inclination is altered within reasonable limitations, such as 25–658 degrees, the effectiveness of the stay is not greatly harmed in practice. The outer stay, inclined at 258 degrees, will run from the anchor pier to the top of the pylon, as well as the deck panel close to the main span's center. The stay inclined at 658 will be located there.

The optimal pylon height above the deck (H) to main span (L) ratio, (H\L), will be between 0.2 and 0.25, as shown in Figure 3.9.

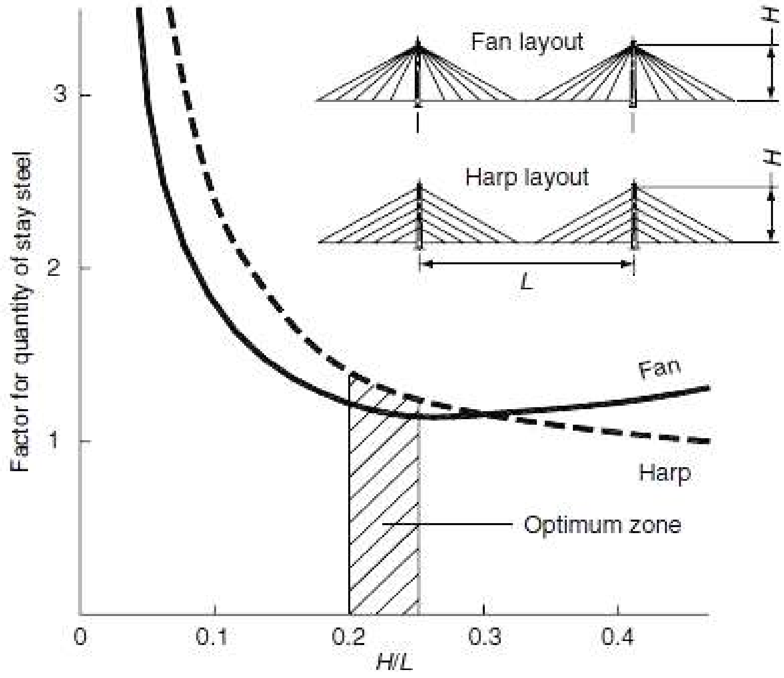

Figure 3.9 Optimum pylon height

In the case of the two models that were represented, the height of the tower to the main span ratio is 0.2, As a result, within the limits of the optimum ratio recommended by ICE code when designing cable-stayed bridges.

3.8 Summary of This Chapter

The theoretical three-dimensional finite-element models developed here are capable of precisely predicting the elastic behavior and mode forms of cable-stayed bridges. The capabilities and limits of (ANSYS), a commercial finite element analysis tool for building a three-dimensional finite-element model capable of anticipating the structural reaction of cable-stayed bridges, are investigated in this study. This study comprises a variety of aspects that are considered and proven through trials, particularly the shape of the girder, which is obviously important in cable stayed bridges.

Vertical displacements at critical sections were predicted numerically and compared to those measured experimentally. The consistency between both the FE models and the experimental models demonstrates that the FE model can assist engineers in cable-stayed bridge design techniques.

A new technique for modeling the cable stayed-bridge has been adopted; this technique depends on the usage of a link element to represent cables and a solid element to represent pylons and girders in this study. This type of element is ideal for modeling cables, pylons, and girders, as well as simulating intricate cable-stayed bridges. It may also be utilized as a practical modeling tool for long span cable-stayed bridges.

CHAPTER IV

EFFECTS OF DIMENSIONAL DETAILS ON BEHAVIOUR OF CABLE-STAYED BRIDGE

4.1 General

Many issues affect design and analysis of cable-stayed bridges, including interactions between the steel deck and girder, local buckling of the cable-stayed bridge's walls, torsional warping, and interactions between different forms of cross-sectional forces. It's difficult to come up with the best design for cable-stayed bridges. This is as a due to the fact of a vast number of factors influence the design, including geometric arrangements, how many stay cables there are, the kind of pylons, the placement of a stay-cables, and the major girder types. Stay cables on a cable-stayed bridge are often post-tensioned to counterbalance the effect of dead load on the deck and pylon, as discussed in the preceding chapter. The performance and cost efficiency of cable-stayed bridges are directly affected by these cable forces. The design variables were the positions of stay-cables along the major girder and pylon, as well as the deck's cross-sectional diameter, pylons, and stay cables. The number of stay cables and the mid-span length were thought to be fixed characteristics in their research (Zadeh 2012).

Medium and long span continuous bridges are two types of cable-stayed bridges. The bottom flange of the bridge girder is susceptible to compressive forces in cable-stayed bridges due to negative moment zones above the inner supports. Because of the bridge's bending moment is generally greatest at the intersection of the tower and the bridge piers, To determine the compressive strength of a cable supported bridge, identify the section shape as well as the metal from which the tower and pillar are made. Cable-stayed bridges' continuous steel girders are often used as major load-bearing elements for multi-span roadway and boardwalk. The use of haunches at the interior supports results in a variation in negative moment strength that is equivalent to the design specifications, enhancing the structural efficiency of the design. The span length, mid-span depth, form and size of section for tower plus girder, and the shape of cables, as well as the configuration and thickness of the flanges and webs for cable-stayed bridge piers, all affect the economics of a cable-stayed bridge profile; All of these should be taken into account to produce the right design for the cable-stayed bridge's adoption and geometry.

The results of a parametric research comparing cable-stayed bridges are showed in this chapter. The static performance of these two models are evaluated by analyzing the following three measures:

1. The girder's maximal deflection at the major span's center.
2. A maximum cable stress in the cable that anchors the main span's center.
3. The girder's maximum bending moment.

4.2 Standard Model (SM)

Using a commercial FE program, cable-stayed bridge models were simulated. Because the materials in the preceding chapter were stressed to their elastic limits. The current research involves a linear analysis of bridge models. Because full contact between the two pylons and the girder was envisaged, they were joined together. 3D modeling of pylons and girders is done with brick parts. They are defined by eight nodes, each with three degrees of freedom. The links element is used to depict a cable constraint between two bodies, in this case a steel pylon and steel girder, which is utilized to transport forces and moments in all of the previous models.

Figure 4.1 presents FE Standard Model (SM), Solid elements are used in the cable-stayed bridge parts, while solid brick is used for the pylons and deck. The steel pylons and deck component of the bridge are only represented by brick elements, but the cables are still represented by links elements. The use of 3D brick pieces is a practical technique to illustrate this model. Because the cross section is entirely made of steel, using 3D brick parts would not complicate the modeling process. Figure 4.1 shows how for a cable-stayed bridge, the vertical movement degrees of freedom of nodes over the deck are connected. Coupling is a technique for ensuring that a group of nodes have the same DOF value. The DOF value, unlike a constraint, is usually calculated by the solver rather than given by the user. A linked set is a set of nodes connected in just one direction. A boundary conditions have been treated in such a form that they resemble test specimen situations that are easily supported. The point load was compared to this sort of loading in the previous chapter, and the findings were more accurate. In this study. For a suitable aspect ratio of the components, every girder flange was divided into 46 square elements, with no divides to every steel flange. The web of the girder, on the other hand, was split into 138 parts. The model also includes two pylons, each with a flange divided into 45 parts and webs subdivided into fifteen square elements.

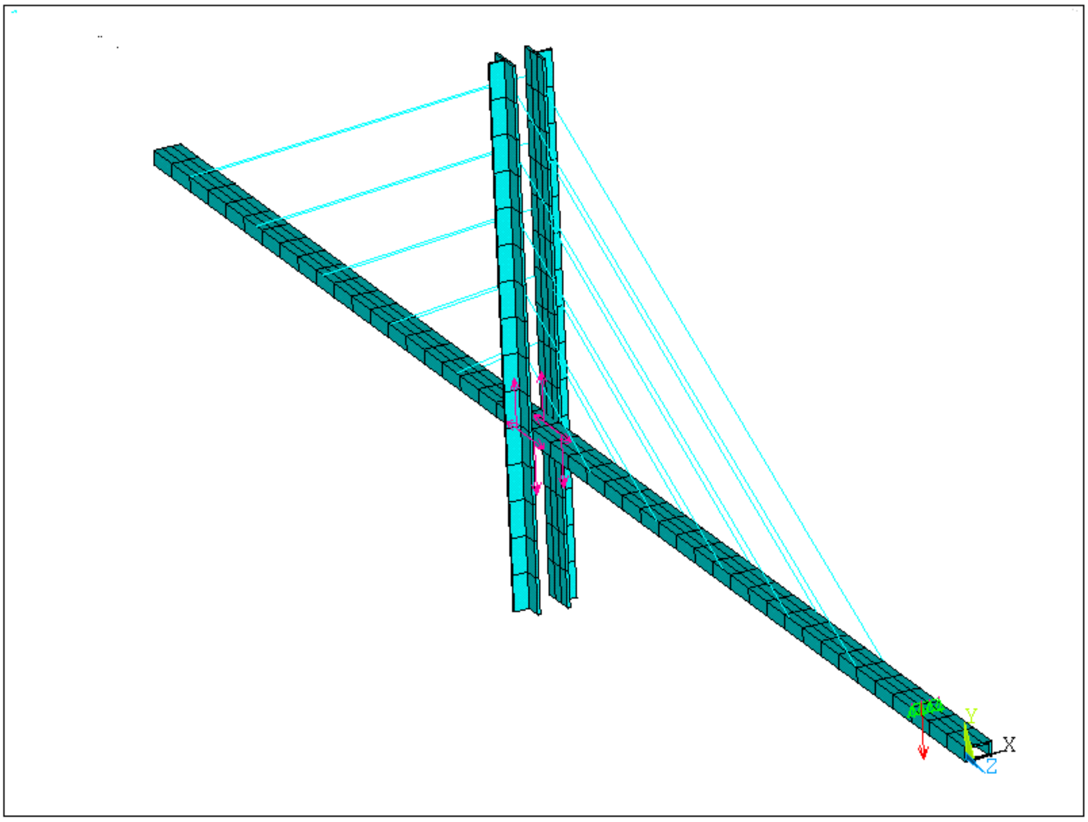

Figure 4.1 Presents FE Standard Model (SM)

4.3 Cross Section Parameter Variations

The structure analyzed is the cable-stayed bridge, with span lengths of the girder 4500mm. It's out-to-out width is 75mm. The thickness of web girder is 5mm and the width of every girder's flange is 40mm and thickness for them 7mm. and the length of every pylon is 1480mm where width of flange is 100mm and thickness 8mm while the width of web is 50mm and thickness is 6mm. All of these dimensions are for model harp1, as for model harp 2, all dimensions are protected and the change is only in the depth of the flanges for the girder where became 25mm. The following are some of the factors that could affect the strength of a cable-stayed bridge:

- Thickness and width of steel flange and web for the pylons and the girder.
- Diameter of the cables.
- Number of the cables that connect between the tower and girder.
- Strength and weight of steel.
- The quality of the method used to connect the cables.
- The quality of the materials from which the cable-stayed bridge is made.
- The length of the space between each towers in relation to the long distances.

The girder's flange depth and the value of the limited load are the most relevant dimensions for the aims of this study. The performance and cost of a cable-stayed bridge are influenced by a variety of factors. The span length, the structure's depth at mid-span, the form and size of the haunches, and the thick of the web and flange are all significant factors to consider. The effect of each of these design characteristics is investigated in this study using a well-designed cable-stayed bridge, referred to as the SM design herein. The parameters investigated in this study are as follows, the Effects of the girder's flange depth (DF) and the constrained load (P). Each of the previously listed parameters was changed to produce a different analysis to compare to the SM condition. Two parameter was changed the change was in DF (the girder's flange depth), constrained load (P), respectively of SM. The resulting cable-stayed bridge was analyzed for each change in a parameter. The results were compared to those from the SM, which revealed a possible alternate analysis that included the selected changes. In this parametric investigation, the SM analysis was also examined for a constant span length. The influence of span length was ignored in this section, and just the cross section parameters were studied. Also, keep in mind that in the analysis, we selected a different load for each model that was equal to the experimental concentrated load values.

4.4 The Combination of Cross Section Parameter Variations

A parametric study focuses on the combination of variation of flange depth and load to explore the effect of various bridge factors on the response of cable-stayed bridge. Two bridge cross-section shapes were evaluated at each parameter and would chick with other factors. For this aim a software utilizing parametric analysis using

ANSYS. Language of Design (APDL) is constructed to examine the behavior of the structure, that contain two values for each parameter and two models were obtained With the exception of the total section depth, the section geometry remains the same across varied bridge span lengths. Table 4.1 shows the values of the parameters.

Table 4.1 Parameters values for cable-stayed bridge

Parameters	**Girder's flange depth (mm)**	**Constrained load (N)**
Value1	40	5330
Value2	25	2500

4.5 The Combination of Variations of Cross Section Parameters and Haunch

The Eigen functions of the cable-stayed bridge are derived by numerical method. The width of girder's web, the vertical variable depth for the girder's flange, In order to evaluate cable-stayed bridges, these functions were combined in the longitudinal direction with suitable finite element boundary conditions in the radial direction. For this purpose a program using ANSYS Parametric Design Language (APDL) was written. This program has values which represents length of girder L, and the depth for the girder's flange DF. In this study. The cross-sectional moment of inertia will vary along the span as the depth of the girder's flange varies, resulting in a non-constant depth design.

The following parameters were investigated in this study:

1. Depth for the girder's flange DF.
2. Constrained load P.

The partition of members into finite elements could be important for the haunch. A haunch, on the other hand, is an element having a constant cross-section. As a result, the effect of varying cross-section (most commonly varying cross-section depth) must be represented using a tighter finite element mesh. There's a big difference between the precision of results for extremely coarse and very fine division. The gap between very fine division and considerably fine division is virtually insignificant. Negative bending resistance at internal supports is often less than positive bending resistance in the mid-span zone of the cable-stayed bridge. As a result, the addition of a haunch may be an alternative for overcoming the deficit because it will improve the capacity of the hogging portion. It is critical to understand the parameters that influence the design of a haunched cable-stayed bridge in order to maximize the structural system's efficiency.

The geometry of the models was determined using the cable-stayed bridge section from Chapter 3.

The effects of the previously listed parameters on stress and deflection are investigated. The findings can be utilized to create analytical and design guidelines for cable-stayed bridge haunches.

4.6 Numerical Results and Discussions

4.6.1 Results for Variations of Cross Section Parameters

Table 4.2 and Figures 4.2-4.7 show the results of the Finite Element Analysis (FEA) employing SM, DF, P, DF1, and P1.

Along with the results of the loading tests and the design values, the design study finds that the stresses and longitudinal movement obtained during the loading test are overstated. A cable-stayed bridge's behavior may be determined by using displacements and stresses obtained from finite element models. It's also possible to compare stress profiles with it.

The cable-stayed bridge's flexural stiffness demonstrated linear elastic behavior in the elastic range of loading. The mid-span deflections in the studies were compared to the test findings in chapter three. The mid-span deflection during the experiment was 245 mm, while it was 224 mm in an analysis done at a load of 5330N for model 1 by using ANSYS program. At a load of 2500N, the mid-span deflection for model 2 was 180 mm in experimental, while it was 167 mm in analytical by ANSYS program. The SM analysis mid-span deflection was compared to the test results. Figures following show the deflection results obtained from FE models with various parameters, which are summarized in Table 4.2:

Table 4.2 Deflection results for FE models

Model name	Constrained load (N)	Girder's flange depth (mm)	Mid-span deflection (Analytical) Results (mm)	Mid-span deflection (Experimental) Results (mm
Harp type1	5330	40	224	245
Harp type2	2500	25	167	180

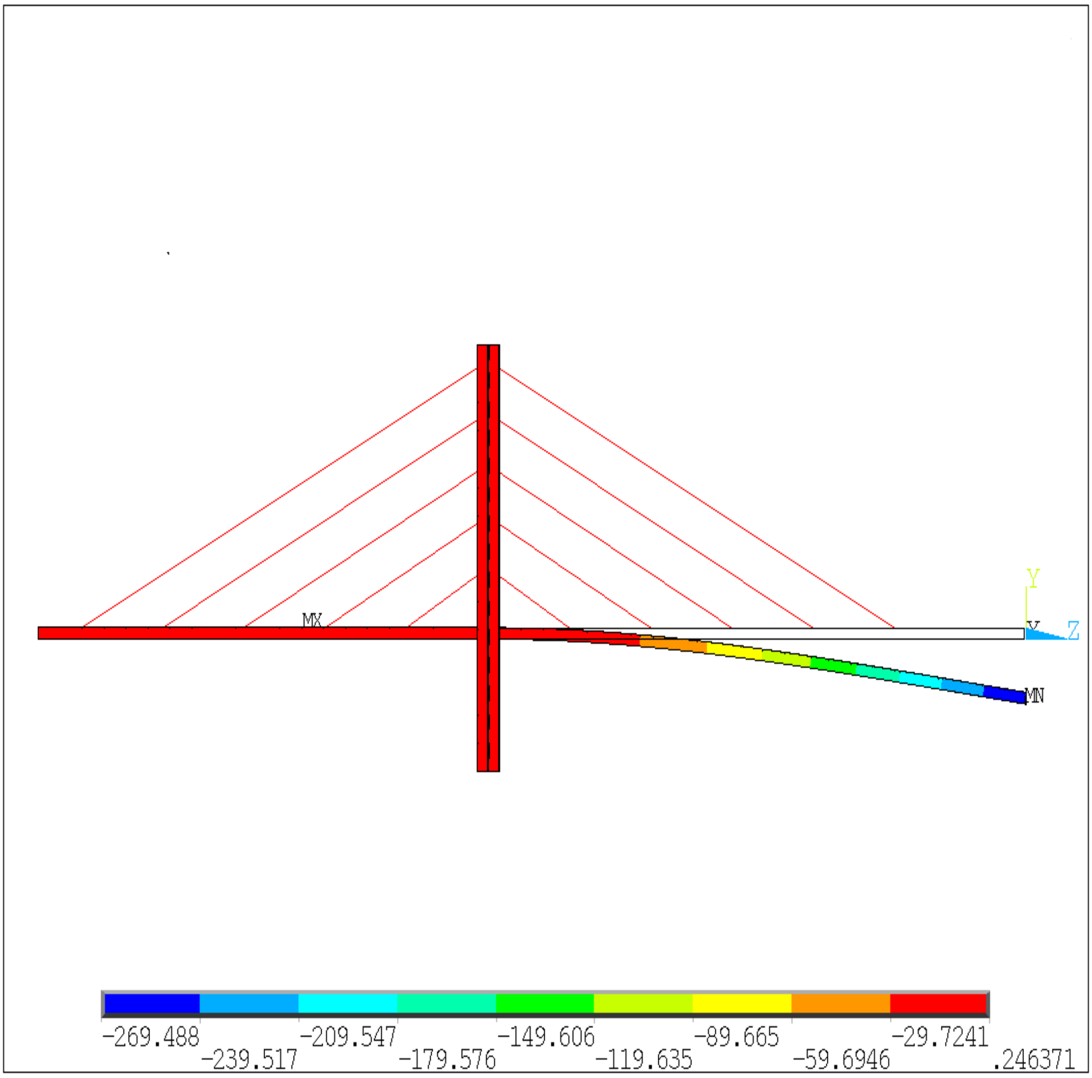

Figure 4.2 Deflection of numerical model Harp Type 1

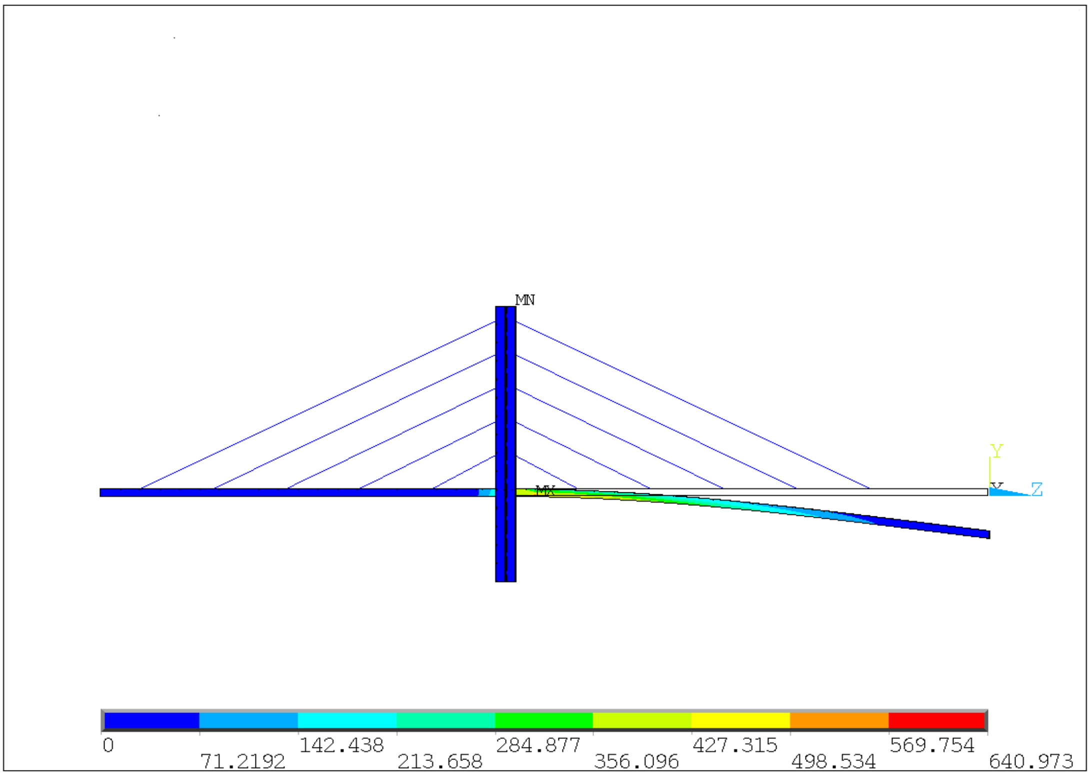

Figure 4.3 Stress of numerical model Harp Type1

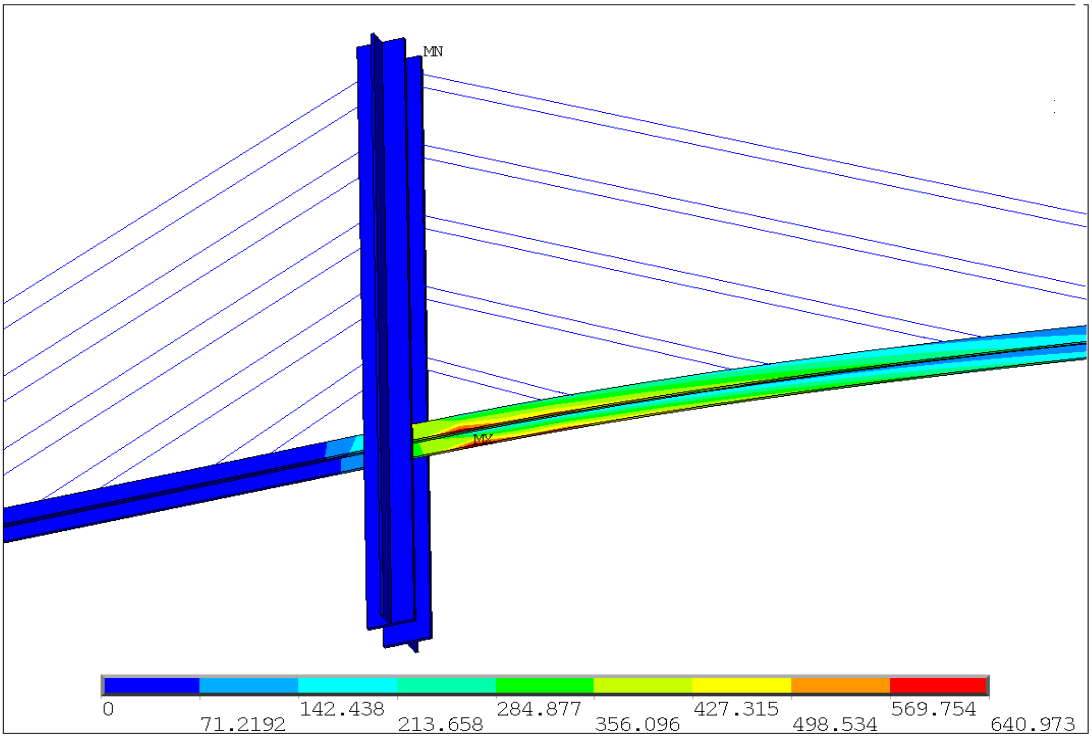

Figure 4.4 Maximum stress of numerical model Harp Type1

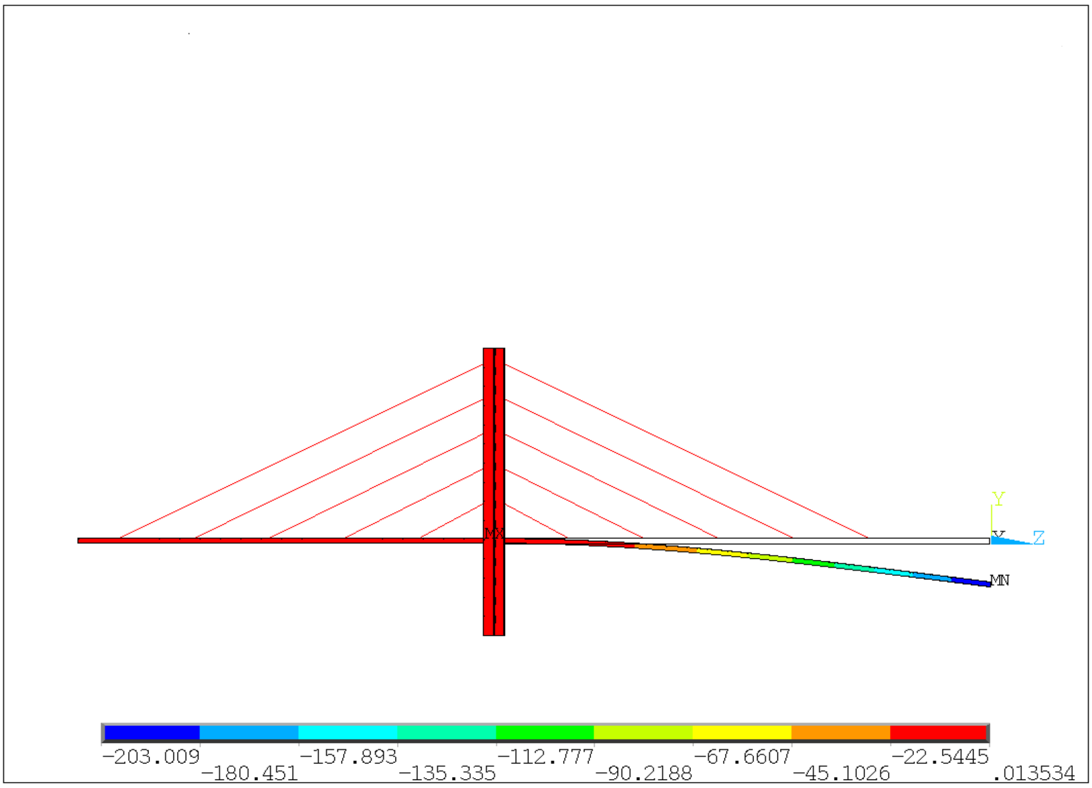

Figure 4.5 Deflection of numerical model Harp Type 2

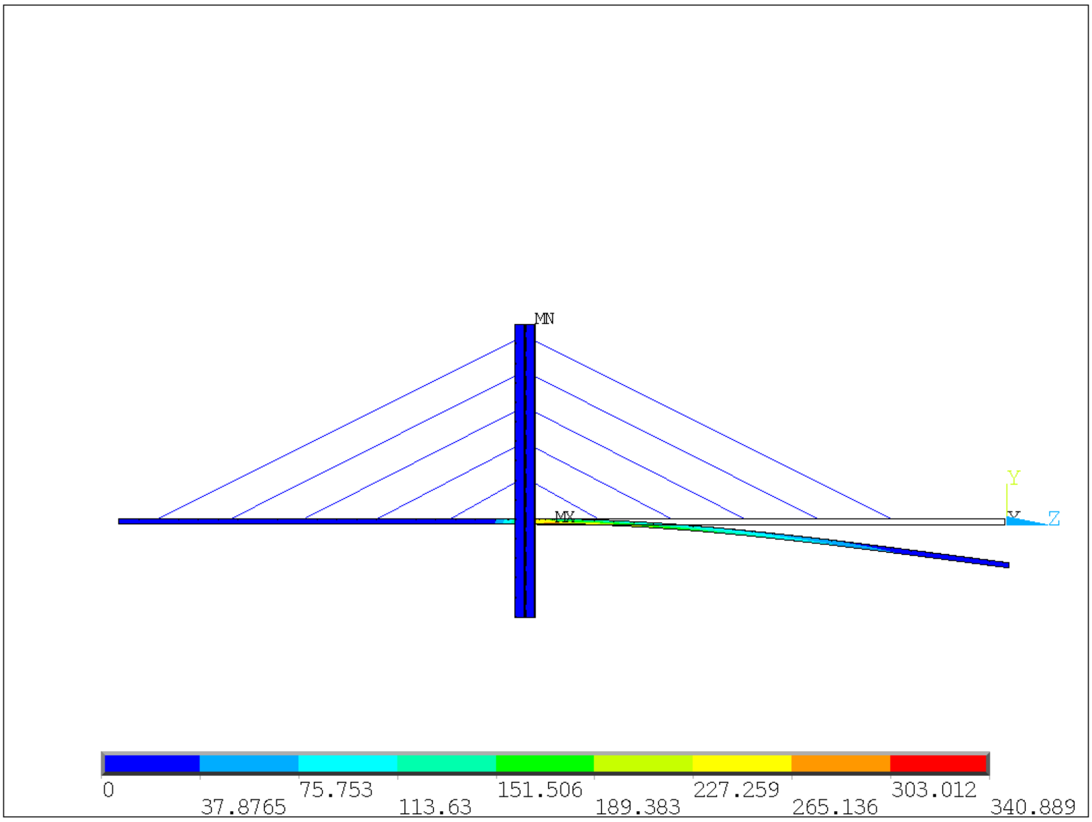

Figure 4.6 Stress of numerical model Harp Type 2

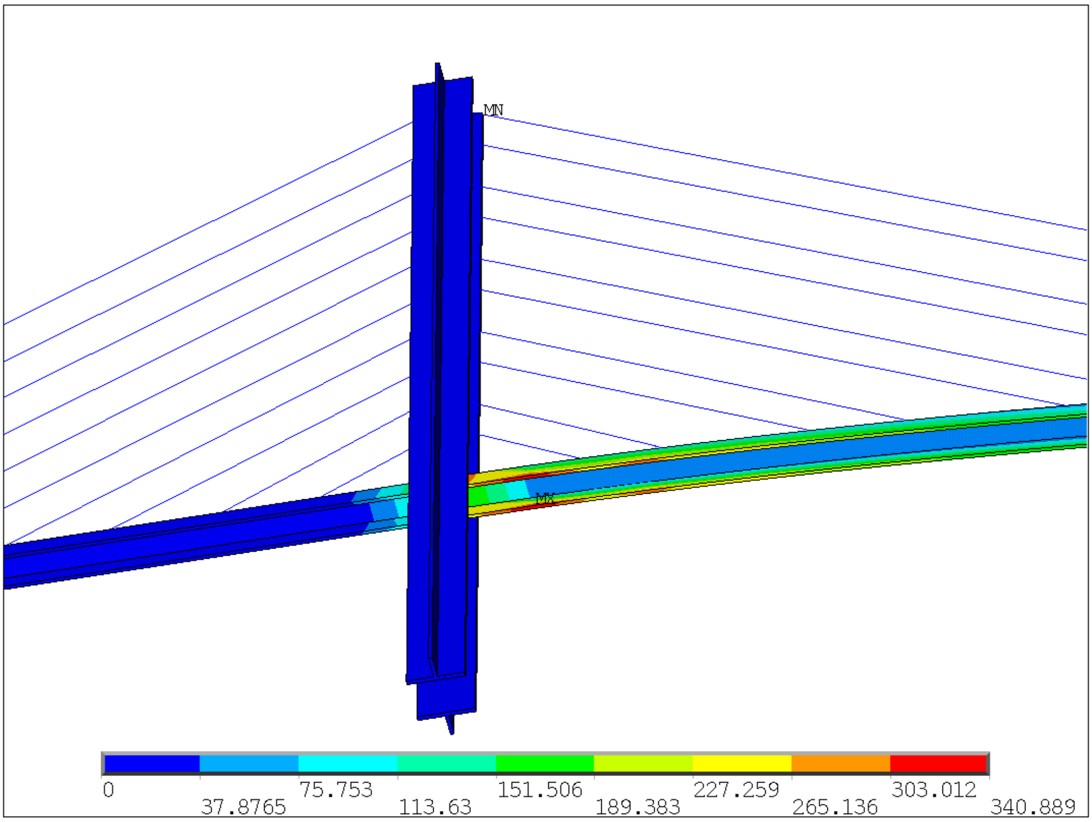

Figure 4.7 Maximum stress of numerical model Harp Type 2

4.7 Comparison between the Experimental and Analytical Results of the Harp Type1 Model

There are many mechanical and dimensional changes that dramatically alter the way cable-stayed bridges respond to external forces and influences, which in turn transform as concentrated or axial force or stresses on the towers or girders. Also, do not forget the force that affects the cables and limits the way they perform. The girder of the extradosed cable-stayed bridge not only resists the axial pressures, but also bears a considerable part of the moment and shear force. So its distribution of stress is complicated. There are many factors that affect the analytical values and their results that must be studied and taken into consideration to study the extent of their impact on cable-stayed bridges. So through the empirical data obtained from the experimental, whose models were represented, it was found that there is a good degree of similarity and closeness between the results obtained through the experiment and analysis of the cable-stayed bridge through the ANSYS program by comparing the graphs mentioned in The figure below implies a relation between the concentrated load and the displacement at the loading point in the mid- span.

The load-displacement mid-span when concentrated loading is applied is represented by the failure mode of the Harp type 1 model. The ultimate load is 5330 N, and a plastic hinge forms at the girder's attachment to the pylons. The stiffness begins to drop fast after the emergence of the plastic hinge, and this tendency is well supported by the analytical analysis results. We see that the maximum stress in the bridge girder fixed with cables, which we see through the results of the analysis obtained

from the program ANSYS that the effect is in under the flange of the beam, while the stress values decrease in the region of the mid-span, as shown in the Figure 4.8.

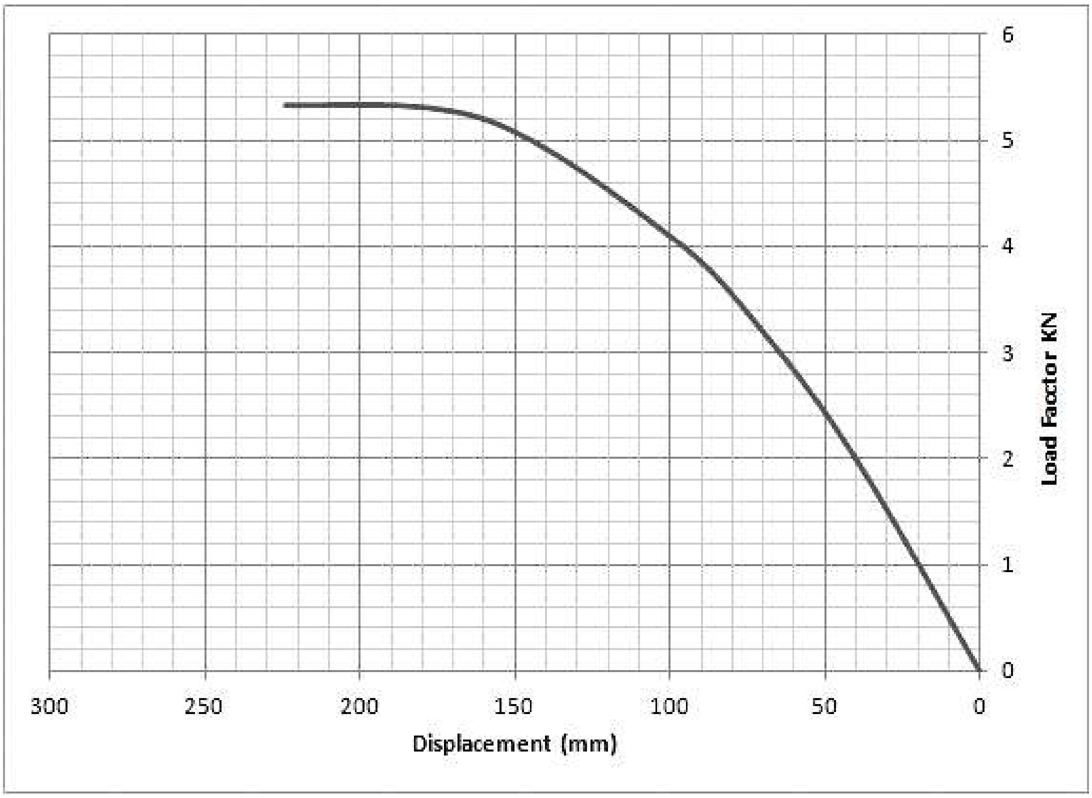

Figure 4.8 Numerical (load-displacement curve) at the loading point for model Harp Type1

4.8 Comparison between the Experimental and Analytical Results of the Harp Type2 Model

Also in the second model (harp type2) through the empirical data obtained from the experimental, whose models were represented, it was found that The degree of similarity of approximation is good between the results obtained through the experiment and analysis of the cable-stayed bridge through the ANSYS program by comparing the graphs mentioned in the Figure 4.9 represents the relationship between both the concentrated load with the displacement at the loading point in the mid- span similar to that of Harp type 2. The curve follows a pattern that is quite close to Harp type 1. The ultimate concentrated load is 2500N, indicating that the analytical results are in good accord. The experimental model's rigidity is lower than that of the analytical result. The discrepancy is thought to be due to problems with the manufacturing process. However, because the cable remained bridge is a highly uncertain construction, the discrepancy is within acceptable bounds, and the experimental result is deemed to be in excellent agreement with analysis. As in harp model 1 we see that the maximum stress in the bridge girder fixed with cables, which we see through the results of the analysis obtained from the program ANSYS that the effect is in under the flange of the beam, while the stress values decrease in the region of the mid-span, as seen in Figure 4.9.

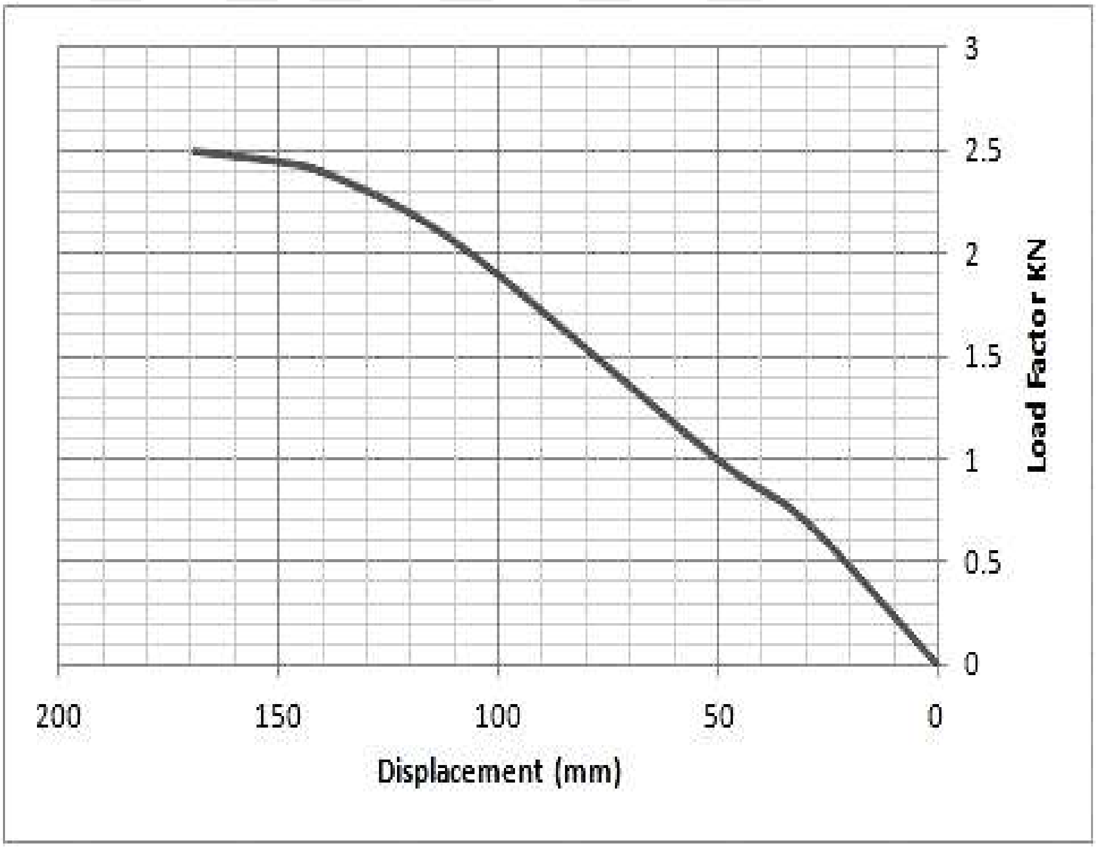

Figure 4.9 Numerical (load-displacement curve) at the loading point for model Harp Type2

4.9 Summary of This Chapter

In order to simulate the behavior of even a cable-stayed bridges, a computer software based on the finite element approach is utilized. Using widely accessible software, the bridges are modeled as three-dimensional structures. The parameters considered in the study include the depth of the girder's flange and the constrained load.

The goal of this study is to give a complete description and explanation of the behavior of cable-stayed bridges in two different types of models under various scenarios. Stresses in the case of the cable-stayed bridge are compared to stresses for the girder bridge based on a variety of alterations in the girder bridge's specifications. Finally, parametric analyses are carried out from the Cable-stayed bridge model in order to determine the effect of various critical factors on the girder's behavior. The findings of this study will help bridge engineers construct more reliable and cost-effective cable-stayed bridges.

CHAPTER V

CONCLUSION AND DISCUSSION

5.1 Conclusion

This thesis develops a fundamental grasp of cable-stayed bridge systems. The results of fundamental behavioral patterns of cable-stayed systems for static loads are presented in Chapter 4. Two types of cable-stayed bridge models are studied: harp type 1 and harp type 2. The bridge's performance is assessed using the two measures introduced in Chapter 4. The performance of a cable-stayed bridge with a long span is enhanced by adding additional depth to the flange of the girder, according to analytical results derived by the simulation program ANSYS, which employs the finite element approach in the study.

The installation of additional slanted cables is beneficial as the main span lengthens. In contrast, adding cables to a short span bridge does not greatly increase its performance. The highest stress is found near the junction of the tower and the girder, according to the findings. The ideal number of cables is determined by the S/M ratio of the side span to the main span. Furthermore, the study reveals that the towers' height is determined by the main span's length rather than the number of cables in the system. For the two models where the dimensions is 4900 mm for the main span and high of pylon above deck is 980 mm, the ratio (H/M) is 0.2 Therefore, this ratio is considered within the limits of the ideal region according to the code, Because, according to the ICE code, the ideal ratio of tower height to main span (H/M) for the two models is between 0.2 and 0.25, as detailed in Chapter 3, the optimal ratio of tower height to main span (H/M) for the proposed model is between 0.2 and 0.25.

The optimal H/M ratio does not change much as the number of cables increases. Because the cable angle is excessively severe when the tower is short, the bridge does not operate efficiently. Since the towers are much too flexible to sustain the span when they are tall, the bridge's performance suffers.

Also, in the case of the two models that were represented, the ratio of the back space to the main space is 0.418 which is considered less than 0.5 Therefore, the two models are considered within the limits of the specification set by the ICE code.
As engineers and even the general public get a better understanding of cable-stayed bridges, more of them will be built in the future. The current limit will almost certainly be pushed as the trend toward larger primary span continues. With cable-stayed bridges, the greatest possible main span is presently projected to be around 1,200m. Composite materials plus improved computer technology will undoubtedly enable softer systems to be built at a lower cost and in a shorter amount of time.
For cable-stayed bridges' static dejection analysis.

- As the numerical findings demonstrate, the large deflection influence is definitely the most essential and has a significant impact on the static deflection behavior of cable-stayed bridges. When the considerable deflection impact is disregarded in the deflection analysis, a noticeable deviation from the precise solution occurs, making convergent calculation difficult at times.
- Because high axial forces exist in all structural components and substantial deflections occur, a linear approach for the static displacement analysis in cable-stayed bridges is nonsensical and unsatisfactory.

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CIRRUCULM VITAE (CV)

Personal profile

Carrying out many training and motivational courses for human development, as well as participating in project management conferences, in particular engineering and urban projects, as well as contributing to guiding young people to develop their mental abilities for the sake of more creativity and self-development for the benefit of societies all over the world

Education

Tikrit University, Mosul, Nineveh, Iraq
Bachelor of Civil engineering, 2012

Experience

2018 Work as a site engineer with private company in Tikrit.

- Managing parts of construction projects
- overseeing building work
- undertaking surveys
- setting out sites and organizing facilities
- checking technical designs and drawings to ensure that they are followed correctly
- supervising contracted staff
- providing technical advice and solving problems on site

2017 Worked as a Mathematics teacher at School Imam Muslim – Mosul

- Planning, preparing and delivering lessons to all students in the class.

- Teaching according to the educational needs, abilities and achievement of the individual students and groups of students.
- Adopting and working towards the implementation of the school development plan of the particular school they are giving service

2016 Worked as a team leader of CHW with IMC in Tikrit-Alalam

- To ensure that a socially inclusive approach is delivered by the team that is recovery and enablement focused
- To provide effective and dynamic team leadership that will facilitate staff in ensuring that service users are enabled to live as safely and independently as possible with the best quality of life they can achieve.
- Overall responsibility for the team's casework - ensuring delivery of care services within professional and policy standards and within a climate of empowerment and inclusion.

Skills

- AutoCAD
- Proficient in [Excel, Word, Power point]
- Independent worker
- Team leader
- Supervisor engineer
- Mentoring

Training

- I have got training about infection diseases, first aid and privacy policy of organizations
- I got all of them from IMC organization in Baghdad.

Language

English language is strong as Arabic language

I have a course of English language at Cambridge institute for 3 months.