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COLLEGE OF GRADUATE STUDIES

Evaluation of the Optimum Pre-tensioning Forces and Construction Stage Analysis of Cable - Stayed Bridges

تقييم قوي الشد المسبق المثلي والتحليل لمراحل التشييد للجسور المدعومة بالكوابل

Thesis submitted to The School of Civil Engineering in Fulfillment of the requirements of the Degree of Doctor of Philosophy in Civil Engineering

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Abstract

This research is concerned with the study of the problems of the analysis and design of Cable-stayed bridges which are structural systems effectively composed of cables, main girders and towers. These types of bridge are highly indeterminate structures that require a highly complex degree of technology for analysis and design. Hence they demand sophisticated structural analysis and design techniques when compared with other types of conventional bridges. For cable-stayed bridges the cable forces are an important factor in the design and construction process. The cables being flexible supports require pre-tensioning. Since the response of the bridge is highly non-linear an optimization procedure is required to evaluate the pre-tensioning forces. The optimization method used to determine the cable forces is the unknown load factor method; The techniques and methods of erecting cable –stayed bridges are varied and numerous. The erection method not only affects the stresses in the structure during erection but may also have an effect on the final stresses of the complete structure. The required pretension forces in cable-stays and the corresponding structural configurations of the bridge at different erection stages have been examined and compared in detail. The objective of the construction stage simulation is to identify stresses and deformations of the concrete girder and towers, as well as the cable tension stress, to meet the design requirements. Because of the large deflections that occur in bridge during cantilever construction, construction stage analysis was performed for the cable stayed bridge. The finite element analysis program Midas Civil was used in the nonlinear analysis. In this study, TUTI BAHARI cable stayed bridge, under consideration for construction was evaluated, as a case study, determining the cable tension under the effect of Dead load (Self weight, additional loads), Initial pre- tensioning cable force, and live load (moving load) according to AASHTO

LRFD 2010, by considering the boundary conditions. Then, analysis of this bridge at different erection stages during construction was carried out using the backward construction process analysis. The stage by stage construction of the TUTI BAHARI cable stayed bridge was performed using Midas civil Software. The maximum cable forces were found to be within the allowable limits. Also, the stresses and displacements satisfied the requirements. The results obtained show that the method presented indeed leads to optimal structural performance for the cable stayed bridge in particular, and might be a useful reference for the design of other similar bridges.

مستخلص الدِرَاسة

هذا البحث يختص بدراسة مشاكل تحليل وتصميم الجسور المدعومة بالكوابل والتي يتألف نظامها الإنشائي من الكوابل والعارضة الرئيسية والابراج. هذا النوع من الجسور يتطلب درجة عالية من التكنولوجيا في التحليل و التصميم وايضاً التشييد وبالتالي تتطلب تقنيات تحليل وتصميم هيكليه متطورة عند مقارنتها بانواع اخري من الجسور التقليدية. وتعتبر قوة الكوابل عاملاً مهماً واساسياً في العملية التصميمية للجسور المدعومة بالكوابل. ولأن الكوابل سواند مرنة فإنها تتطلب الشد المسبق. وبما أن للجسر درجة عالية من اللاخطية تستخدم طريقة التحسين للحصول على قوى الشد المسبق، وطريقة التحسين لمعامل الحمل المجهول هي التي استخدمت للحصول على القوى فى الكوابل. هنالك العديد من التقنيات والطرق المستخدمة في تشييد الجسور المدعومة بالكوابل، ولا تؤثر طرق التشييد هذه علي الاجهادات التي يتعرض لها المنشأ اثناء فترة التشييد فحسب بل قد يكون لها ايضا تاثير علي الاجهادات في المرحله النهائيه للتشييد. بما أن الهدف من محاكاة مراحل التشييد هو التعرف علي الاجهادات والتشوهات للعارضة الخرسانية والابراج المكونه للجسر وايضا اجهادات الشد للكوابل ونسبة لحدوث انحراف كبير للجسر اثناء التشييد فقد تم عمل تحليل للجسر في هذه المراحل من التشييد. مع الاختبار والمقارنة التفصيليه لقوي الشد المسبق المطلوبه في كوابل الشد والتكوينات الانشائية المقابلة في كل مرحلة

في هذه الدراسه تم استخدام برنامج تحليل العناصر المحددة Midas Civil في التحليل اللاخطي لتقييم قوة الشد المثلي في الكوابل للجسر ثم التأكد من استقرار الجسر خلال مراحل التشييد المختلفه معتمدا في التحليل باعتبار مراحل التشييد من المرحلة النهائيه الي الاولي. تم التطبيق لجسر توتي – بحري الرابط بين جزيرة توتي ومدينة بحري كدراسة حالة . تم إيجاد قوى الشد المثلى لانواع مختلفه من كوابل الجسر التي

تحقق الحدود المثالية للاستقرار مع الأخذ في الاعتبار الحالات الحدودية والتغيير في التحميل وكذالك إجراء التحليل تحت تاثير الاحمال الحية واحمال الرياح للجسر وفقا للمدونه الامريكيه AASHTO LRFD2010 ووجد من خلال التحليل أن اقصي قوة للكوابل تقع في الحدود المسموح بها وكذالك قيم الاجهادات والازاحات اظهرت النتائج التي تم الحصول عليها ان الطريقة المقدمة تؤدي بالفعل الي الأداء الهيكلي الأمثل للجسر المدعوم بالكوابل علي وجه الخصوص، ويمكن أن تعتبر مرجعاً مناسباً لتصميم الجسور المماثلة الاخري.

Published papers:

Two scientific papers from the research content have been published as follows:

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- *p: Increment of load*
- *A: Gross metallic area*
- *e: Elongation caused by load increment*
- *l: Gauge length*
- *E: Modulus of elasticity of straight cable.*
- *l:horizontal length of the cable.*
- *γ:specific weigth of the cable.*
- *σ:tensile stress in the cable.*
- $[M]$*: The mass matrix.*
- $\{D\}$*: The displacement vector (u, υ and θ at each joint)*
- $[C]$*: The damping matrix.*
- $\{\dot{D}\}$*: The velocities at the nodes.*
- $[K]$*: The matrix formed from member stiffness matrices [K] evaluated at the deformed position of the member.*
- $\{\ddot{D}\}$: *The vector containing the accelerations* $\ddot{\theta}_x$, $\ddot{\theta}_y$,.$\ddot{\theta}_z$$\{P\}$*: The force matrix, made up of concentrated forces and couples at the nodes.*
- V_{DZ} *: design wind velocity at design elevation, Z (mph).*
- V_{10} *: wind velocity at 10m above low ground or above design water level (mph).*
- V_B *: base wind velocity of 100 mph at 10m height, yielding design pressures specified in AASHTO Articles 3.8.1.2 and 3.8.2*

- *Z : height of structure at which wind loads are being calculated as measured from low ground, or from water level, > 10m.*
- *V_0 : friction velocity, a meteorological wind characteristic taken, as specified in AASHTTO Table 3.8.1.1-1, for various upwind surface characteristics (mph)*

- *P_B : base wind pressure specified in AASHTO Table 3.8.1.2.1-1*
- *ε_{11}: is a coefficient somewhat greater than 1*
- *W:weight per unit length*
- *T:tension force*
- *{U}: Displacement vector*
- *{P}: Load vector*

CHAPTER ONE

Introduction

1.1 General introduction

Cable–stayed bridges have been widely applied, especially in Western Europe, Canada, South America; Japan, Sweden and the united states. Cable-stayed bridge obtained more popularity for long-span bridges because the design of this bridge is adjudged by the financial, practical, and technical requirements, also to a great extent, by aesthetical appearance and architectural considerations. A cable-stayed bridge is a more economical solution for spans up to about 1000 m. this bridge form has a fine-looking appearance and fits in with most surrounding environments.

The main structural elements of a cable stayed bridge are an orthotropic deck, continuous girders, piers, abutments, towers and the stays. In a cable –stayed bridge the girders are supported at several locations, namely, abutments and piers, usually considered as fixed and non-yielding supports and at cable points with the cables emanating from the towers. The latter are flexible supports as the cables change length under load and because the towers are also flexible and can move. The structure can therefore be modeled as a continuous beam on both rigid and flexible supports. The tower, girder and cable members are under dominantly axial forces, with the cables under tension.

The arrangement of the cables in the longitudinal direction of the bridge could be divided into four basic systems, namely, fan system, harp system, radiating system and the star system. The selection of cable configuration and number of cables is dependent on the length of span, type of loading, number or roadway lanes, height of towers, and economy.

For cable stayed bridges the cable forces are an important factor in the design process. The height of the tower frequently affects the stiffness of the bridge system. As the angle of inclination of cable with respect to the stiffening girder increases, the stresses in the cables decrease, as does the required cross section of the tower. However, as the height of the tower increases, the length of the cables, also, the axial deformations increase.

The engineer must consider all the loads that are predicted to be applied to the bridge during its service life. The loads may be divided into two broad categories: permanent loads and transient loads. Also, the lateral loads such as those due to water and wind, ship collisions, and earthquakes.Depending on the structure type, other loads such as those from creep and shrinkage may be important. Each type of load is presented individually with the appropriate reference for example the AASHTO specification.

The erection procedure depends on the structure system of the bridge, the site conditions, and dimensions of the shop-fabricated bridge units, equipment and other characteristic of the particular bridge. The methods of erection for cable - stayed bridges are broadly described by three general methods: the staging method, the push – out method, and the cantilever method.

1.2: Research Problem Statement

- The non-linear structural behavior of cable stayed bridges is not yet established. Structural stability of the whole bridge under gravity and wind loads presents increasingly important problems both in design and construction. These problems result from the high compressive force in the towers and deck as well as the large forces on the flexible bridge.

- The cable and the pre-tension required are still under investigation. Most of the methods proposed in literature are either inadequate or prone to have draw backs.
- The large deflections occur in bridge during construction, it's necessary to perform construction stage analysis for the cable stayed bridge. Also; the construction method not only affects the stresses in the structure during construction but may also have an effect on the final stresses of the complete structure.
- Thus, this research is an attempt to carry out an optimization procedure of cable pretension under static and dynamic loading and to carry out a construction stage analysis.
- Based on the outcomes of the above there will be an attempt to developing, implementing and validation of a non-linear procedure for the structural behavior of the bridge.

1.3: Research Hypothesis and Questions

- The flexibility of the cable stays results in a highly geometrically non -linear behavior of the cable stayed bridge.
- Non –linear Finite Element analysis poses a suitable mean of correctly evaluating the non-linear behavior of cable stayed bridges.
- Is there a scientifically viable method to determine the optimum pre-tensioning other than the trial and error methods usually used?
- Subjecting the bridge components to different and variable types of loading during the construction stages can adversely affect the strength and stability.

- Is it always necessary to carry out a construction stage analysis of cable stayed bridges?
- Is there a possibility of developing implementing and validation of a non-linear procedure for the structural behavior of cable stayed bridges?

1.4: Objectives of the Research

The general objectives of this study are:

- To study the effect of gravity and wind loads on the analysis and design of cable-stayed bridge.
- To study the nonlinear behavior of cable-stayed bridge using non-linear Finite Element analysis.
- To evaluate the influence of geometrical, mechanical parameters of different types of cable on the behavior of bridge structures.

The main objectives of this study are:

- Using different type of cable (strand cable – new parallel wire strand and carbon fiber cable) in the analysis of the model of cable stayed bridge.
- To determine the optimum pre-tensioning cable forces for cable-stayed bridges.
- To study the effect of the different Construction stages of the cable stayed bridge on its strength and stability.
- To check the results obtained by using Midas civil programs and determine the factors affecting the analysis and design of cable-stayed bridges.
- To develop, implement and apply a geometrically non-linear Finite Element model for the analysis of cable stayed bridges.

1.5: Methodology of Study

- Collecting relevant data and information through various sources including books, journals and different references.

- Carrying out state of the art literature review to point out the problems in analysis and design of cable stayed bridges.
- Developing the necessary theoretical relations for a geometrically non-linear Finite Element analysis of cable stayed bridge
- Modeling and simulation of Tuti- Bahri cable stayed bridge as a case study using nonlinear Finite Element packages for optimization of pre-tension and construction stage analysis.
- Developing, a geometrically non-linear Finite Element program and applying program for analysis of cable stayed bridge.
- Validation of the results of applications of the programs, by carrying out the parametric study and analysis and discussing the results.
- Drawing conclusions and proposing recommendations.
- Finalization of the research and writing up.

1.6: Research organization

- **Chapter one:** General introduction including introductory remarks briefly introduces the background to the research and its definition research hypothesis and question, Research Problem Statement, Objectives of the Research, Methodology of Study and thesis out lines
- **Chapter two:** literature review (collecting relevant data and information through various sources including books, journals and different references), type of cable stayed bridge, method of analysis and design of cable stayed bridge.

- **Chapter three:** Modeling and simulation of cable stayed bridges using nonlinear finite element packages (Midas civil programs). The developed geometrically non-linear finite element theory.
- **Chapter four:** analysis and design stages nonlinear analysis results and discussion.
- **Chapter five:** construction stages analysis results and discussion
- **Chapter six:** conclusions and recommendations.
- References
- Appendices

CHAPTER TWO

Literature Review

2.1 Introduction

A bridge is a structure that crosses over a river, bay, or other obstruction, permitting the smooth and safe passage of vehicles, trains, and pedestrians. A bridge structure is divided into an upper part (the superstructure), which consists of the slab, the floor system, and the main truss or girders, and a lower part (the substructure) which are columns, piers, towers, footings, piles, and abutments. The superstructure provides horizontal spans such as deck and girders and carries traffic loads directly. The substructure supports the horizontal spans, elevating above the ground surface.

As stated by Barker and Puckett, (2007), one of the key submittals in the design process is the engineer’s report to the bridge owner of the Type, Size, and Location (TS & L) of the proposed bridge. The TS & L report includes a cost study and a set of preliminary bridge drawings. The design engineer has the main responsibility for the report, but opinions and advice will be sought from others within and without the design office.

Selection of a bridge type involves consideration of a number of factors. In general, these factors are related to function, economy, safety, construction experience, traffic control, soil conditions, seismicity, and aesthetics.

2.2 Cable-stayed bridges

According to Chen and Duan, (2000), since the completion of the Stromsund Bridge in Sweden in 1955, the cable-stayed bridge has evolved into the most popular bridge type for long-span bridges. For spans up to about 1000 m, cable-stayed Bridges are more economical.

The concept of a cable-stayed bridge is simple. A bridge carries mainly vertical loads acting on the girder, Figure (2-1). The stay cables provide intermediate supports for the girder so that it can span a long distance. The basic structural form of a cable-stayed bridge is a series of overlapping triangles comprising the pylon, or the tower, the cables, and the girder. All these members are under predominantly axial forces, with the cables under tension and both the pylon and the girder under compression.

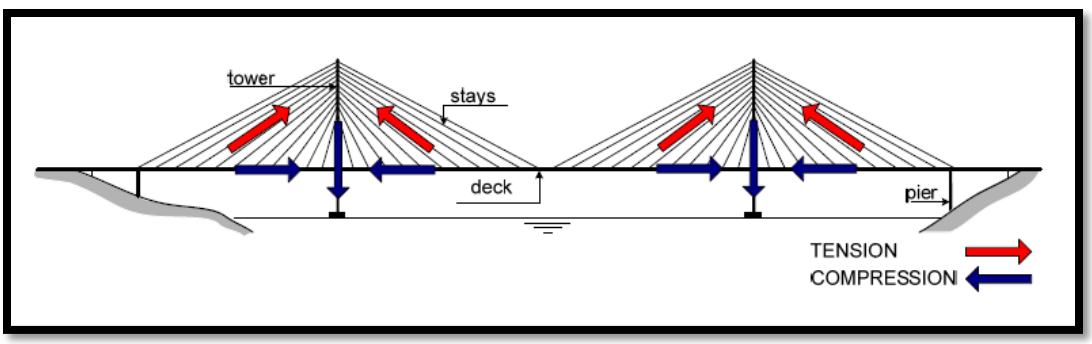


Figure (2-1): Behavior of a cable stayed bridge, Carlos Miguel ,(2011).

2.2.1 Cable arrangements

Podolny and B.Scalzi, (1986), had shown that the Cable-stayed systems are classified according to the different longitudinal and transverse cable arrangements. Cable layout is fundamental issue that concerns cable stayed bridges. It not only affects the structural performance of the bridge, but also the method of erection and the economics .the various cable arrangements as shown in the following:

1. **Radial or converging system**

As stated by Troitsky, (1988), in this system all cables are leading to the top of the tower or, as Podolny and B.Scalzi clamis, intersect common point, Figure (2-2). Structurally, this arrangement is perhaps the best, as by taking all cables to the tower top the maximum inclination to the horizontal is achieved and consequently it needs the smallest amount of steel. The cables carry the maximum component of the dead and live load forces, and the axial component of the deck structure is at a minimum.

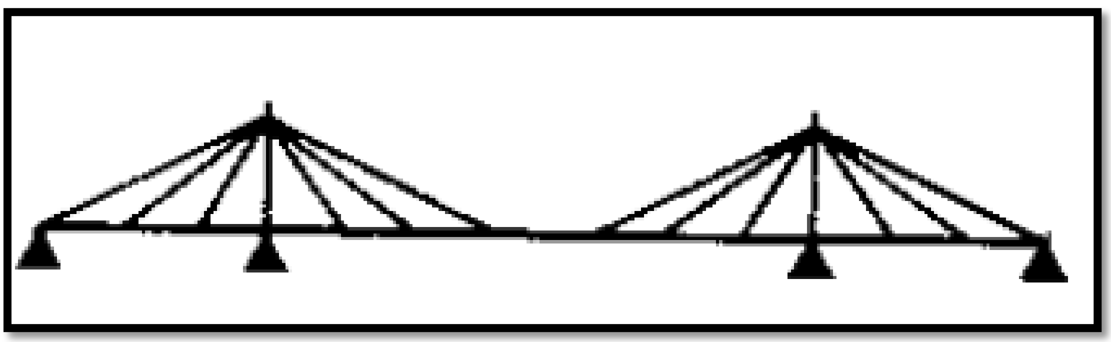


Figure (2-2): Radial system, Podolny and BScalzi, (1986)

2. **Harp or parallel system**

As stated by Gimsing and Georgakis, (2012), in the harp system, the number of cables leading to the main span will have to be the same as in the side spans, Figure (2-3). With the anchor pier positioned at the end of the side span harp, the length of the side span will be very close to half of the main span Length .while Troitsky, (1988), noted that this system of cable are connected to the tower at different heights, and placed parallel to each other.

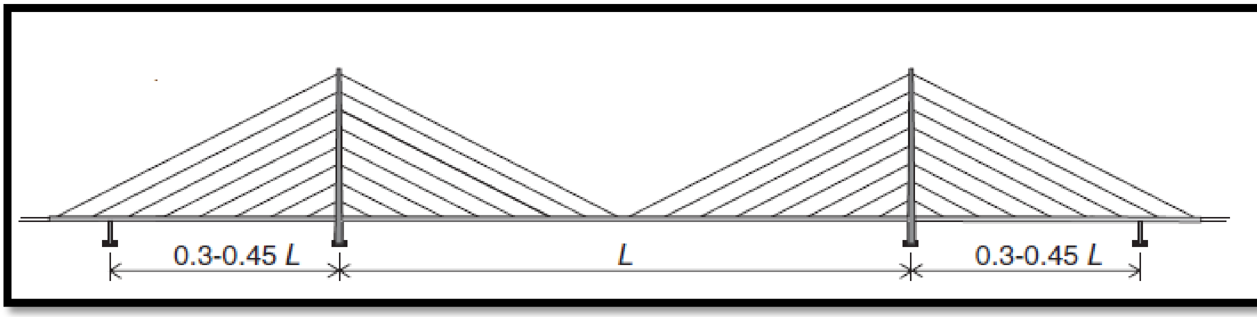


Figure (2-3): Harp or parallel system, Gimsing and Georgakis, (2012)

3. **Fan or intermediate system**

Gimsing and Georgakis, (2012), claimed that the fan system has become the favorite's cable system due to its efficiency and the degree of freedom regarding geometrical adaption. The fan system is most commonly applied in the form of a semi-fan system where the cable anchor points are spread over a certain height at the pylon top to give room for an individual anchorage of each stay cable.

the fan or intermediate stay cable arrangement represents a modification of the harp system. The forces of the stays remain small so that single ropes could be used. All ropes have fixed connections in the tower. As shown in Figure (2-4)

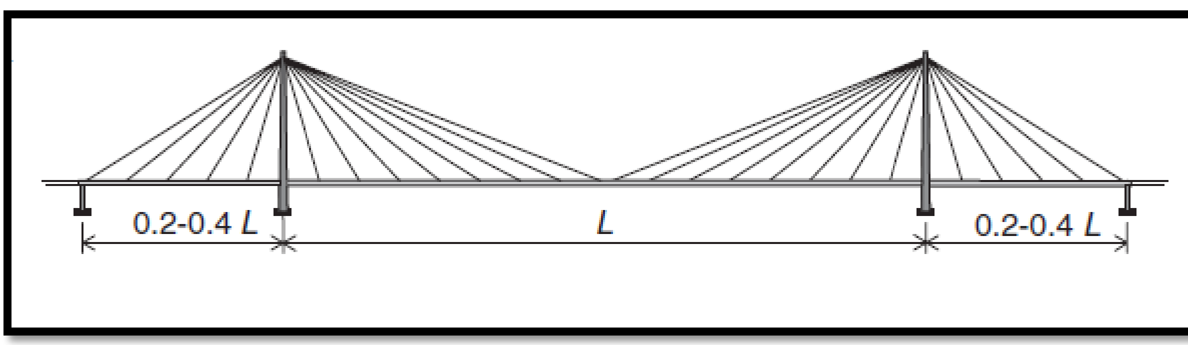


Figure (2-4): Fan system, Gimsing and Georgakis, (2012)

4. **Star system**

As stated by Podolny and B.Scalzi, (1986), In the star arrangement, the cables intersect the tower at different heights and then converge on each side of the tower to intersect the roadway structure at a common point .Figure (2-5).

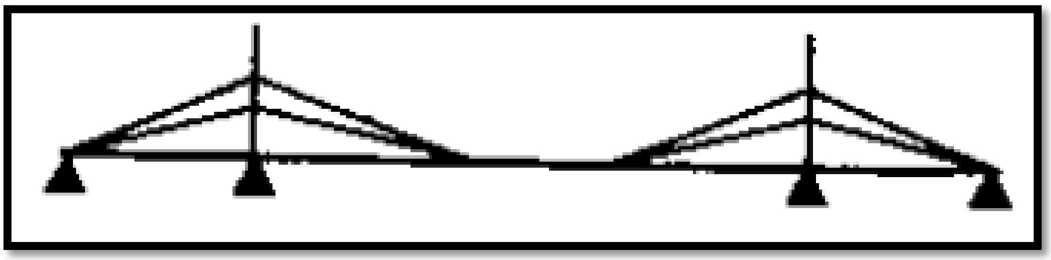


Figure (2-5): star system, Podolny and B.Scalzi, (1986).

2.2.2 Components of cable stayed bridge

The main structural elements of a cable stayed bridges are the bridge deck, piers, towers and the stays. The deck supports the loads and transfers them to the stays and to the piers through bending and compression. The stays transfer the forces to the towers, which transmit them by compression to the foundations

1. Tower types

Chen and Duan, (2000), claimed that, it was found that the Towers can be defined as vertical steel or concrete structures projecting above the deck, supporting cables and carrying the forces to which the bridge is subjected to the ground. By this definition, towers are used only for suspension bridges or for cable-stayed bridges, or hybrid suspension–cable stayed structures. The word pylon is sometimes used for the towers of cable–stayed bridges. Both pylon and tower have about the same meaning— a tall and narrow structure supporting itself and the roadway.

The main structural function of the towers of cable-stayed and suspension bridges is carrying the weight of the bridge, traffic loads, and the forces of nature to the foundations. The towers must perform these functions in a reliable, serviceable, aesthetic, and economical manner for the life of the bridge, as towers, unlike other bridge components, cannot be replaced. Towers of cable-stayed bridges can have a wide variety of shapes and forms. For conceptual design, the height of cable-stayed towers above the deck can be assumed to be about 20% of the main span length. To this value must be added the structural depth of the girder and the clearance to the foundation for determining the approximate total tower height. The final height of the towers will be determined during the final design phase.

According to Troitsky, (1988), the various possible types of tower construction are illustrated in Figure (2-6), which shows that they may take the form of:

1. Trapezoidal portal forms.
2. Twin towers.
3. A- Frames.
4. Single towers.

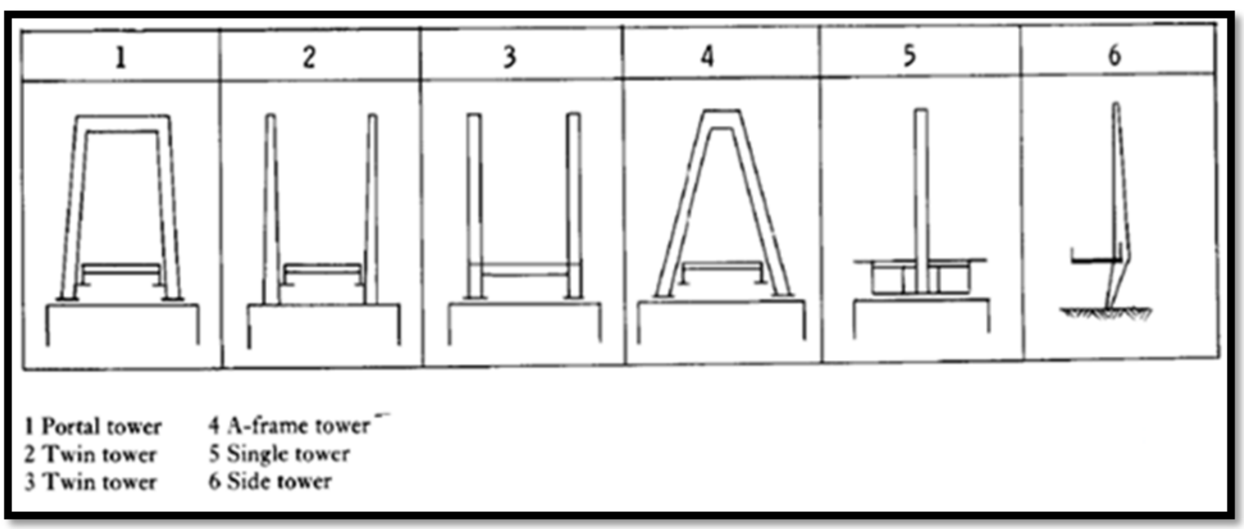


Figure (2-6): Tower types, Troitsky, (1988).

- There are three different solutions possible regarding the support arrangement of the tower as following :

1. Tower fixed at the foundation.
2. Towers fixed at the superstructure.
3. Hinged towers.

2. Deck type

Troitsky, (1988), mentioned that most cable stayed bridges have orthotropic decks which differ from one another only as far as the cross- sections of the longitudinal ribs and the spacing of the cross – girder is concerned. Typical ribs used in an orthotropic deck are shown in Figure (2-7). Cross-girders are usually 1.8 to 2.5 m apart for decks stiffened by flexible ribs, and 4.6 - 5.5m apart in the case of decks stiffened by box –type ribs possessing a high degree of torsional rigidity.

The orthotropic deck performs as the top chord of the main girders or trusses. It may be considered as one of the main structural elements which lead to the successful development of modern cable stayed bridges.

For relatively small spans in the 60-90m range it is convenient to use a reinforced concrete deck acting monolithically with the main reinforced or pre-stressed concrete girders.

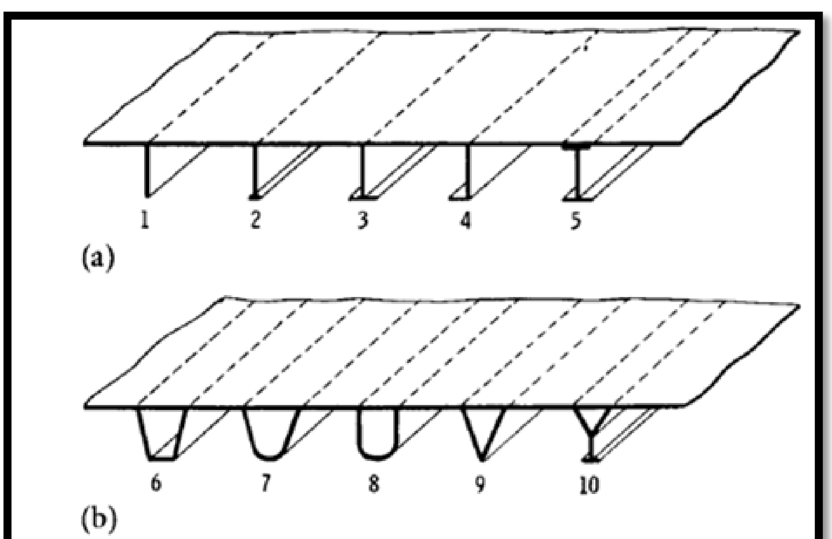


Figure (2-7): Rib types (a)Torsionally weak or open type (b)Torsionally stiff or box type, Troitsky ,(1988).

3. Main girders and trusses

As stated by Troitsky (1988), the following three basic types of main girders or trusses are used for cable - stayed bridges:

- **Steel girders**

Bridge built with solid web main girders may be divided into two types: those constructed with I –girders and those with one or more enclosed box section, as shown in Figure (2-8).

Plated I- girders with a built- up bottom flange comprising a number of cover plates have been used in some bridges .It is considered that in this way, the required inertia of the section can be made to fit the moment envelope exactly, that no excess steel is being used, and thus the minimum weight of steel is attained. It is felt, however, that this arrangement does not necessarily produce the most economical solution.

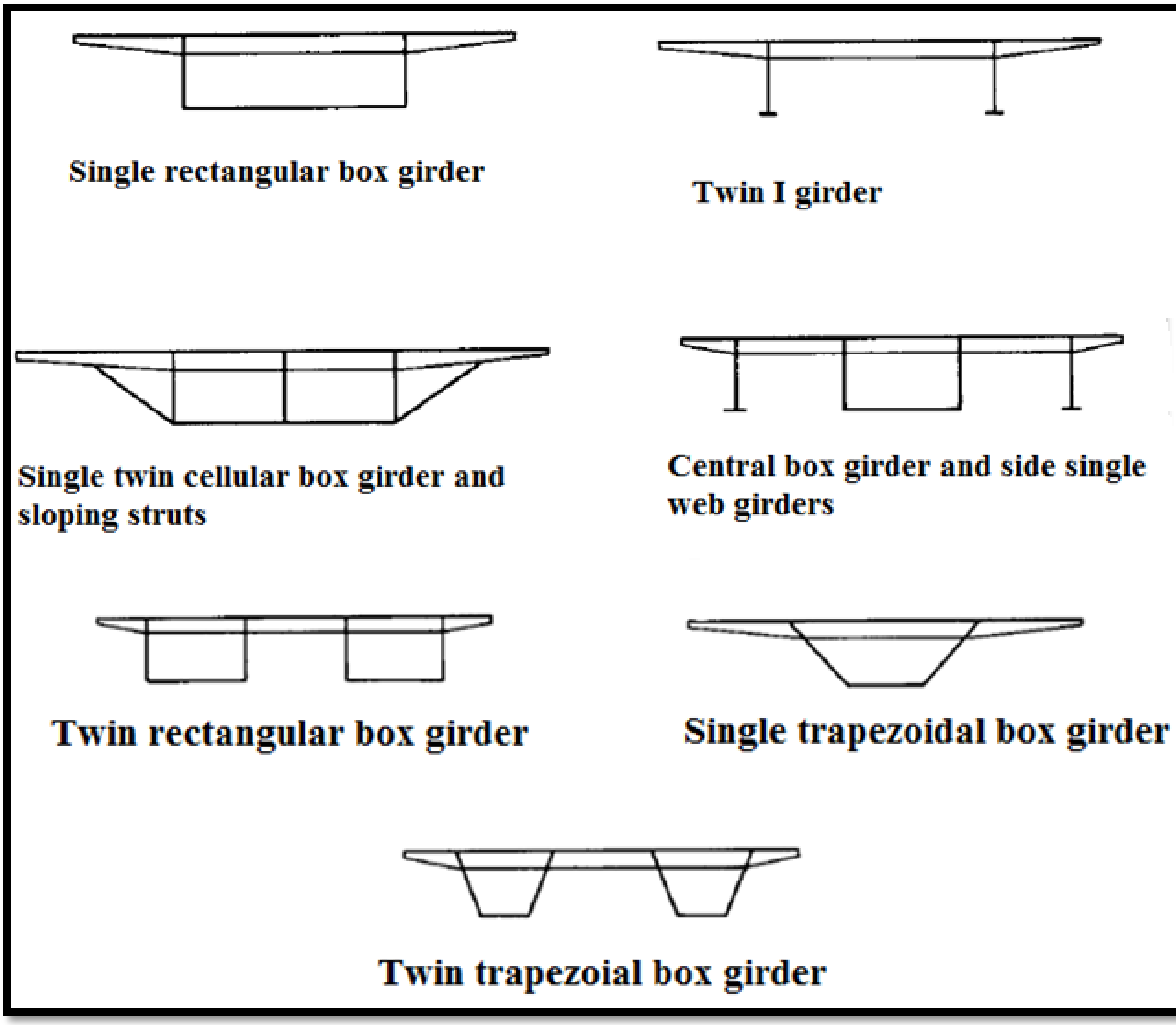


Figure (2-8): Types of main girder,Troitsky ,(1988).

Box girders in comparison often have portions of their span where a certain minimum plate thickness has to be maintained to prevent local buckling and to provide protection from corrosion, even though the desired inertia does not require such thickness. They do, however, have the great advantage of simplicity of fabrication in comparison to plate I- girders, and most important, a standard section with only the plate thickness varying can be produced in series, which significantly reduces fabrication costs. Also, the inside surfaces are not exposed to the atmosphere, and thus initial protective treatment and later maintenance costs are reduced.

Box girders may be rectangular or trapezoidal in form, i.e. with web plates vertical or sloping. The trapezoidal section is often used in order to keep the bottom flange area to the desired size, whilst the support to the deck plate from the webs is provided at an optimum position.

- **Trusses**

During the last decade trusses have rarely been used in the construction of cable-stayed bridges. Compared to solid web girders, trusses present an unfavorable visual appearance; they require a great deal of fabrication and maintenance, and protection against corrosion is difficult. Thus, except in special circumstances, a solid web girder is sons. More satisfactory both from an economical and an aesthetic viewpoint. However, trusses may be used instead of girders for aero dynamical reasons. In the case of combined highway and railroad traffic, when usually double deck structures are used, trusses should be provided as the main carrying members of such bridges.

- **Reinforced or pre-stressed concrete girders**

During the last decade a number of cable – stayed bridges have been built with a reinforced or pre stressed concrete deck and main girder. These bridges are economical, possess high stiffness and exhibit relatively small deflection. The damping effect of these monolithic structures is very high and vibrations are relatively small.

4. Cables

Chen and Duan, (2000), claimed that Cables are the most important elements of a cable-stayed bridge. They carry the load of the girder and transfer it to the tower and the back-stay cable anchorage

As highlighted by Gimsing and Georgakis, (2012), the basic element for all cables to be found in modern cable supported bridges is the steel wire characterized by a considerably larger tensile strength than that of ordinary structural steel.
In most cases, the steel wire is of cylindrical shape wires with diameters up to 7mm are used for parallel wire strands in cable stayed bridges.

- **Basic Types of Cables**

According to Gimsing and Georgakis, (2012), the basic types of cable using in cable –stayed bridge as shows follows and presented in Table (2-1).

1. Locked-coil strands

Locked-coil strands for bridges are fabricated with diameters in the range from 40 mm to 180 mm, with an outer diameter of 150 mm has a metallic cross section of 15900 mm^2 corresponding to a void ratio of only 0.10. The wires used for locked-coil strands generally have tensile strengths of 1370–1570 MPa, the equivalent density γ_{eq}(defined as the weight per unit length divided by the steel cross section) is approximately 88 kN/m^3. a nominal modulus of elasticity typically 180 GPa.
2. Parallel-wire stays cables
Parallel-wire stay cable with an outer diameter of 200mm had a metallic cross section of maximum 14584mm^2 corresponding to a void ratio of 0.54

3. New parallel-wire strand PWS

The equivalent density (ɤeq) which typically has a value around 82 kN/m^3. New parallel-wire strand PWS Cables are fabricated in sizes ranging from 7 Nos. 7mm wires to 421 Nos. 7mm wires

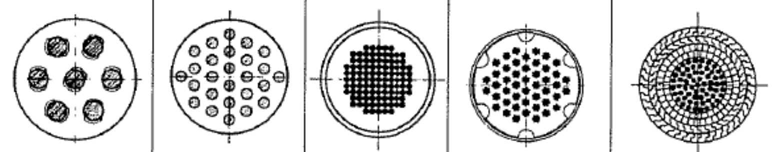
4. Carbon fiber cables

According to Walther,(1999),several companies have developed cables consisting of carbon fiber (CFRP wires) which, due to their outstanding technological properties, are in principal ideally suited for cable stayed structures.

The potential advantages of CFRP wire are as follows:

- non-corrosive material
- very high tensile strength of about 3000 N/mm^2
- high fatigue strength
- low density (1.6 g/cm^3) which is about one –fifth that of steel
- high resistance to chemical attack
- the modulus of elasticity of about 160 kN/mm^2 is some what lower than that of steel (~ 210 kN/mm^2)

Table (2-1): Type of cable Walther,(1999).

Type of cable	Coupled bars 7 ∅ 36 Steel 835/1030	Uncoupled bars 26 ∅ 16	Parallel wires 128 ∅ 7	Strands 27 ∅ 15	Locked-coil cables
					
				15 mm	
Tendons	Bars ∅ 26·5, 32, 36 mm	Bars ∅ 16 mm	Wires ∅ 6, 7 mm	Strands ∅ 0·5, 0·6, 0·7 in of 7 twisted wires	Wires with different profiles ∅ 2·9–7 mm
0·2% proof stress, $\sigma_{0.2}$ (N/mm^2)	835 1080	1350	1470	1570 ~ 1670	—
Ultimate tensile strength, β_z (N/mm^2)	1030 1230	1500	1670	1770 ~ 1870	1000 ~ 1300
Fatigue*					
$\Delta\sigma$ (N/mm^2)	80	—	350	300 ~ 320	120 ~ 150
σ_{max}/β_z	0·60	—	0·45	0·5 ~ 0·45	~0·45
Modulus of elasticity, E (N/mm^2)	210 000	210 000	205 000	190 000 ~ 200 000	160 000 ~ 165 000
Failure load (kN)	7339	7624	7487	7634	7310

2.3 Cable Anchorage and connection

As stated by Gimsing and Georgakis,(2012),in cable supported bridges the structurral connections between elements of the deck and the pylon ,as well as connections between the superstructure and the substructure , can be desigened by principles generally known from other types of structure. only when it comes to the structural connections where the elements of the cable system are attached to the deck, the pylon, and the substructure do special details is extermely important, as the cables constitute the main load-caring elements of the structure.the type of connections as shows follows:

1. Cable supports on the towers

Troitsky, (1988),mentioned that Cable supports on the towers may be either fixed or movable or a combination of both . A typical arrangement of these supports may be described by examining the supports on existing bridges. Supports are usually provided at the top of the tower as well as at the intermediate loactions along the tower, depending on the number of cables used for the particular bridge system.

- **Fixed supports**

Different soluations have been used for the construction of fixed supports at towers. For the stromsund bridge, the tension cable terminate on the top of the tower and their attachments are shown in Figure (2-9).the cable sockets at this point are provided with eyes attached to the ribs of the tower- head bearings by pins. thus, while the cable cannot slide ,they are free to pivot vertically. This relative fixing of the cables increases the stiffness of the system. Adifferent type of fixed cable connection at the top of a saddle, in three levels, is shown in Apendix A Figure(A-1).

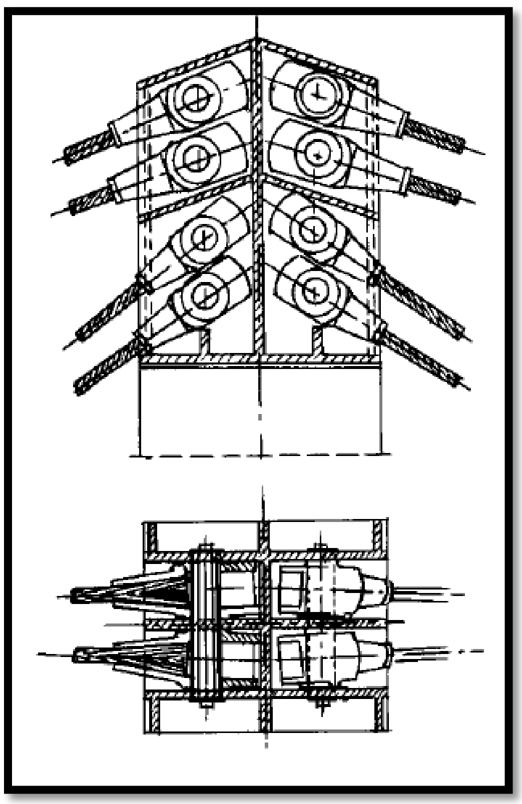


Figure (2-9): Attachment of cables on top of the tower,stromsund bridge, Troitsky, (1988).

Such saddles are typical for the harp type of cable arrangment. However, for the fan type, with all cables concentrated at the top, it is possible to anchor all cables at the tower top or to used one large or used one large and wide semicircular saddle In the case of many stay cables of the fan type ,Leonhardt suggested as the best soluation the use of a wide semicircular saddle of concrerte covered with a steel plate Figure (2-10).

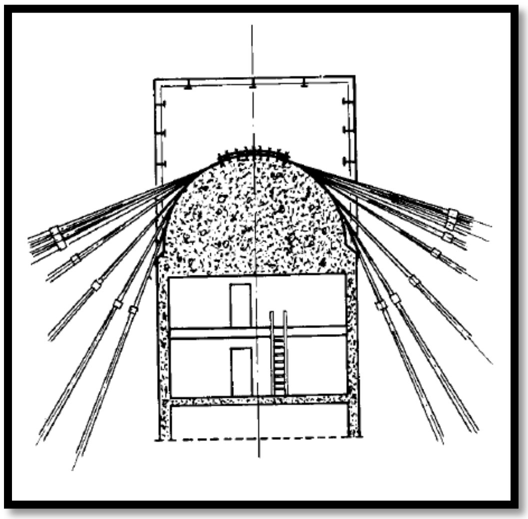


Figure (2-10): Semicircular saddle for large numbers of cables of the fan type,Troitsky ,(1988).

- **Movable cable supports**

in the case of application of movable supports, these take the form of appropriate rocker or roller devices. Typical arrangements of the movable cable connections are shown in Figure (2-11) .

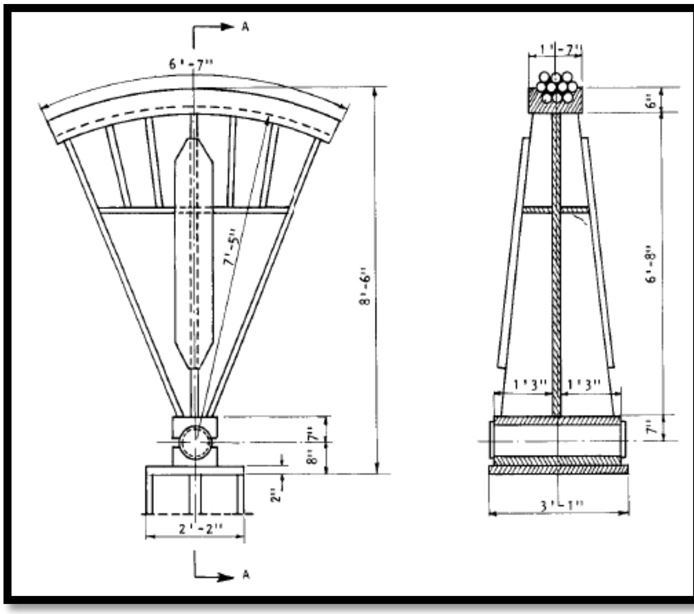


Figure (2-11): Rocker support for cables, North bridge ,Dusseldorf ,Troitsky,(1988).

2.3.1 Anchoring of the cables at the deck

- **Single strand Anchorages**

Troitsky, (1988), claimed that Stay cables constitute the main load carrying elements; therefore the details of their structural connections to the stiffening girders, the towers and substructure are very important. These cable connections should provide full transfer of loads, protection against weather, initial tensioning and adjustments, as well as access for inspection.

A socket widely used for the anchoring of parallel –wire strands is shown in Figure (2-12). The wires are led through holes in a licking plate at the end of the socket and have the bottom heads providing the resistance against slippage of wires. The cavity inside the socket is filled with hot zinc alloys. To improve the fatigue resistance of the anchor a cold casing material is used. To indicate a high amplitude socket it is called HiAm as shows in Appendix A Figure (A-2).

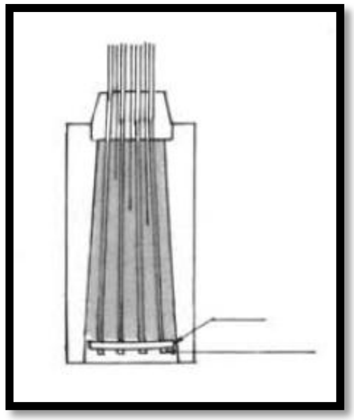


Figure (2-12): Socket for parallel –wire strands,Troitsky ,(1988).

2.3.2 Stiffening girder anchorages

Trotsky, (1988), had posited when negative bearing loads will occur at the free end bearings of the bridge, the main girders are connected to the abutment by vertical anchors, resisting tension and compression. The arrangements of these anchorages are illustrated by the example, of Stromsund Bridge Figure (2-13); in which the vertical anchors resist changeable vertical reactions and permit horizontal movements of the stiffening girder. However, the horizontal forces are transferred perpendicularly to the abutments through special spur bearings allowing longitudinal movements.

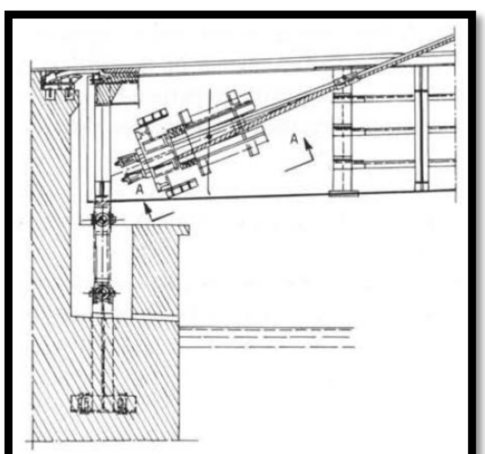


Figure (2-13): Anchorage of stiffening girder at abutment. Stromsund Bridge Troitsky,(1988).

2.4 Modulus of elasticity:

As stated by Podolny and B.Scalzi, (1986), The magnitude of the elastic elongation of a cable under tension is dependent upon the value of the Young's modulus of elasticity, which is defined as "the ratio of unit stress in the cable to a corresponding unit strain within a defined stress range". Unlike the usual conventional tension test, the value for the modulus of elasticity for cables is determined from a gauge length of not less than 100 in (254 cm) and is computed on the basis of the gross metallic area, which includes the zinc coating. Experience in prestretching has indicated that stress-strain data taken from 1600ft (487.7m) lengths are much more accurate than those taken from a 100 in (254 cm)gauge length.

The value for the modulus of elasticity is determined by calculation using the conventional expression for elastic elongation of a specified length of the material

Such that:

$$E = \frac{pl}{Ae} \qquad (2.1)$$

Where:

E:Young's modulus of Elasticity

p: Increment of load

A: Gross metallic area

e: Elongation caused by load increment

l : Gauge length

In fact, the value of E varies with the type of cable, such as strand, rope, or parallel wires, and is also determent on the amount of zinc coating applied to the wires. The ASTM Specifications state minimum values to be used for the various sizes and coating.

The equivalent or ideal modulus of elasticity of the cable as expresses by Ernst is

$$E_{i_I} = \frac{E}{1+(\frac{\gamma^2 l^2 E}{12\sigma^3})} \quad \ldots\ldots(2.2)$$

Where

E: Modulus of elasticity of straight cable.

l: horizontal length of the cable.

γ: specific weigth of the cable.

σ: tensile stress in the cable.

2.5 Optimum inclination of the cables

According to Troitsky, (1988), the height of the tower greatly affects the stiffness of the bridge system.

As the angle of inclination of cable with respect to the stiffening girder increases, the stresses in the cables decrease, as does the required cross section of the tower. However, as the height of the tower increases, the length of the cables, and therefore their axial deformations, also increase, as well as the amount of metal in the cables. The diagram shown in Figure (2-14) indicates that the optimum angle of the cable inclination is $45°$

And may vary in the reasonable limits of $25° - 65°$. The low values of the angle of inclination correspond to the external cables, while the greater values correspond to the internal cables, while the greater values correspond to the cables nearest to the tower

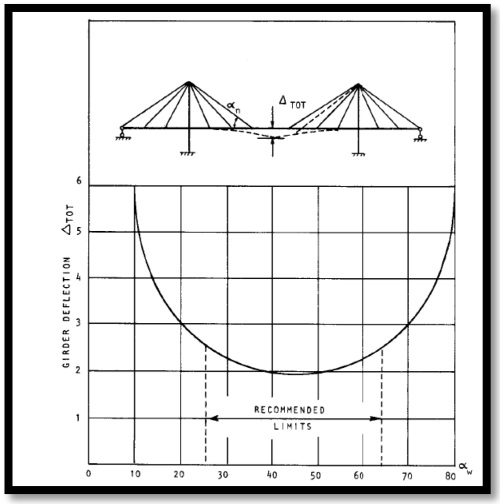


Figure (2-14): Relation between the cable inclination and deflection of the joint, Troitsky, (1988).

2.6 Number and spacing of the cables

Troitsky, (1988), mentioned that some cable stayed bridges have only a few stay cables, other have a large number of stays to support the stiffening girder, as shown in Figure (2-15).it is evident that using a small number of stay cables leads to larger cable force. There is no doubt that a larger number of stay cables with smaller spacing simplifies the anchoring and permits use of a shallower main girder. This shallowness facilitates a favorable cross-section for aerodynamic stability and simplifies erection. A large number of stay cables with a small spacing leads to the optimum in economy and structural simplicity. The spacing should be decrease from the tower to mid-span so that the cable forces do not become too different

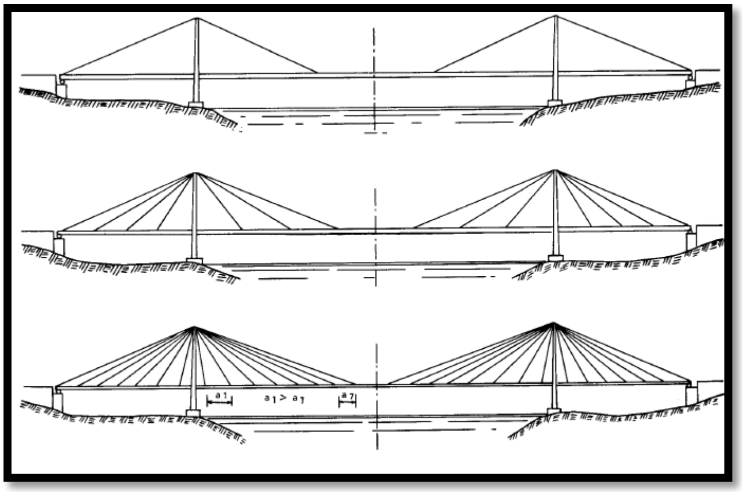


Figure (2-15): comparison of bridge systems, Troitsky, (1988).

2.7 Bridge systems

Troitsky, (1988), presented these structural systems possess, the following characteristics, as shown in Figure (2-16):

a. The towers are sufficiently flexible to be regarded as pinned at both ends.
b. The stiffening girders are continuous.
c. The stay cables are connected to the towers and to the stiffening girders.
d. The bearing at the abutments have vertical anchors to transfer negative reactions

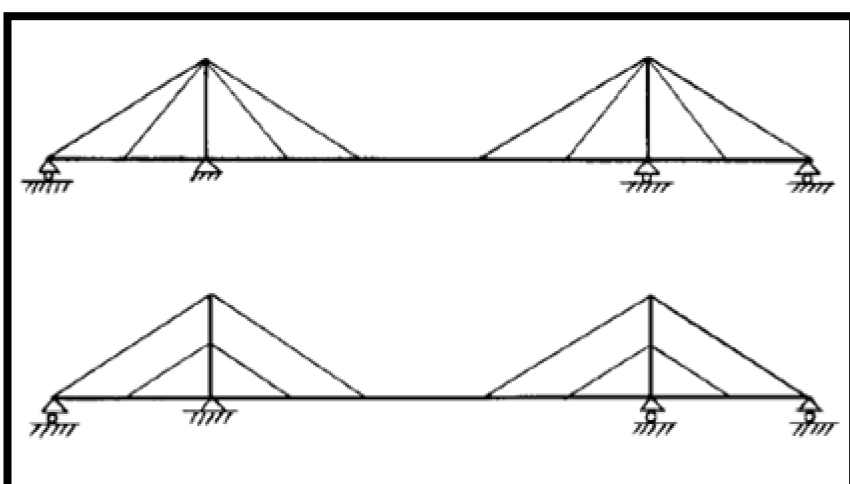


Figure (2-16): typical cable stayed structural systems, Troitsky, (1988).

2.8 Loads

As highlighted by Barker and Puckett, (2007), the engineer must consider all the loads that are expected to be applied to the bridge during its service life. Such loads may be divided into two broad categories: permanent loads and transient loads. In addition, all bridges experience temperature fluctuations on a daily and seasonal basis and such effects must be considered. Depending on the structure type, other loads such as those from creep and shrinkage may be important.

2.8.1 Gravity Loads

As stated by Barker and Puckett, (2007), gravity loads are those caused by the weight of an object on and the self-weight of the bridge. Such loads are both permanent and transient and applied in a downward direction (toward the center of the earth).

1. Permanent Loads

Permanent loads are those that remain on the bridge for an extended period of time, perhaps for the entire service life. Such loads include, as show in Appendix (A), Table (A-1) and Table (A-2).

- Dead load of structural components and nonstructural attachments (DC)
- Dead load of wearing surfaces and utilities (DW)
- Dead load of earth fills (EV)
- Earth pressure load (EH)
- Earth surcharge load (ES)
- Locked-in erection stresses (EL)
- Downdrag (DD).

The permanent load is distributed to the girders by assigning to each all loads from superstructure elements within half the distance to the adjacent girder. This includes the dead load of the girder itself and the soffit, in the case of box girder structures. The dead loads due to concrete barrier, sidewalks and curbs, and sound walls, however, may be equally distributed to all girders.

2. Transient loads

Transient loads, are those loads which are placed on a bridge for only a short period of time relative to the lifetime of the structure. They may be applied from several directions and/or locations, and typically include gravity loads due to vehicular, railway and pedestrian traffic.

2.8.2Design Vehicular Live Load

The AASHTO "design vehicular live load," HL93, is a combination of a "design truck" or "design tandem" and a "design lane." The design truck is the former Highway Semitrailer 20 ton design truck (HS20-44) adopted by AASHTO. Similarly, the design lane is the HS20 lane loading from the AASHTO Standard Specifications. A shorter, but heavier, design tandem is new to AASHTO and is combined with the design lane if a worse condition is created than with the design truck. Superstructures with very short spans, especially those less than 12 m in length, are often controlled by the tandem combination.

The AASHTO design truck is shown in Figure (2-17). The variable axle spacing between the 145 kN loads is adjusted to create a critical condition for the design of each location in the structure. In the transverse direction, the design truck is 3 m wide and may be placed anywhere in the standard 3.6-m-wide lane. The wheel load, however, may not be positioned any closer than 0.6 m from the lane line, or 0.3 m from the face of curb, barrier, or railing. The AASHTO design tandem consists of two 110-kN axles spaced at 1.2 m on center. The AASHTO design lane loading is equal to 9.3 N/mm and emulates a caravan of trucks. Similar to

The truck loading, the lane load is spread over a 3-m-wide area in the standard 3.6-m lane. The lane loading is not interrupted except when creating an extreme force effect such as in “patch” loading of alternate spans. Only the axles contributing to the extreme being sought are loaded. When checking an extreme reaction at an interior pier or negative moment between points of contraflexure in the superstructure, two design trucks with a 4.3-m spacing between the 145-kN Axles are to be placed on the bridge with a minimum of 15 m between the rear axle of the first truck and the lead axle of the second truck. Only 90% of the truck and lane load is used.

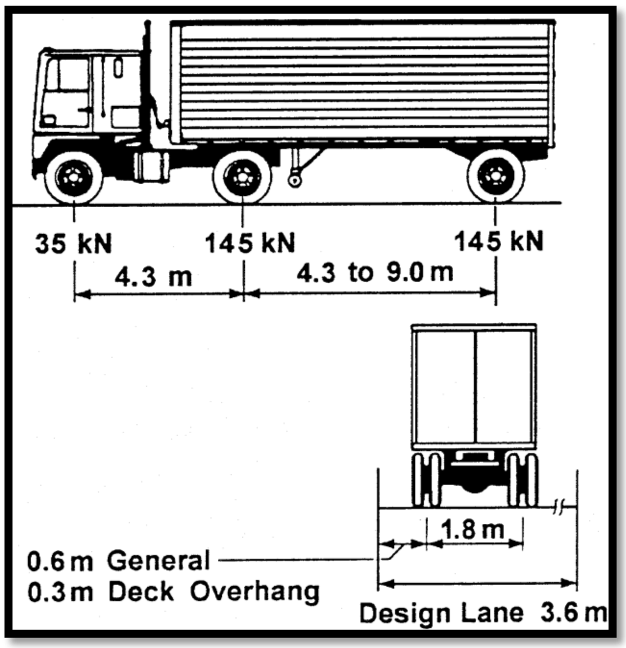


Figure (2-17):AASHTO-LRFD design truck. AASHTO LRFD, (2010).

2.8.3 Dynamic Load Allowance:

Unless otherwise permitted in Articles 3.6.2.2 and 3.6.2.3, the static effects of the design truck or tandem, other than centrifugal and braking forces, shall be increased by the percentage specified in AASHTO Table 3.6.2.1-1 for dynamic load allowance. The factor to be applied to the static load shall be taken as:

$(1 + IM/100)$. The dynamic load allowance shall not be applied to pedestrian loads or to the design lane load.

2.9 Dynamic analysis

Troitsky ,(1988),claimed that The majority of cable –stayed bridge methods of analysis are limited to static loads with very little information being presented concerning dynamic behavior. However, under the influence of wind, sesimic and traffic loads, there is adynamic response bycable –stayed bridges . therefore, the dynamic analysis of cable – stayed bridges is concerned with their areodynamic and seismic behavior.dynamic studies include the determination of the natural modes and frequences of the bridge under aerodynamic forces. The second type deals with the response of the bridge under earthquake action.

- Natural frequencies and principal modes of vibration
- Methods of determination of natural frequencies

Walther Rene,(1999), mentioned that There are a number of methods for calulating the frequency of a strucure to be strictly parctical, study of these will be limited to methods which can be used manally .To simplify the numerical operation , the mass of the structure is concentrated at a certain number of separate points and the influence of damping (which is small in practice)is neglected the methods treated are the following :

1. Classical method based on the differential equation of movement
2. Rayleighs method based on considerations of energy simply method

- **Classical method**

The dynamic equations were written by Morris in the following matrix form:

$[M]\{D\} + [C]\{\dot{D}\} + [K]\{D\} = \{P\}$………………………………………….... (2.3)

$[M]$: The mass matrix. It is made up of lumped masses consisting of half the weight of each member framing into anode. The rotary inertia contribution was computed by assuming that half of each member rotates as a rigid bar out of the node.

{D}: The displacement vector (u, υ and θ at each joint)

[C]: The damping matrix.

{Ḋ}: The elements which are the velocities at the nodes.

[K]: The matrix formed from member stiffness matrices [K] evaluated at the deformed position of the member.

$\{\ddot{D}\}$: The vector containing the accelerations $\ddot{\Theta}_x$, $\ddot{\Theta}_y$,.$\ddot{\Theta}_z$$\{P\}$: The force matrix, made up of concentrated forces and couples at the nodes.

2.10 Wind effect

2.10.1 Wind Enviroment

Podolny and B.Scalzi, (1986), mentioned that Before wind instability studies are conducted for a particular bridge,it is important to estimate the wind enviroment at the particular site,it is deisrable, in the determination of wind action on asupended bridge structure, to obtain information

Of strong wind activity at the site over a period of years. Required are the wind velocity, direction, and frequency.this type of data is generally obtainable from meteorological record of the U.S.weather Bureau and similar local weather records. However, these data are generally recorded at an airport or federal building at a nearby city which may be some distance from the bridge site. these records should be carefully used because the effects of the terrain at the instrument location may be somewhat different from those at the bridge site.

2.10.2 Horizontal Wind Pressure

Pressures specified herein shall be assumed to be caused by a base design wind velocity, V_B, of 100 mph. Wind load shall be assumed to be uniformly distributed on the area exposed to the wind. The exposed area shall be the sum of areas of all components, including floor system and railing, as seen in elevation taken perpendicular to the assumed wind direction. This direction shall be varied to determine the extreme force effect in the structure or in its components. Areas that do not contribute to the extreme force effect under consideration may be neglected in the analysis. For bridges or parts of bridges more than 10 m above low ground or water level, the design wind velocity, V_{DZ}, should be adjusted according to (AASHTO 2010) as following:

$$V_{DZ} = 2.5V_0 \left[\frac{V_{10}}{V_B}\right] \ln\left[\frac{Z}{Z_0}\right] \ldots\ldots (2.4)$$

Where:

V_{DZ} : design wind velocity at design elevation, Z (mph).

V_{10} : wind velocity at 10m above low ground or above design water level (mph).

V_B : base wind velocity of 100 mph at 10m height, yielding design pressures specified in AASHTO Articles 3.8.1.2 and 3.8.2

Z : height of structure at which wind loads are being calculated as measured from low ground, or from water level, > 10m.

V_0 : friction velocity, a meteorological wind characteristic taken, as specified in AASHTTO Table 3.8.1.1-1, for various upwind surface characteristics (mph)

2.10.3 Wind Pressure on Structures: *WS*

If justified by local conditions, a different base design wind velocity may be selected for load combinations not involving wind on live load. The direction of the design wind shall be assumed to be horizontal, unless otherwise specified in AASHTO Article 3.8.3. In the absence of more precise data, design wind pressure, may be determined as (AASHTO 2010)

$$P_D = P_B \left[\frac{V_{DZ}}{V_B}\right]^2 \ldots\ldots (2.5)$$

Where:

P_B : base wind pressure specified in AASHTO Table 3.8.1.2.1-1

2.11 Construction

Podolny and B. Scalzi, (1986), stated that fabrication and erection costs add significantly to project cost estimates, and as a result, present trends are to fabricate components as large as possible for simplified erection. In this manner larger components of the project are assembled in the shop in contrast to assembling many smaller units in dangerously elevated, exposed positions on the project site. The techniques and methods of erecting cable – stayed bridges are as varied and numerous as the ingenuity and number of erector contractors.

2.11.1 Methods of erection

Podolny and B.Scalzi, (1986), mentioned that the methods of erection for cable - stayed bridges are broadly described by three general methods: the staging method, the push – out method, and the cantilever method, more specific details are provided in the following section:

- The staging method of erection is most often used where there is a low clearance requirement to the undesired of the structure and temporary bents will not interfere with any traffic below the bridge. Its advantage is its accuracy in maintaining required geometry and grade it's relatively low cost for low clearance.
- The push – out technique has been used successfully on a number of occasions in Europe but is relatively new to American construction. This method is commonly used in Europe where care must be taken not to interfere with traffic below the bridge and where Cantilever construction is impractical. In this method, large sections of bridge deck are pushed out over the piers on rollers or sliding Teflon bearings. The deck is pushed out

from both abutments toward the center, or, in some instances, from one abutment all the way to the other abutment. Assembling the components in an erection bay at one or both ends of the structure and progressively pushing the components out into the span as they are completed can simplify construction and reduce costs. With this method as much as 1500 tons of steel, spanning a number of supports, have been pushed out and, in some instances, it has been used where a horizontal curvature is required.

- The cantilever erection method is very often employed in cable – stayed bridge construction where temporary supports are necessary. It may increase the steel requirements over that required for final positioning to accommodate the increased moments and shears during the erection process. The principal advantage is that it does not interfere with traffic below the bridge.

2.11.2 Construction errors

Wang and C. Fu, (2015), claimed that Construction errors, which may cause incorrect assumptions in structural analyses, are due to the quality control of construction and may include the following features:

1. Material properties such as errors in Young's modulus, temperature expansion factor, and material densities

2. Sectional properties such as errors in girder dimensions due to installation or formwork deformations

3. Temporary construction loads

4. Creep and shrinkage properties for a concrete cable-stayed bridge

2.12 Methods of structural analysis

As stated by Troitsky, (1988), the analysis of the cable- stayed bridges under static and dynamic loads, considering their linear and nonlinear behavior along with the above parameters, is discussed in the following section:

1. Linear analysis

Cable –stayed bridge systems are generally many times statically indeterminate. Astatically determinate basic systems may be formed by different methods. The deflections of the basic system under applied loads may be determined by applying the classical theory of structures or so- called first order theory. For statically determined basic system, the resulting equations are linear in the loads and in the internal forces, and linear superposition is valid for the internal forces caused by different loads or load groups.

If Hooke's law is assumed to be valid, linear superposition applied also to the displacements, and therefore, to the determination of the stresses of cable stayed bridge systems.

If the analysis of cable –stayed bridges is based generally on the assumption that the elastic displacements of the structure are proportional to the applied load, it is defined as linear behavior.

2. Non-linear analysis

Non-linear performance of the cable- stayed bridges generally depends on the behavior of the cable, stiffening girders and pylons.

- Nonlinearity of the cables

Nonlinearity of the cables originates with an increase in the loading followed by a decrease in the cable sag, which produces an elongation of the cable and corresponding axial tension. To overcome this nonlinear effect, the method of equivalent modulus of elasticity was proposed to include the normal modulus and the effect of sag and tension load. These factors are expressing changeable stiffness of stay cables. Actually, the stiffness depends on the tensile stress, length of the cable and its deflection.

- Non-linearity of stiffened girders and pylons

When stiffened girders and pylons are subjected to the simultaneous action of compression loads and bending moments, then the inter action of loading and axial forces results in nonlinear behavior. The degree of nonlinearity depends on the intensity of the compressive load compared with the buckling load and the magnitude of deflection caused by the relatively slender. Because of the presence of high compressive forces in the relatively slender stiffening girder and towers, the girder and the towers need to be analyzed as a beam column, the axial compression force increases the bending moment of beam column, and the resulting relationship is nonlinear.

- Non-linearity due to deformation of the structure

In a cable –stayed bridge , the deformation of the superstrucure under loading affecte the value of the stress. Therefore, the principle of superposiotion may be applied only with certain limitations. This problem is treated by the deformation theory or so –called second order theory by taking into account the effect of the deflections of the structure in calculating the stresses and forces. The equilibrium conditions are written down for the geometry of the deformed structure and they are no longer linear. At the first stage, the stresses are calculated considering the initial geometry of the structure, applying the princiles of linear analysis . the deformations obtained are used further to determine the modified geometry of the structure. At the second stage , the linear analysis is applied again for the structure with modified geometry. This methods is repeated until the deformations remain constant from one stage to the next. Two or three times are genarally sufficient.

For the nonlinear performance of cable – stayed bridges, the analysis procedure is to consider the nonlinear behavior of the cables, caused by the variation in sag with tensile force, and the nonlinear behavoir of the bending members, caused by the interaction oa axial and bending deformations.

2.13 Geometrical Non-linear Theory for cable –stayed bridge

Shikalgar and Sanade,(2021),stated that, the Nonlinearities can be broadly divided in geometrical and material nonlinearities. While the latter depend on the specific structure (materials used, loads acting, design assumptions), geometric nonlinearities are present in any cable-stayed bridge.

As stated by Xion-Hui and Zi-Qiang,(2015), Based on the nonlinear theory of cable-stayed bridge, There are three aspects of geometric nonlinearity, cable sag, beam column effect, large displacement are calculated accurately using element geometric stiffness matrix, CR formulation, bar unit, Ernst formula and the catenary equation. The treatment of the three nonlinear factors is discussed as Following.

1. Non-linear Effects Generated by Cable Sag

The truss element is the simplest option for modeling the cables of cable-stayed bridges. It may be used both in static and dynamic analysis, on condition that the tensile stresses in the cables are high enough to make the sag effect neglectable. In order to allow large displacements to be handled, geometrical stiffness factors have to be added to the mathematical formulation of the element. The multiple-straight link approach is one of the most powerful ways for modeling the actual behavior of cables using truss elements, allowing for both the sag effect due to the self-weight and the vibration modes of the cables to be accounted for.

Axial stiffness of stay cables change with sag, in turn sag depends on the tension of stay cable, there is a clear nonlinearity between tension and distortion of stay cable if nonlinearity is considered, and stiffness can be described by equivalent elastic modulus and equivalent elastic modulus of stay cable. AS stated by Freire,(2006), the Ernst method or modified elastic modulus method is often used in the analysis of cable-stayed bridges, given its capability to account for the sag effect and the ease of use, combining a rather simple mathematical formulation with a linear analysis methodology can be described as following (Ernst formula):

$$E_{eq} = \frac{E_0}{1+[(\frac{\gamma l)^2}{12\sigma^3}]E_0} \quad \text{……(2.6)}$$

Where

E_{eq}=is equivalent elastic modulus E_0=is the elastic modulus of cable

σ = tensile stress l=is the length of stay cable

2. Deck and towers: Beam-Column Coupling Effect

Under the tension of cable, main girders,towers and other components receive bending moments and axial force action simultaneously. Under this situation, the material that satisfies Hooke's Law also presents non-linear characteristics. Lateral deflection of the component under axial force can cause additional moment and, in turn, the moment has effect on the size of the axial stiffness. The method based on the finite element discretization and used to handle beam-column coupling effect nonlinearity of cable-stayed bridge is stabilizing Function.

The stabilizing function of axial rod is

$$\mathrm{S_1} = \frac{\mathrm{u^3 \sin hu}}{\mathrm{12(2-\cos h\,u+u\sin h\,u)}} = \frac{\mathrm{u^3 \sin hu}}{\mathrm{12\,R_2}} \quad \text{……(2.7)}$$

$$\mathrm{S_2} = \frac{\mathrm{u^3\,(coh\,hu-1)}}{\mathrm{6(2-\cosh u+u\sin h\,u)}} = \frac{\mathrm{u^3\,(\cos hu-1)}}{\mathrm{6\,R_2}} \quad \text{……(2.8)}$$

$$S_3 = \frac{u(u\cosh u - \sin hu)}{4(2-\cos h\, u + u\sin h\, u)} = \frac{u(u\cosh u - \sin hu)}{4\ R_2} \qquad (2.9)$$

$$S_4 = \frac{u(\sinh u - u)}{2(2-\cos h\, u + u \sin h\, u)} = \frac{u(\sinh u - u)}{2\ R_2} \qquad (2.10)$$

$$s_5 = \frac{1}{(1-\frac{EAR_{im}}{4F^3L^2})} \qquad (2.11)$$

Where:

*S (*1-5): is stabilizing function, $u = 1\sqrt{F/EI}$

F: is the axial force. EI: is flexural rigidity of the component,

l :is the length of the cable, A: is cross-section of cable.

3. Large Deformation Effect

According to Wang and C. Fu, (2015), as the main span of the bridge increases, the global stiffness decreases and the displacements (not the deformation) become significant .large displacement behavior in long-span cable-stayed bridges should be investigated case by case. In general, when large displacements are considered, The lateral stiffness of a bridge will be enhanced due to the cables' geometric stiffness under significant tensions, that is, the tendency to maintain its lateral positions.

Xion-Hui and Zi-Qiang,(2015),claimed that, The geometry position of the superstructure of cable-stayed bridge changes significantly under normal loads, and the coordinates of a structural node changes significantly. The length, angle and other geometrical characteristics of each unit also change significantly, so, the balance equation present nonlinear relationship.

- **Newton-Raphson iteration method**

In the geometric nonlinear analysis of a structure being subjected to external loads, the geometric stiffness is expressed as a function of the displacement, which is then affected by the geometric stiffness again. The process requires repetitive analyses. The Newton-Raphson method is a widely used method. The stiffness matrix is rearranged in each cycle of repetitive calculations to satisfy equilibrium with the load given in the equilibrium equation of load-displacement. A solution within the allowable tolerance is obtained using the stiffness matrices through the process of iteration.

$(\mathrm{K} + \mathrm{K}_{\sigma})\mathrm{u} = \mathrm{p}, \mathrm{K}_{\mathrm{T}}\mathrm{u} - \mathrm{p}$

$\mathrm{K}_{\mathrm{T}} = \mathrm{K} + \mathrm{K}_{\sigma}, \mathrm{K}_{\sigma} = \mathrm{f}(\mathrm{u})$…………………………………………………….....(2.12)

$\mathrm{K}_{\mathrm{T}}(\mathrm{u}_{\mathrm{m}-1})(\mathrm{u}_{\mathrm{m}-1} + \Delta\mathrm{u}_{\mathrm{m}}) = \mathrm{R}_{\mathrm{m}}$

- **P-Delta Analysis**

The P-Delta analysis option in MIDAS/Civil is a type of Geometric nonlinearity, which accounts for secondary structural behavior when axial and transverse loads are simultaneously applied to beam or wall elements. The P-Delta effect is more profound in tall building structures where high vertical axial forces act upon the Laterally displaced structures caused by high lateral forces.

The P-Delta analysis feature in MIDAS/Civil is founded on the concept of the numerical analysis method adopted for buckling analysis. Linear static analysis is performed first for a given loading condition and then a new geometric stiffness matrix is formulated based on the member forces or stresses obtained from the first analysis. The geometric stiffness matrix is thus repeatedly modified and used to perform subsequent static analyses until the given convergence conditions are satisfied.

The static equilibrium equation for P-Delta analysis used in MIDAS/Civil can be expressed as

[K]{u}+[KG]{u} = {P}…………………………………………………….....(2.13)

Where,

[K] : Stiffness Matrix of pre-deformed model

[KG] : Geometric stiffness matrix resulting from member forces and stresses at - each step of iteration

{P} : Static load vector

{u} : Displacement vector

2.14 Preliminary manual calculations

As proposed by Podolny and Scalzi, (1986), Because of the large degree of indeterminateness of cable –stayed bridge structures, exact calculation by manual procedures is virtually an impossible task. The many parameters involved present a formidable hurdle to manual calculations. This is not an accurate solution that the calculations provide a means of determining first- trial values of required cable stayed bridge area. If the cable forces acting under gravity (dead) loads are such that all deformations in the girder and pylon are zero. If as a first –trial approximation, live load is applied to the same system, the stay forces p_i in Figure (2-18) can be determined by equation

$$p_i = \frac{R_i}{\sin\propto_i} \quad \text{……(2.14)}$$

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

$$A_i = \frac{R_i}{\sigma_{allow} \sin\propto_i} \quad \text{……(2.15)}$$

The reaction, $\boldsymbol{R_i}$ at each cable- stayed node May simple is determined as $\boldsymbol{R_i = sw}$**,** where **s** equals spacing between cable supports and **w** equals the uniformly distributed load. However, at the end of the girder $\boldsymbol{R_i}$ may be determined by other means.

To determine the force $p_{\circ}$in the back- stay cable the horizontal force in the pylon top $\boldsymbol{F_h}$ must be first calculated. Maximum force in the back stay cable will be

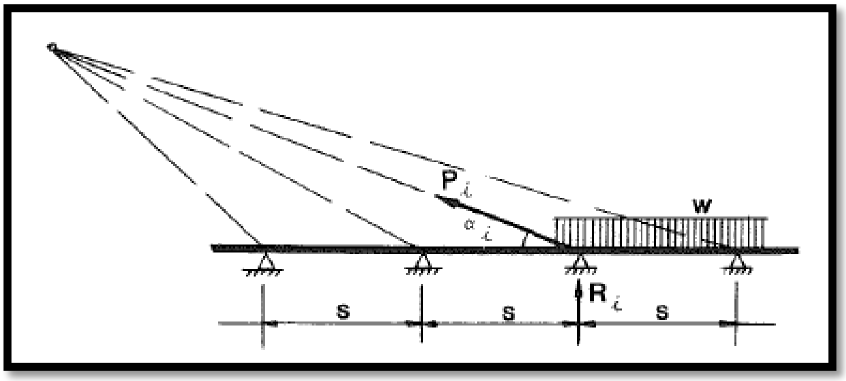


Figure (2-18): Cable force p_i, Podolny and B.Scalzi, (1986),

Produced with dead plus live load in the center span and dead load only in the side span if the pylon head is assumed to be immovable. Then F_h can be determined from the following equation:

$$F_h = \sum \frac{R_i^r}{\tan\alpha_i^r} - \sum \frac{R_i^l}{\tan\alpha_i^l} \quad \text{.....(2.16)}$$

Where:

R_i^r: Reaction at right support . α_i^r: Angle at right support

R_i^l: Reaction at left support. α_i^l: Angle at left support

If in Figure (2-19) the change in angle$\alpha_{\circ}$ is assumed negligibly small as the pylon deflects under the loadF_h then the load in the back stay cable can be determined as

$$p_{\circ} = \frac{F_h h_t^3 \cos\alpha_{\circ}}{3l_{\circ}(E_c I / E_s A_{\circ})} \quad \text{.........(2.17)}$$

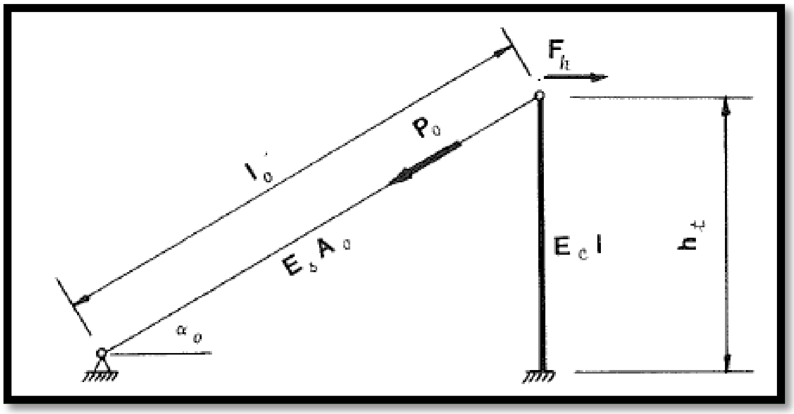


Figure (2-19): Back stay force diagram, Podolny and B.Scalzi, (1986).

2.15 Back ground Review

The study of the cable stayed bridge has been of great interest for many years. Various studies are available on cable stayed bridges. A brief scenario of some of these studies is presented below.

1. As stated by Zhang and ZhiMin,(2011) , in this paper the computer program MiDAS is used to model and analyse the examples. The bending moments in the main girder and the pylon are minimized by the chosen cable forces. Structural analysis programmes apply optimisation methods to minimize the internal forces in the calculation of the ideal cable forces. The calculation considers user defined restrictions for forces or moments, stresses and displacements; the reasonable results are obtained.
2. Chaudhari and Sorathia, (2019), mentioned that cable stayed bridge will be modeled with proper technique from the guidance of different tutorial survey of Bridge design that is provided by software. A static moving load analysis is carried out and various response quantities like Bending moment, Shear force, Displacement, Torsion and axial force are

represented. Then for carrying out parametric study of different shape of pylon and the different central span of the bridge, different models have been prepared in MIDAS civil. The Shape here taken for study are H shape and A shape pylon. The span taken for study varied from 480m, 950m, 1412m.It can be seen that Shear force value are increasing for H shape pylon bridge girder as increasing span of the bridge and decreasing for A shape pylon bridge girder. The result of this study is represented in graphical form for the various response quantities.as shown in Apendix (A) Figure (A-3) to (A-5)

3. According to Raj Ritu, (2016), the intent of this research is to present a detailed study of various phenomenon's' that induce vibrations in Cable-stayed bridges. The wind load analysis is carried out for a Cable-stayed bridge model for Indian terrain conditions. The wind load analysis is carried out on MIDAS CIVIL 2012 to check the behavior of bridge vulnerability against wind forces. It involves the determination of natural frequency of stay cables and profile of the cable under axial tension and self- weight of the cable. The result obtained of this work as show following

- **Static analysis**

Static analysis of the cable-stayed bridge model has been done considering the dead loads only. For the static analysis under dead loads, the dead load values of various components of the bridge is taken by the software automatically using the volume of the components and the density of the materials assigned to it. MIDAS Civil has large capabilities of static analysis procedure, shows Table (2-2).

Table (2-2): Maximum Responses static analysis, Raj Ritu, (2016).

Absolute Maximum Values	Vertical Deflection	Along the Deck	Axial Force	Shear Force	Bending Moment
	M	M	KN	KN	KN.m
Girder	0.34	-	19303	4209	15127
Pylon	-	0.103	76812	550	43989

- **Wind analysis results**

Raj Ritu, (2016), stated that Wind analysis of the hypothetical cable-stayed bridge model has been done considering the wind loads calculated below. For the wind analysis, the wind load Values of various components of the bridge are taken from the calculated values as shows in Table (2-3) and see appendix (A) Table (A-3). These loads are introduced as nodal loads at appropriate locations on the Cable-stayed bridge structure.

Table (2-3): Maximum Responses wind analysis. Raj Ritu, (2016).

Absolute Maximum Values	Transverse Deflection	Along the Deck	Shear Force	Transverse Bending Moment
	M	M	T	t-m
Girder	0.00512	-	-	-
Pylon1	0.0534	-	164.47	2639.39
Pylon 2	0.034	-	71.82	906.39

3 As stated by Vikas et al, (2013), in the present work a cable stayed bridge of fan type arrangement is analyses for static and dynamic load. The analysis is done with all the cables under normal condition, different percentage of corrosion of one cable and the failure of one cable due to excessive corrosion. The analysis is performed using finite element method software MIDAS

Civil. The software is validated with simple bridge model. The bridge is analyses for moving load case as per the IRC 6-2000 and also for earthquake load (Time History of El Centro (earthquakes response spectrum)) and for different load combinations load case as per the IRC 6-2000 and also for earthquake load (Time History of El Centro) and for different load combinations. Static and dynamic analyses are performed on the cable stayed bridge. Static analysis is performed to find the dead load and live load behavior of the structure. The dynamic analysis is to find the dynamic properties of the structure. The result obtained from this paper as shows follows and in Appendix (A), Table A-4 and Table A-5.

4 Koyani and Koradia,(2016), claimed that the parametric study of cable stayed bridge is carried out. In this study three span, two plane cable stayed bridge with box girder deck is considered. For analysis IRC class AA moving load is considered and its effect on cable stayed bridge girders is studied. Analysis is carried out with help of MIDAS CIVIL software. The various parameters were considered for analysis of cable stayed bridges; those are side span to main span ratio, upper strut height, cable system, number of cables per plane and cable diameter. The result obtained from these studies with the increase in side to main span ratio maximum moment is decrease up to certain limit and then increases. With increase in number of cables maximum moment in girder decreases.

5 Hararwalal and Maaru, (2016), studied the effect of the shape of pylon on the dynamic response of cable stayed bridge, modeling cable stayed bridges with different shapes of pylons using SAP 2000 software. Only the pylon shape was varied (A type, H type, inverted Y type, Single pylon, Diamond) but the height of bridge and span dimension were kept constant. The result obtained in this paper is, the angles which have been made by inclined pylons with

deck shall be lies between “600 – 750 and the minimum spacing between cables which was jointed at pylon shall not be less than H/100 in meters where H= height of pylon from the ground level. Also, The intermediate supports which were used as side span supports shall have the minimum spacing of 20 meters between them.

6 Chengfeng et al, (2015), studied the numerical analysis of long –span cable stayed bridge in the construction phase. A general methodology for construction processes had been presented to simulate a cable –stayed bridge. The Sutong Bridge was simulated with finite element analysis ANSYS software package. The cable tensions were realized with ANSYS parametric design language. Results of the construction stage analysis showed that the temperature method could simulate the adjustment of the inclined cable force successfully, and the global stiffness of the Sutong Bridge was very small before closure. During the construction of the Sutong cable-stayed bridge, extensive field measurements have been made to monitor the geometry of the deck and tower, as well as the cable force. These field measurement results are Compared with the calculated results to evaluate the behavior of the actual structure

7 Chen, (2000), proposed a force equilibrium method for finding the cable stresses in cable-stayed bridges. He considered three stages of the structure model in the optimization procedure. The bending moments were considered controlling parameters in his study, instead of the displacement constraints. Because of the three modeling stages of the analysis this approach is more time-consuming than the other methods.

8 Rageh and Maslennikov, (2013), presented in their paper a study of cable-stayed bridges having three spans with double plane of cables. Three types of bridge arrangements were considered namely harp, fan and radiating shapes.

Also, they examined the influence of the arrangements of cables on the bridge deformation. Analysis of bridge model was carried out using a computer program in FORTRAN language. The result of non-linear static analysis using energy method showed that the smallest deflection value of deck was in radiating type and the biggest one was in harp type model. For pylons' tops lateral displacement, the smallest value was in radiating type and the biggest one was in harp. It was observed that the most forced cables were the long ones in the middle span and edge spans. The biggest cable forces were in harp type, which contributes to larger normal axial force in the deck and smallest forces were in radiating type. The finding suggests that the harp cable arrangement appears less suitable than the fan and radiating cable arrangement especially for long-span bridges.

9 Jani and Amin,(2017),carried out a study of the Bandra-Worli Sea link, Vidyasagar Setu, Atal Setu cable stayed bridge in India under cable loss. The bridge was modeled in SAP2000 software .The aim of their study was to present the effect of corrosion on mixed and fan type cable stayed bridge and loss of cable due to increasing corrosion as well as sudden cable loss. The result obtained from this paper is following: The cable axial strength decreases with increase in corrosion. Due to corrosion modulus of elasticity decreases, which resulting in reduction of structural stiffness. The cable forces near the vicinity of ruptured cable increases as much as 27% in case of cable loss due to excessive corrosion. The cable in middle and far away from pylon suffers from extreme effect of cable loss compared to cables nearer to pylon in excessive corrosion. While the axial forces drastically increase in the adjacent cable as much as 88% in sudden cable loss. When the middle cable abruptly fails, forces also increase significantly in the cables near the vicinity of the ruptured cable.

10 Garg and Chaturvedi, (2019), studied the behavior of cable stayed bridges of fan arrangement under static and vehicle loading. They used two different types of structural models, the Spine Model and Area Object Model, for the analysis of the cable stayed bridge. The study results were compared using tables and graphs to find out the best structure model for analysis by using software CSI Bridge. The result obtained from this paper as following: The shear force in area object model is 10% less than spine model. The shear force produced in area object model is at ends of the bridge which is more acceptable than that produced at center in spine model, the maximum positive moment about horizontal axis for both model is approximately same and no much deviation is observed but the maximum negative moment about horizontal axis for area object model is less as compared to the spine model. The reduction in bending moment is almost 17% in area object model and the maximum deflection in area object model is comparatively more than spine model. The increasing percentage of deflection in area object model is more than 4%. Hence increase of deflection in area object model is more adequate.

11 Pipinato et al, (2012), studied the analysis of cable-stayed bridges at various erection stages during construction. The forward construction process and the backward construction process analysis were investigated by using MIDAS 2000. They also compared the use of either the linear computation procedure (linear theory) or the nonlinear computation procedure (nonlinear theory). It is confirmed that both the forward and backward methods can be used successfully for the partial and the full cantilever method in both the partial and full cantilever method, the nonlinear theory offers theoretically more accurate results than that determined by the linear theory, even if the computation becomes more complicated and time-consuming when the nonlinear theory is utilized.

12 Granataa, et al, (2012), proposed a methodology for the evaluation of initial cable forces in composite bridges, based on simple partial elastic schemes of construction stages and presented reports for a comparison between different stay stressing sequences in order to evaluate the consequences of each one to the states of stress and to the final geometry of deck and pylon.

13 Karthik, and Bage ,(2017), carried out the static analysis on an actual three lane cable stayed bridge in Nagpur known as “Ram Jhulla”, which was under construction above the Nagpur Railway Station, with overall span of 200 m. Also, the construction stage analysis considering time dependent material properties like creep and shrinkage was carried out using Finite Element MIDAS Civil software. They also checked various parameters like cable forces, deflection, axial force, bending moment during construction stages. The results obtained for deflections were compared with actual field measurements. The maximum cable forces varied from 1100 kN near pylon to 3400 kN at extreme end .The forces in cables are maximum when the next cantilever deck segment gets activated and minimum when next cables get activated to take weight of cantilever deck. the moments in each deck element in terms of sagging and hogging can be understood ,maximum sagging moment is 28000kN-m The top end of pylon deflects the most during stressing of C26 cable(end cable) and the maximum horizontal deflection in pylon top is 320 mm towards right side.

- **Summary of the problem obtained in these studies:**

1. The determination of the initial cable forces in concrete cable-stayed bridge. (Traditional method ``zero displacement" is difficult task)

2. Study the influence of horizontally axial pressure in the main girder for cable –stayed bridge.

3. One of the important issues in the design and analysis of cable-stayed bridges is determining the pre-tensioning cable forces that optimize the structural performance of the bridge.

4. Various wind effects and the different vibrations which are induced due to the wind on cable-stayed bridges.

5. The precise nature of excitation and vibration development due to the wind loads still stay to be precisely understood.

6. The effect of shape of pylon on the dynamic response of cable stayed bridge

7. The aim to obtain a convenient final geometry through the control of deformations from the first stage to the last one, coincident with the service life configuration

8 adopting the optimal values of cable force, and determining the reasonable finished bridge state for large span Cable-stayed bridge with hybrid girder

9. Investigation of the response of cable – stayed bridge under static wind load.

10. Changes of the geometric configuration, restraints and consequently stress during construction sequence of cable-stayed bridge

11. Static behavior of cable stayed bridges and effect of various parameters on cable stayed bridge.

12. Study of the behavior for longer span cable stayed bridges under static and vehicle loading is important.

13. The failure of cable may be one of the accidental or eventually event, which must be considered during cable stayed bridge design. The cables in

a cable stayed bridge are exposed to corrosion, abrasion and fatigue processes which may cause a reduction in their section and a decrease in their resistance capacity.

14. The influence of the arrangements of cables on the cable- stayed bridge.

CHAPTER THREE

Analysis, Design of Cables, Modeling and Simulation of Cable - Stayed Bridge

3.1 Introduction

This chapter contains the methods of structural analysis, preliminary manual calculations, numerical analysis model of cable stayed bridge using Midas civil computer program description of Tuti - Bahri cable – stayed bridge under study and a brief description of computer program Midas civil. Explaining the steps of input data and review of the result obtained.

3.2 Methods of structural analysis of cable – stayed bridge

A number of techniques can be used for analysis of cable - stayed bridges. Examples include the use of a scaled- down model for testing, and the use of an analytical model which considers the linear and nonlinear behavior of the cable - stayed bridge when subjected to static and dynamic conditions of load. For the analytical model, certain parameters should be defined and idealized, such as the restraints at the joints, the stiffness or flexibility of each member, and connections between the cables, stiffening girders and towers. It is proposed to use the equations presented in Appendix (B) which have been modified to develop a model of the cable – stayed bridge, these equation for analysis under dead load only .They were proposed by Troitsky,(1988), for a limited number of cables .They have been modified for any number of cables .can be easily programmed to reduce the bending moments and deflection of the stiffening girder.

3.3 Finite Element Analysis and design of cable stayed bridges

3.3.1 Introduction:

For the Finite Element analysis (F.E.A) and design MIDAS / Civil will be used which is a total integrated solution system for analysis and design in civil infrastructure especially bridges. It includes the design and structural analysis of bridges, steel, concrete and composite bridges, suspension bridges and cable stayed bridges.

3.3.2 Special features of MIDAS/Civil for cable stayed bridge analysis:

MIDAS/Civil provides pre- and post-processors in conjunction with Cable Stayed Bridge Wizard, which readily creates a cable stayed bridge model. Initial tension forces in cables can be also calculated through the Unknown Load Factor function. MIDAS/Civil enables carrying out construction stage analysis, which is a prerequisite for cable stayed bridge analysis. Such analyses allow examining structural displacements, forces, stresses, etc. during construction.

MIDAS/Civil is also capable of carrying out analyses for traffic moving loads, response spectrum, time history, buckling that are applicable for a completed structure all within the same program. It contains a truss cable element, which is used to reflect geometric nonlinearity of a cable stayed bridge for both stage analysis and analysis for a completed structure.

3.4 Numerical Analysis Model of cable stayed bridge using MIDAS/Civil

3.4.1 Numerical Analysis Model

According to Midas /Civil / manual, the analysis model of a structure includes nodes, elements and boundary conditions. Finite elements are used in data entry, representing members of the Structure for numerical analysis and nodes define the locations of such members. Boundary conditions represent the status of connections between the structure and neighboring structures such as foundations.

Finite elements, accordingly, should be carefully selected so that they represent the real structure as closely as possible. This can be accomplished by comprehensive understanding of the elements’ stiffness properties that affect the behaviors of the real structure. Real structures generally comprise complex shapes and various material properties.

The elements and analysis types that will be used in this research are as follows:

1. Cable Element

Two nodes define a tension-only, three-dimensional line element, which is Capable of transmitting axial tension force only. A cable element reflects the change in stiffness varying with internal tension forces. The cable element as shown in Figure (3-1).

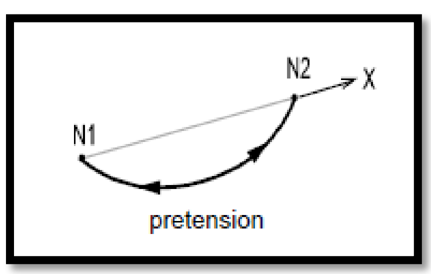


Figure (3-1): Schematics of a cable element

A cable element is automatically transformed into an equivalent truss element and an elastic catenary cable element in the cases of a linear analysis and a geometric nonlinear analysis respectively

- **Equivalent truss element**

The stiffness of an equivalent truss element is composed of the usual elastic stiffness and the stiffness resulting from the sag, which depends on the magnitude of the tension force. Expressions (3.1) to (3.4) calculate the stiffness for the equivalent truss element

$$K_{comb} = \frac{1}{{}^{1}/_{K_{sag}} + {}^{1}/_{K_{elastic}}} \quad (3.1)$$

$$K_{comb} = \frac{EA}{L\left[1+\frac{W^2\,L^2\,EA}{12\,T^3}\right]} \quad (3.2)$$

$$K_{elastic} = \frac{EA}{L} \quad (3.3)$$

$$K_{sag} = \frac{12T^3}{W^2L^3} \quad (3.4)$$

Where:

E: Modulus of elasticity A: cross-sectional area

L: Length W: weight per unit length($w = \gamma * A$)

T: Tension force ($T = \sigma * A$)

- **Elastic catenary cable Element**

The tangents stiffness of a cable element applied to a geometric nonlinear analysis is calculated as follows, Figure (3-2) illustrate a cable connected by two nodes where displacements $\Delta_{1,}, \Delta_2$ *and* Δ_3 occur at node I and Δ_4, Δ_5 *and* Δ_6 occur at node j the equilibrium of the nodal forces and displacements are expressed refereeing to Figure (3-2) as follows :

$F_4 = - F_1$

$F_3 = -F_2$

$F_6 = - F_3 - \omega_0 L_0$ ($\omega_0 = \omega$ assumed)……………………………. (3. 5)

$I_x = I_{X0} - \Delta_1 + \Delta_4 = f\ (F_1, F_2, F_3)$

$I_Y = I_{y0} - \Delta_2 + \Delta_5 = g\ (F_1, F_2 . F_3)$

$I_Z = I_{Z0} - \Delta_3 + \Delta_6 = h\ (F_1, F_2, F_3)$

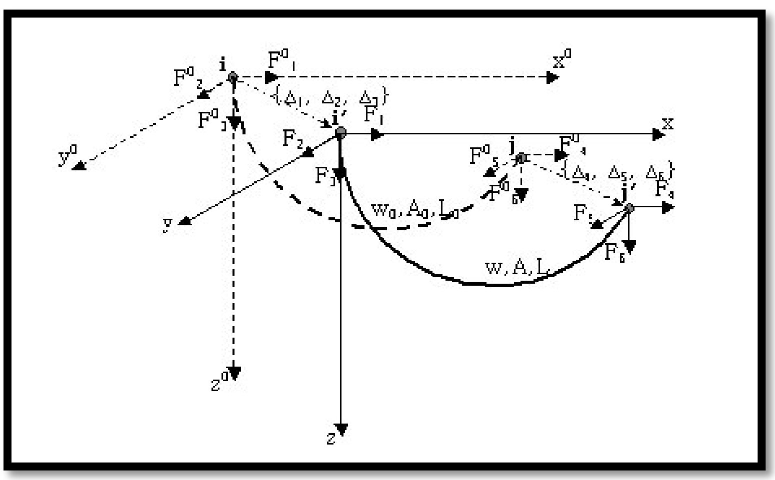


Figure (3-2): Schematics of tangent stiffness of an elastic catenary cable element

The differential equations for each directional length of the cable in the Global Coordinate System are noted as below:

$$dI_x = \frac{\partial f}{\partial F_1}\partial F_1 + \frac{\partial f}{\partial F_2}\partial F_2 + \frac{\partial f}{\partial F_3}\partial F_3$$

$$dI_Y = \frac{\partial g}{\partial F_1}\partial F_1 + \frac{\partial g}{\partial F_2}\partial F_2 + \frac{\partial g}{\partial F_3}\partial F_3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.6)}$$

$$dI_Z = \frac{\partial h}{\partial F_1}\partial F_1 + \frac{\partial h}{\partial F_2}\partial F_2 + \frac{\partial h}{\partial F_3}\partial F_3$$

When these load-displacement relations can be obtain the flexibility matrix, ([F]). The tangent stiffness, ([K]), of the cable can be obtained by inverting the flexibility matrix. The stiffness of the cable cannot be obtained immediately; rather repeated analyses are carried out until it reaches an equilibrium state. These equations are as follows:

$$\begin{Bmatrix} dI_x \\ dI_y \\ dI_z \end{Bmatrix} = \{F\}\begin{Bmatrix} dF_1 \\ dF_2 \\ dF_3 \end{Bmatrix}, \left[\{F\} = \begin{bmatrix} \frac{\partial f}{\partial F_1} & \frac{\partial f}{\partial F_2} & \frac{\partial f}{\partial F_3} \\ \frac{\partial g}{\partial F_1} & \frac{\partial g}{\partial F_2} & \frac{\partial g}{\partial F_3} \\ \frac{\partial h}{\partial F_1} & \frac{\partial h}{\partial F_2} & \frac{\partial h}{\partial F_3} \end{bmatrix} = \begin{bmatrix} f_{11} & f_{12} & f_{13} \\ f_{21} & f_{22} & f_{23} \\ f_{31} & f_{32} & f_{33} \end{bmatrix} \right], [[K] = [F^{-1}]] \ldots\ldots \text{(3.7)}$$

2. Linear Static Analysis

The basic equation adopted in MIDAS/Civil for linear static analysis is as in equation 3.8.

$$[K]\{U\} = \{P\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.8)}$$

Where:

[K]: Stiffness Matrix, {U}: Displacement vector and {P}: Load vector

MIDAS/Civil allows unlimited numbers of static load cases and load combinations.

3. Non-linear Analysis

The assumption of linear behaviors is valid in most structures. However, Non-linear analysis is necessary when stresses are excessive, or large displacements exist in the structure. Construction stage analyses for cable stayed bridges are some of the large displacement structure examples.

4. Boundary Conditions

Boundary conditions are distinguished by nodal boundary conditions and element boundary conditions.

1. Nodal boundary conditions are represented by

- Constraint for degree of freedom(support)
- Elastic boundary element (spring support)
- Elastic link element (Elastic Link)

2. Element boundary conditions are represented by

- Element end release
- Rigid end offset distance.
- Rigid link

5. Moving Load Analysis for Bridge Structures

The moving load analysis function in MIDAS/Civil is used to statically analyze and design bridge structures for vehicle moving loads. Important features are included as follows:

- Generation of influence lines and influence surface for displacements,

Member forces and reactions due to moving loads.

- Calculation of maximum/minimum nodal displacements, member forces and support reactions for a given moving vehicle load using the generated influence line and influence surface.

6. Analysis Options

When a structure is subjected to external loads, the corresponding structural response may exhibit material nonlinearity to a certain extent. However, in most

Structural analyses for design purposes, structures behave almost linearly provided that the member stresses remain within the limits of design codes. Material nonlinearity thus is rarely considered in practice.

MIDAS/Civil is formulated on the basis of material linear analysis, but it is capable of carrying out geometric nonlinear analyses. MIDAS/Civil implements nonlinear elements (tension or compression-only), P-Delta and large displacement analyses. The structural analysis features of MIDAS/Civil include basic linear analysis and Non-linear analysis in addition to various analysis capabilities required in practice. The following outlines some of the highlights of the analysis features:

- Linear Static Analysis
- Eigenvalue Analysis
- Nonlinear Static Analysis
- Pushover Analysis
- Construction Sequence Analysis
- Moving Load Analysis for bridges

3.5 Case study: General Information for Tuti - Bahri cable-stayed bridge:

3.5.1 Location of Bridge

Tuti Bahri Bridge is over the River Nile on one side of the Tuti Island. Based on the states recommended choice on the structural system and construction design this bridge is a cable-stayed bridge. Given the importance of the River Nile and to coordinate with the existing bridge on the other side of Tuti Island, a bridge with three spans is appropriate. See Figure (3-3) .

3.5.2 Description of Bridge

The bridge is of three spans, as shown in Figure (3-4). The total length of the bridge is 600 meters. So as to have large spans, the bridge is of a cable stayed system. The middle span is 300 meters and the end spans are 150 meters. The dimensions and material properties for the three span continuous cable-stayed Bridge [Tutti-Bahri] are as shown in Tables (3-1), (3-2), (3-3), (3-4) and (3-5) and figure (3-5).

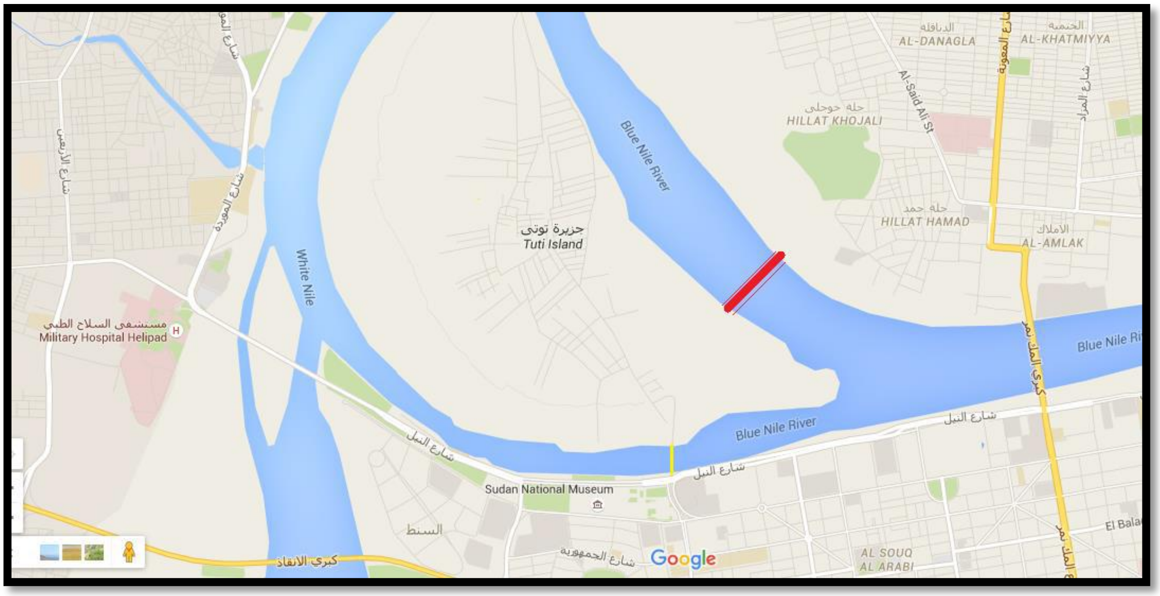


Figure (3-3): location of Tutti-Bahri Bridge. (Google map)

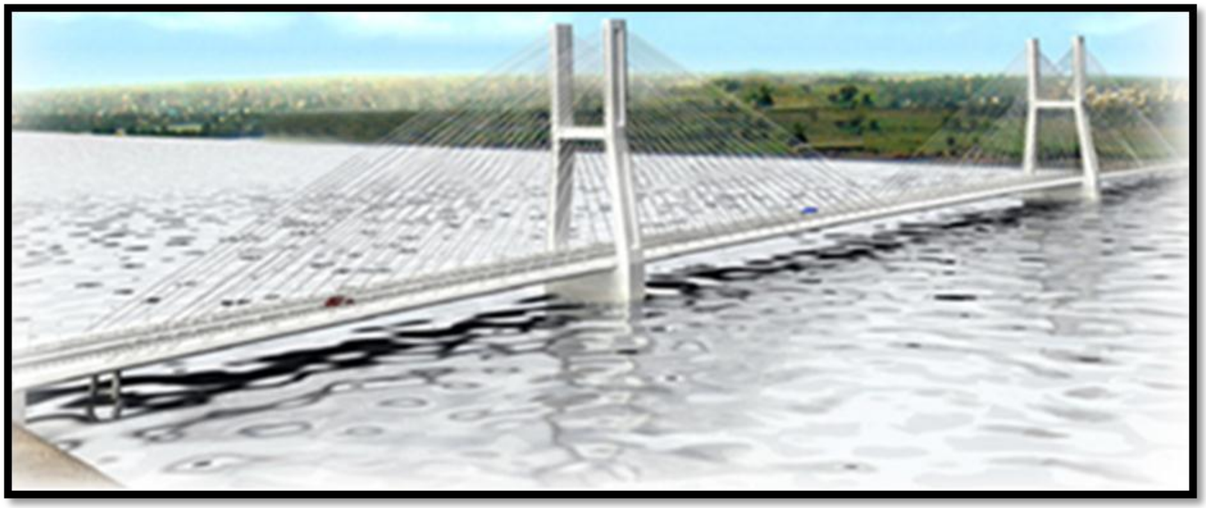


Figure (3-4): Tutti-Bahri cable –stayed bridge

(Ministry of Infrastructure and Bridge- Khartoum State)

Table (3-1): Dimensions of bridge (Ministry of Infrastructure and Bridge- Khartoum State)

Bridge type	Three span continuous cable-stayed bridge (self-anchored
Bridge length	L = 150 m+300 m+150 m = 600 m
Bridge Width	B = 24 m
Pylon type	H shape pylon
Cable arrangement	Semi-Fan
Number of pylons	2 towers
Deck type	Concrete deck
Depth of the deck slab	350mm
Height of pylon	73.8 m
Long girder	2.2 × 2m
Transvers girder	1.6 × 0.6m
Pylon cross beam	3 × 6 m

Table (3-2): Material Properties for strand stays and concrete.

For Strand Stay Cable Steel	**For Concrete**
Modulus of Elasticity = 197 GPa	Modulus of Elasticity = 2.76×10^{7} kN/m^2
Tensile strength = 1860 MPa	Concrete Strength, fcu = 24.5 kN/m^3
Poisson ratio, v = 0.3	Poisson ratio, v = 0.2
Density γ = 78.5 kN/m^3	Thermal coefficient = 5.0 × 10-6 ˚F
Normal diameter of strand =15.2 mm	
Thermal coefficient = 6.50E-06	

Table (3-3): Material Properties for New parallel-wire strands and Carbone fiber

For New parallel-wire strand PWS	**Carbone fiber**
Modulus of Elasticity = 200 GPa	Modulus of Elasticity = 159 kN/m^2
Tensile strength = 1860 MPa	Tensile strength = 2,421 MPa
Poisson ratio, v = 0.3	Poisson ratio, v = 0.3
Density γ = 82 kN/m^3	Density γ = 16 kN/m^3
Cross section 16 202mm^2	Normal diameter of strand =12.5 mm

Table (3-4): Section Properties (Ministry of Infrastructure and Bridge- Khartoum State)

No	Name	Area m^2	Ixx m^4	Iyy m^4	Izz m^4
1	Cable	0.0177	0	0	0
2	Long girder	4.4	2.698	1.775	1.467
3	Transvers girder	0.96	0.088	0.205	0.029
4	Pylon column J-J	24	75.125	32	72
5	Pylon column A-A	18	37.079	13.5	54
6	Pylon column B-B	13.8	27.641	12.814	50.85
7	Pylon girder D-D	18	37.079	13.5	54
8	Pylon girder C-C	11.2	27.497	11.862	44.933
9	Pylon column E-E	15.9	49.639	27.079	65.925
10	Pylon column F-F	18.9	67.139	39.919	74.925
11	Pylon column G-G	27	98.408	45.563	81

Table (3-5): Loading data of the model

Name	Load type	Load
Dead load	Self- weight	Automatically calculated within the program
Additional dead load	Additional dead load (pavement, railing and parapets)	[99 kN/m]
Cable pretension	Pre- tension load	1 KN
Moving load:	Vehicular load type: HL-93TRK - HS20(AASHTO LRFD2010)	C 3.6.1.1.1.
Wind load	AASHTO LRFD 2010	

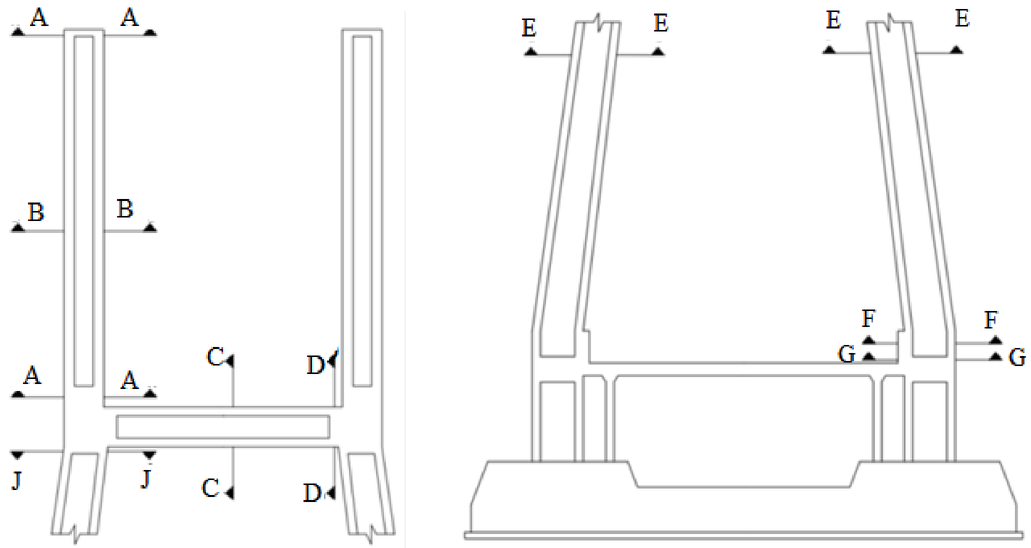


Figure (3-5): Tutti-Bahri Tower Dimensions.

3.5.3 Initial Cable pre- tension Calculation

The initial cable pre- tension, which is balanced with dead loads, is introduced to improve section forces in the main girders and towers, and cable tensions and support reactions in the bridge. Many iterative calculations are required to obtain initial cable pre- tension forces because the cable-stayed bridge is a highly indeterminate structure. And there are no unique solutions for calculating cable Pre- tension directly. Each designer may select a different initial pre- tension for an identical cable-stayed bridge.

Gimsing (1994), developed easy hand-formulae to predict the cable forces by taking the stiffness of the girder and pylon into consideration.

The moment and displacement distribution along the girder and the pylon can reach the ideal state by adjusting the cable stresses. Using vector and matrix calculations, the moment or the displacement of an ideal state I can be written as:

$I = [i_1 \quad i_2 \cdots \quad i_n]^{\mathrm{T}}$ ……………………………………………………..(3.9)

n is the total number of the targets that need to be satisfied and T stands for the transformation of a matrix or a vector. The result cable stresses S can be written as

$S = [s_1 \quad s_2 \cdots \quad s_m]^{\mathrm{T}}$ ……………………………………………………....(3.10)

In which m is the number of cables to be adjusted.

By analyzing the response of the unit pre-tension applied to each tuning cable, the influence values of all the targets can be obtained. The influence matrix T can be written as:

$$T = \begin{bmatrix} t_{11} & t_{12} \cdots & t_{1m} \\ t_{21} & \cdots \cdots & \cdots \\ t_{n1} & t_{n2} \cdots & tnm \end{bmatrix} \text{……………………………………...………………(3.11)}$$

where t_{nm} is the response at the target n by pre-tension the unit stress at cable m. Thus, their relation can be set down as n the total number of cables.

$T * S = I$ …………………………………………………………………....(3.12)

If the number of cables that are to be tuned is the same as the number of targets, the setting I to the designated target values, the cable stresses S can be obtained accordingly by solving the above Equation .

The Influence matrix D, which expresses the influence value of displacement due to each unit shim thickness, is given as:

$$D = \begin{bmatrix} d_{11} & d_{12} \cdots & d_{1m} \\ d_{21} & \cdots \cdots & \cdots \\ d_{n1} & d_{n2} \cdots & d_{nm} \end{bmatrix} \text{…………………………………………………….(3.13)}$$

Where d_{nm} is the displacement increment at section n and n the total number of cables, the following equation can be obtained

$D * M = \Delta\delta$…………………………………………………………………(3.14)

Where $\Delta\delta$ is the effect on the deflection. If the deflection of the girder is different from the required value by $\Delta\delta$, the required shim thickness M gives – $\Delta\delta$

$D * M = -\Delta\delta$………………………………………………………………..(3.15)

Bruer and Pircher (1999), favor a numerical approach to reduce the calculation effort, the Unit Load Method. For the final stage structure including its total dead load, unit load cases as well as the ideal moment diagram must be defined. Commonly, the selected unit forces are:

- A unit shortening of the cable or a unit tensioning causing an axial cable shortening
- A unit translation of a rigid support or an element. A transverse or longitudinal Movement changes the moments in the deck by changing the cable forces which act on the deck.

The same number of unit loading cases must be defined as the number of “Fixed Moment” points, chosen on the structural model to represent the ideal moment diagram. Figure (3-6), illustrates this procedure.

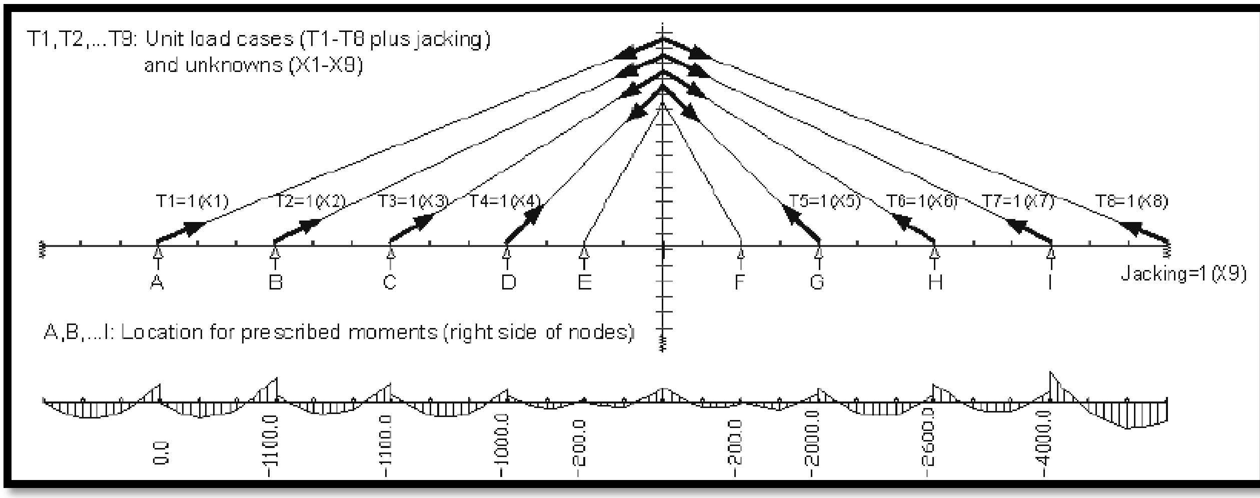


Figure (3-6): Unit Load Case Method for determining the ideal state 0

The ideal dead load bending moment diagram is defined for the deck girder. As shown in the figure above, the bending moments for nine points along the girder are described from position A to I. for example, Nine unit load cases are selected for setting up the simultaneous equations – eight unknown stay cable forces and one unit translation at the end support.

In this example, the equation system follows to be:

$$M_A = M_P + M_{T1=1} * x_1 + \cdots + M_{TS=1} * x_s \qquad (3.16).$$

$$M_I = M_P + M_{T1=1} * x_1 + \cdots + M_{TS=1} * x_s$$

Where M_A to M_I is the final stage moment at positions A to I including tensioning and movement at the end supports, M_P the permanent load moment at the current position without cable tensioning and support movement, $M_{T1=1}$ to $M_{TS=1}$ the bending moments due to each unit tensioning at position A to I, and MJ the bending moment due to unit jacking of the end support.

The solution of the equations, the unknown xi, is the factor by which the unit loads must be multiplied to achieve the defined moment distribution. This basic solution defines the cable forces and the jacking force for the final stage.

Many analysis programmers use an optimization method for determining the cables forces. Under permanent loads, a criterion (objective function) is chosen in a way that the internal forces, mainly the bending moments, are evenly distributed and small. The deflection of the structure can be limited to prescribed tolerances, too.

The analysis programmer Midas also provides the Unknown Load Factor function, which is based on an optimization technique. Similar to the described method above, this can be used to calculate the load factors that satisfy specific boundary conditions (constrains) defined for a system.

3.5.4 Unknown Load Factor Optimization

In the cable- stayed bridges the permanent state of stress under the dead load is determined by the tension forces. These are introduced to reduce the support reactions and bending moment in the main girder and tower in the bridge structure to the minimum values or at least to reduce these as much as possible. Hence, the deck and tower would be mainly under compression under dead load. The analysis program MIDAS Civil provides the unknown load factor function, which is based on an optimization technique. It can be used to calculate the optimum load factors that satisfy specific boundary conditions defined for a system.

To determine the unknown load factors for each cable stay to achieve an ideal state, a unit pretension load is applied for each cable. Performing a linear analysis, the programmer computes the influence on the structure due to each unit tension load. In the Unknown Load Data the unit load cases are then defined as an Unknown Load. Furthermore, the structural restrictions for e.g. moment or displacement values, which are to be realized through the load factors in the combined load case, must be defined.

The initial cable pre-tension forces are obtained by the unknown load factor optimization function and the initial equilibrium state analysis of a complete cable–stayed bridge. Furthermore, the structural restrictions for example vertical displacement or moment values, which are to be realized through the load factors in the combined load case, must be defined. Figure (3-7).Shows the steps that are carried out to generate the unknown load factors.

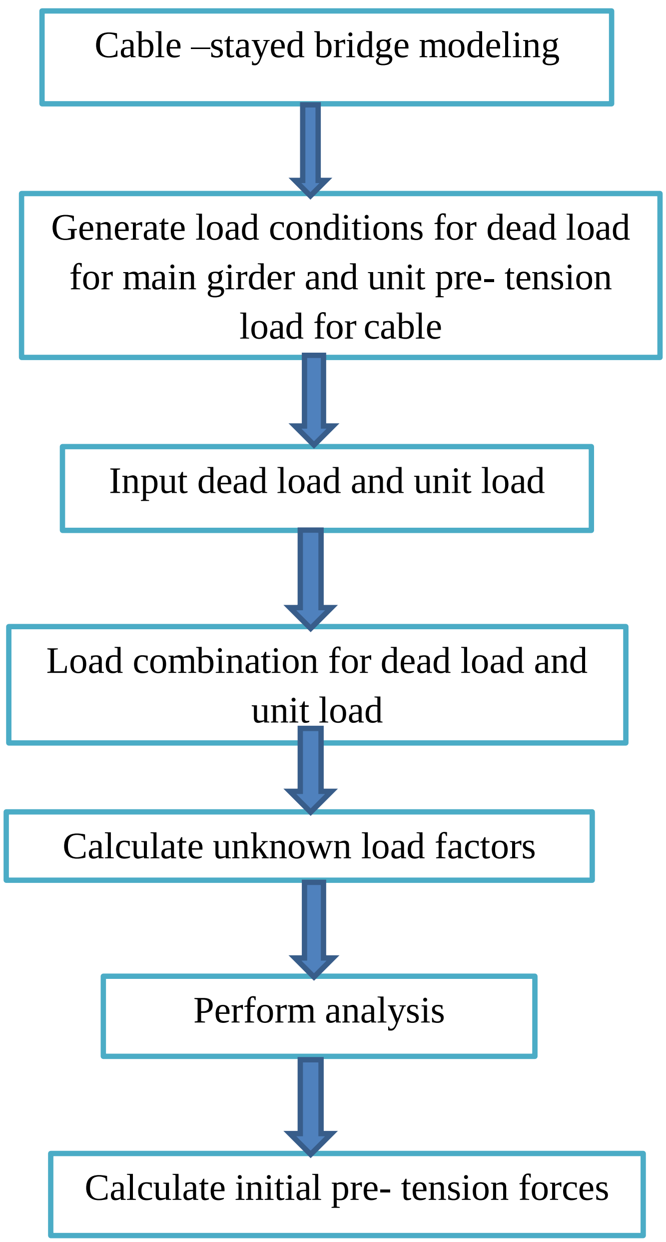


Figure (3-7): Flowchart for Initial Cable Pre-tension Calculation

3.5.5 Static analysis of the Cable-Stayed Bridge Model

The bridge components like deck, pylon, and cables have been modeled as per the actual forces they are subjected to. Cables are modeled as the truss element (160 elements); pylon and deck are modeled as the elastic beam elements (356 elements). The model is first analyzed for the dead loads by static analysis to get the deformed configuration, because the target of static analysis is to get the initial deformed shape of the cable stayed bridge. Deformation under the self-weight of the structure should be small. The procedure of calculating initial pre-tension for cable-stayed bridges by Unknown Load Factor is presented in detail in the following section

3.5 5.1.Procedure of calculating initial pre-tension for cable-stayed bridges

The steps for calculation the initial pre-tension force for cable- stayed bridge are as follows:

Step (1): define the unit system

To perform the final stage analysis of cable-stayed bridge, a new file was opened and saved as ‘Cable -Stayed model ’, modeling was started by assigning ‘m’ to the unit of length and ‘kN’ to the unit of force. This unit system can be changed at any time during the modeling process for user’s convenience. This step was defined following, the steps shown in Appendix B, Figure (B-3).

Step (2): Definitions of Material and Section Properties

The material properties of the cables, main girders, towers, cross beams between the main girders and tower were entered using the steps shown in Appendix B, Figures (B-4).

The section properties for the cables, main girders, towers (pylons), cross beams between the main girders and tower were entered, using the steps as shown in Appendix B, Figure (B-5).

Step (3): Cable Stayed Bridge Wizard:

After entering the material and section properties for all elements, Cable Stayed Bridge Wizard automatically generated the symmetrical cable stayed bridge model including, truss with a specified profile. The steps for this are shown in Appendix B, Figure (B-6).

Step (4): Loading Condition Input

The loading conditions for self-weight, superimposed dead load and unit loads for cables were entered to calculate the initial pre-tension for the dead load condition. the number of required unknown initial cable pretension values were set at 40, as the bridge is a symmetric cable-stayed bridge, which had 40 cables on each side of each tower. The loading conditions for each of the 40 cables were entered. The loading condition input steps for cable are shown in Appendix B, Figure (B-7).

Step (5): Loading Input

The self-weight, superimposed dead load for the main girders and unit loads for the cables is entered, after that inputted the superimposed dead load that includes the effects of barriers, parapets and pavement, and unit pretension loads for the cable elements for which initial cable pretension is calculated. The loading input steps for main girder and unit loads for cables are shown in Appendix B (Figure B-8).

The superimposed dead loads for the main girders were specified and the superimposed dead loads divided and loaded for the two main girders. The superimposed dead load equal –99.7 kN/m, which is due to barriers, pavement, etc was input by the Element Beam Loads function. The steps for these are shown in Appendix B (Figure B-9).

A unit pretension load to each cable was input. For the case of a symmetric cable-stayed bridge, identical initial cable pretension was introduced to each of the corresponding cables symmetrically to the bridge center. As such, the identical loading conditions to the cable pairs were input to form the symmetry. the steps for this are shown in Appendix B (Figure B-10).

Step (6): Boundary conditions input

Boundary conditions for the analytical model were as follows:

• Tower base, Pier base: Fixed condition displacement and rotations (Dx, Dy, Dz, Rx, Ry, Rz)

• Connections between Main Girders and Bearings were Rigid Link (Dx, Dy, Dz, Rx, Ry, Rz). the boundary conditions were input for the tower and pier bases. The steps for these are shown in Appendix B, (Figure B-11 to Figure B-13)

Step (7): Perform analysis

The static analysis for self-weight, additional load and unit pre- tension in the cables was performed, using the step shown in Appendix B, (Figure B-14)

Step (8) Load Combinations for Dead Loads and Unit Loads

The load combinations were created using the 40 loading conditions for cable unit pretension loading, self-weights and superimposed dead loads the steps for these are shown in Appendix B, (Figure B-15). The following combinations were used when evaluating cable pre- tensioning.

.LCB1= Dead loads (self-weight + additional load) +Unit load pre tensioning

LCB2 = Dead loads (self-weight + additional load) +cables pre tensioning force.

Step (9) Unknown Load Factors Calculation

The unknown load factors that satisfy the boundary conditions were calculated by the Unknown Load Factor Function for LCB1.These were generated through load combinations. The constraints were specified to limit the vertical deflection (Dz) of the girders. The load condition, constraints and method of forming the object function in Unknown Load Factor were specified. For more clarification, shown the steps for specification of these are shown in Appendix B (Figure B-16 and Figure B-17)

3.5.6 Moving load condition

In order to carry out a moving load analysis, vehicle loads, traffic lanes or traffic surface lanes and the method of applying the vehicle loads are defined. Then unit loads are applied at various points to traffic lanes or traffic surface lanes to calculate influence lines or influence surface.

Moving load analysis in this study was performed by using AASHTO LRFD 2010 section 3.6.1.2. The vehicles were generated and applied in the existing lanes following the guidelines from AASHTO LRFD 2010.Moving load generation in

MIDAS civil is (a) Traffic line lanes (b) Vehicle load (c) Moving load application the vehicles were applied to the lanes using the vehicle classes. The steps of moving load analysis are as shown in the following:

Step (1): Select the moving load analysis data following the steps shown in Appendix B, (Figure B-18).

Step(2): Define standard vehicle load

The method for defining the moving traffic loads, HL93-TRK and HL93-TDM is

as shown in steps in Appendix B,(Figure B-19).

Step (3): Define moving load case using the steps shown in Appendix B, (Figure B-20).

Step (4): moving load analysis control data is input using these steps shown in Appendix B, (Figure B-21).

Step (5): Perform Structural Analysis using these steps shown in Appendix B, (Figure B-22).

Step (6): load combination

The following combinations were used in evaluating the cable pre- tensioning. Following the steps shown in Appendix B (Figure B-23).

LCB1= Dead loads (self-weight + additional load) +Unit load pre tensioning

LCB2 = Dead loads (self-weight + additional load) +cables pre tensioning force

LCB3 = Dead loads (self-weight + additional load) + cables pre tensioning force +live loads (moving loads)

3.5.7 Construction Stage Analysis

To design a cable-stayed bridge, its construction stages should be defined to check the stability during construction. The structural system could change significantly based on the erection method, and the change of system during construction can result in more critical condition for the structure compared to the state of the final stage. As such, an accurate construction stage analysis should be performed for designing a cable-stayed bridge to check the stability and to review stresses for the structure.

Construction stage analysis for a cable-stayed bridge can be classified into forward analysis and Backward analysis, based on the analysis sequence. Forward analysis reflects the real construction sequence. Whereas Backward analysis is performed from the state of the finally completed structure for which an initial equilibrium state is determined, and the elements and loads are eliminated in reverse sequence to the real construction sequence

To perform a construction stage analysis, construction stages were defined to consider the effects of the activation and deactivation of main girders, cables, cable anchorage, boundary conditions, loads. Each stage was defined to represent a meaningful structural system, which changes during construction. Each stage was defined to represent a meaningful structural system. Considering half the bridge for symmetrically arranged cables:

Number of stages =

Final stage (BCS0) Removal + Superimposed dead load removal + Removal of m girders in turn + Removal of n cables in turn …………..……..……. (3.17)

Where: $m = \frac{n}{2} + 1$.

For the case study bridge:

$m = 40 \qquad \therefore n = 21$

Number of stages = 1 + 1 + 21 + 40 = 63 stages.

Numbered from : BCS0 to BCS62.

The variation of cable tension forces was checked for each construction stage for inner cables in the tower area from the final stage (BCS0) to the first stage (BCS62). Construction stage (BCS62) was the stage in which all the cable elements and main girders in the center span were eliminated, and the temporary bents in the side spans were erected. Actually, this is the 1st stage in the cable-stayed bridge construction.

The cable pretension, which is introduced during the construction of a cable-stayed bridge, was calculated by backward analysis from the final stage. Figure (3-8) shows the cables numbers taking into account symmetry.

The analytical sequence of Backward Construction Stage analysis of the model is as shown in Table (3- 6) and these construction sequences of different stages are shown in Figure (3-9).it can be noted in this Table and Figure, that the cable – stayed bridge under construction has been divided into 62 stages for the different stages of construction, based in the analysis in construct to the construction stages from the final stage to the first, where the girder and cables were eliminated according to the stage, for example in the construction stage BCS0 is considered the final stages of construction, in the construction stage BCS2 where girder no 21 is eliminated and then the construction stage BCS3 where cable no 40 is eliminated, and so the process continues until the first stage, which is construction stage BCS62,which is considered the first stage of construction.

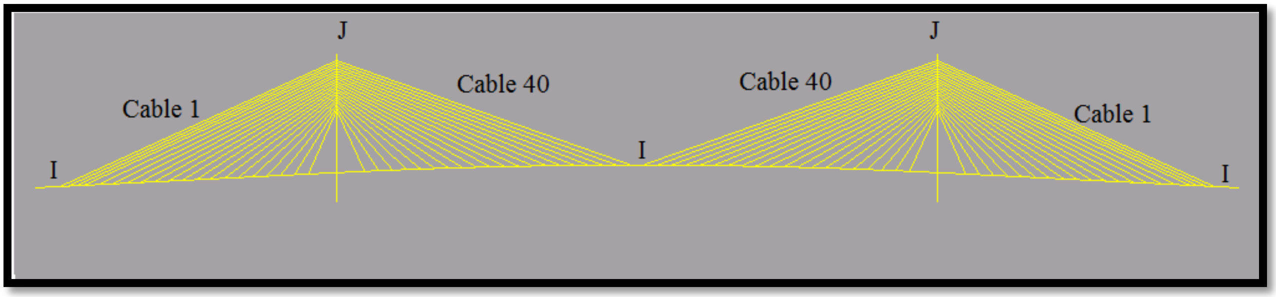
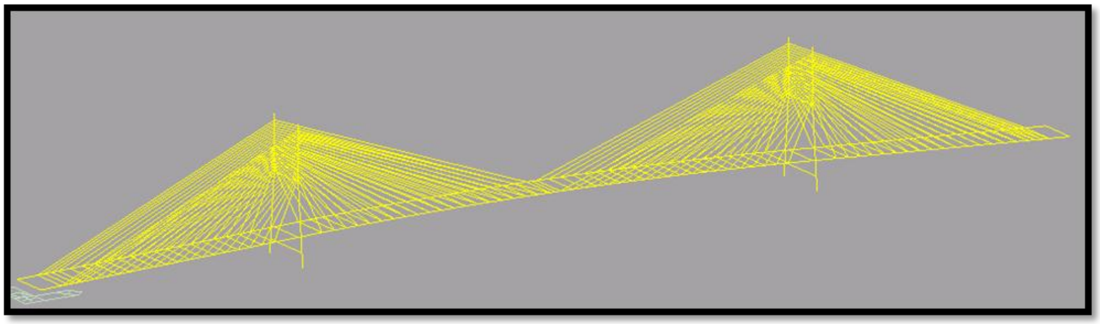


Figure (3-8): Cables numbers

Table (3-6): Analytical sequence of backward construction stage

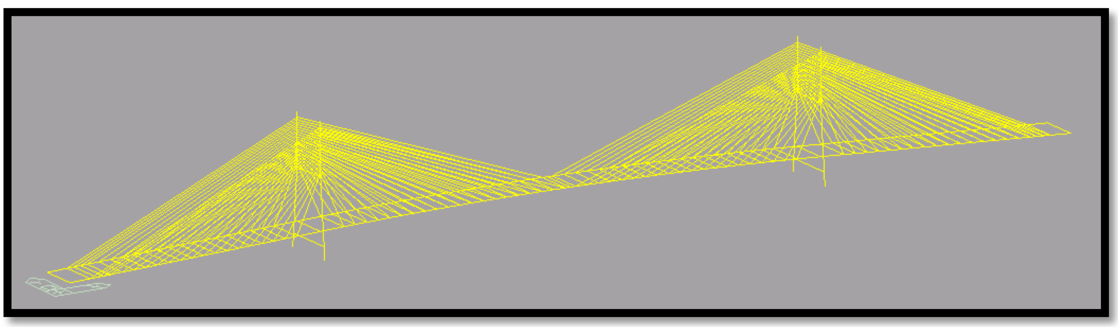
Stage	Content	Stage	Content
BCS0	Final Stage (Dead Load Superimposed Dead Load Initial Pre-tension	BCS32	Main Girder (11) removal
BCS1	Superimposed Dead Load removal	BCS33	Cable (30) removal
BCS2	Apply Temporary Bents & Key Segment removal (Main Girder No. 21)	BCS34	Cable (11) removal
BCS3	Cable (40) removal	BCS35	Main Girder (10) removal
BCS4	Cable (1) removal	BCS36	Cable (29) removal
BCS5	Main Girder (20) removal	BCS37	Cable (12) removal
BCS6	Cable (39) removal	BCS38	Main Girder (9) removal
BCS7	Cable (2) removal	BCS39	Cable (28) removal
BCS8	Main Girder (19) removal	BCS40	Cable (13) removal
BCS9	Cable (38) removal	BCS41	Main Girder (8) removal
BCS10	Cable (3) removal	BCS42	Cable (27) removal
BCS11	Main Girder (18) removal	BCS43	Cable (14) removal
BCS12	Cable (37) removal	BCS44	Main Girder (7) removal
BCS13	Cable (4) removal	BCS45	Cable (26) removal
BCS14	Main Girder (17) removal	BCS46	Cable (15) removal
BCS15	Cable (36) removal	BCS47	Main Girder (6) removal
BCS16	Cable (5) removal	BCS48	Cable (25) removal
BCS17	Main Girder (16) removal	BCS49	Cable (16) removal
BCS18	Cable (35) removal	BCS50	Main Girder (5) removal
BCS19	Cable (6) removal	BCS51	Cable (24) removal
BCS20	Main Girder (15) removal	BCS52	Cable (17) removal
BCS21	Cable (34) removal	BCS53	Main Girder (4) removal
BCS22	Cable (7) removal	BCS54	Cable (23) removal
BCS23	Main Girder (14) removal	BCS55	Cable (18) removal
BCS24	Cable (33) removal	BCS56	Main Girder (3) removal
BCS25	Cable (8) removal	BCS57	Cable (22) removal
BCS26	Main Girder (13) removal	BCS58	Cable (19) removal
BCS27	Cable (32) removal	BCS59	Main Girder (2) removal
BCS28	Cable (9) removal	BCS60	Cable (21) removal
BCS29	Main Girder (12) removal	BCS61	Cable (20) removal
BCS30	Cable (31) removal	BCS62	Main Girder (1) removal
BCS31	Cable (10) removal		

- Final Stage (Dead Load, Superimposed Dead Load and Initial Pre-tension)



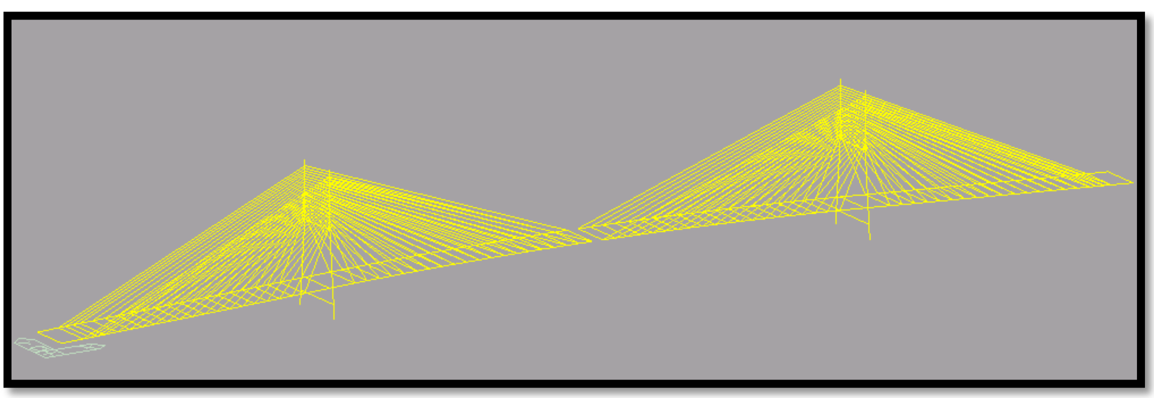
1. Construction stage BCS0

- Superimposed Dead Load Removal



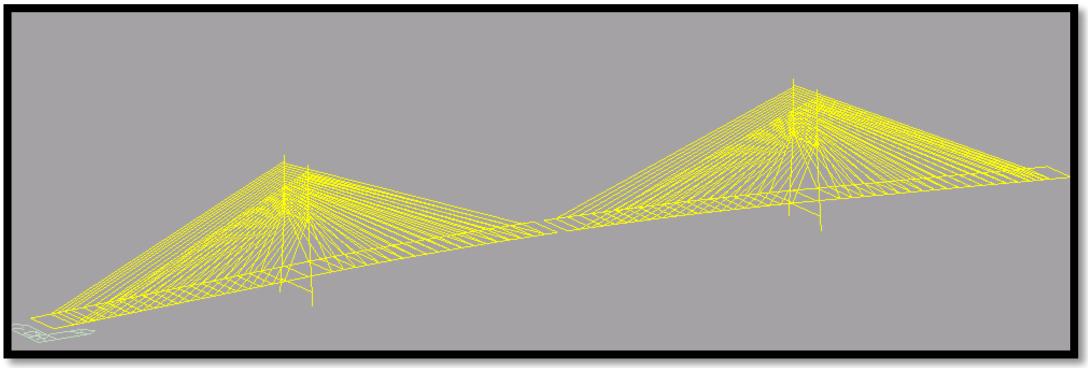
2. Construction stage BCS1

- Apply Temporary Bents & Key Segment removal (Main Girder No. 21)



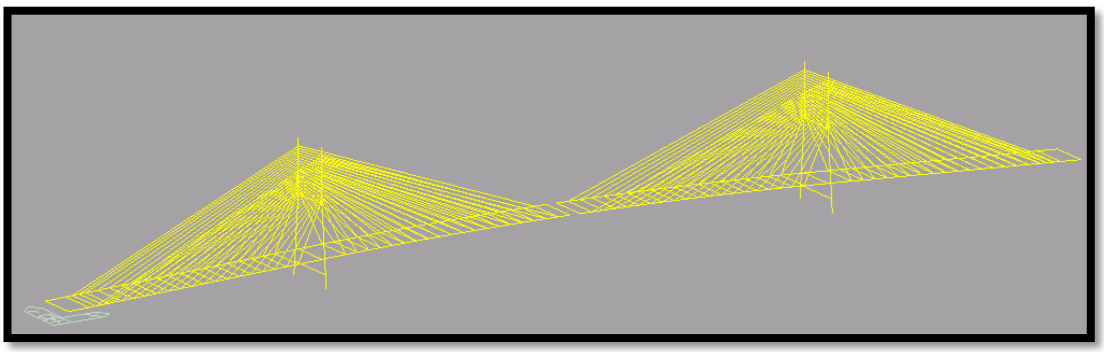
3. Construction stage BCS2

- Cable (40) removal



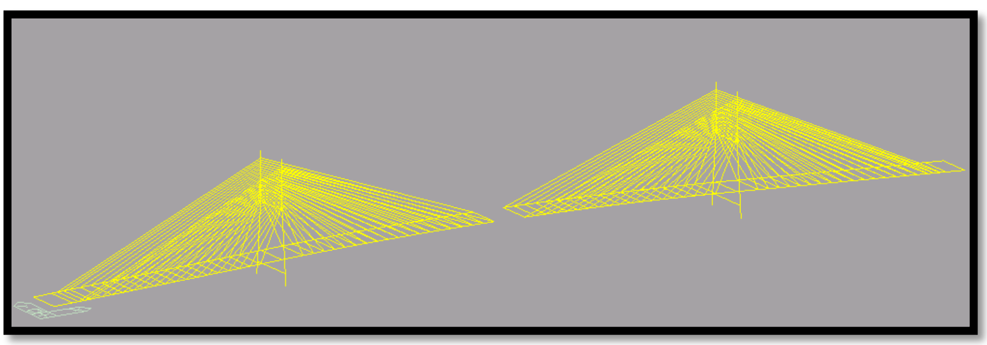
4. Construction stage BCS3

- Cable (1) removal



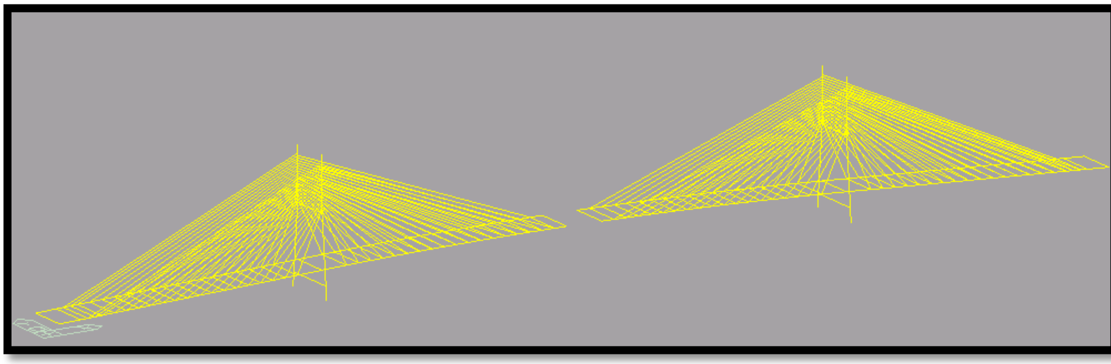
5. Construction stage BCS4

- Main Girder (20) removal



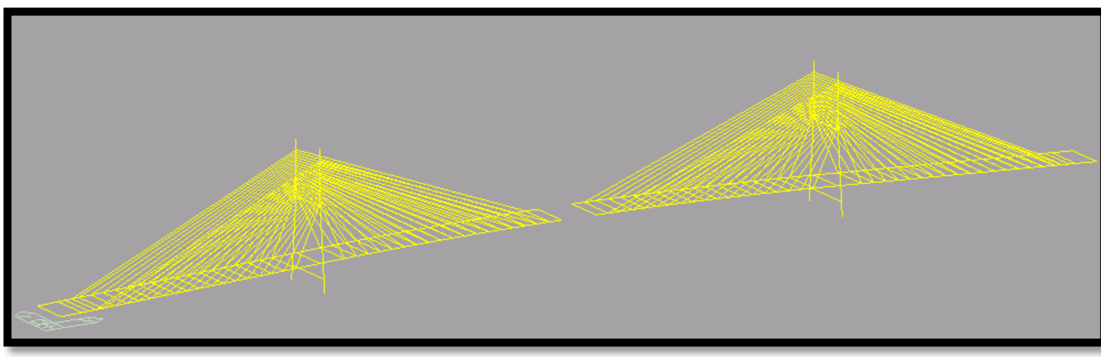
6. Construction stage BCS5

- Cable (39) removal



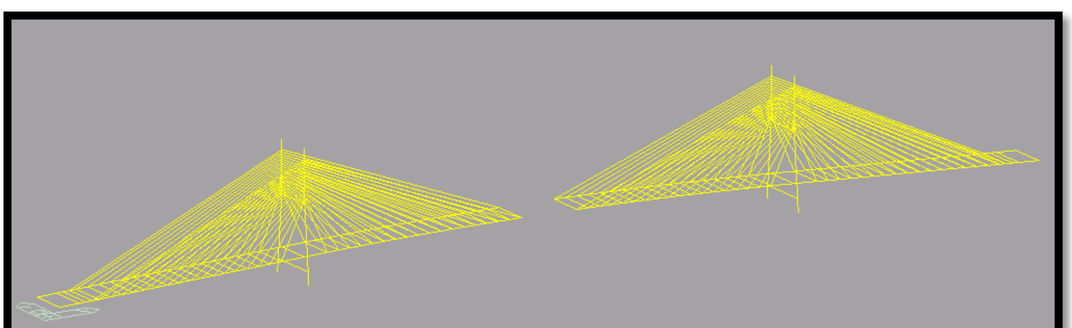
7. Construction stage BCS6

- Cable (2) removal



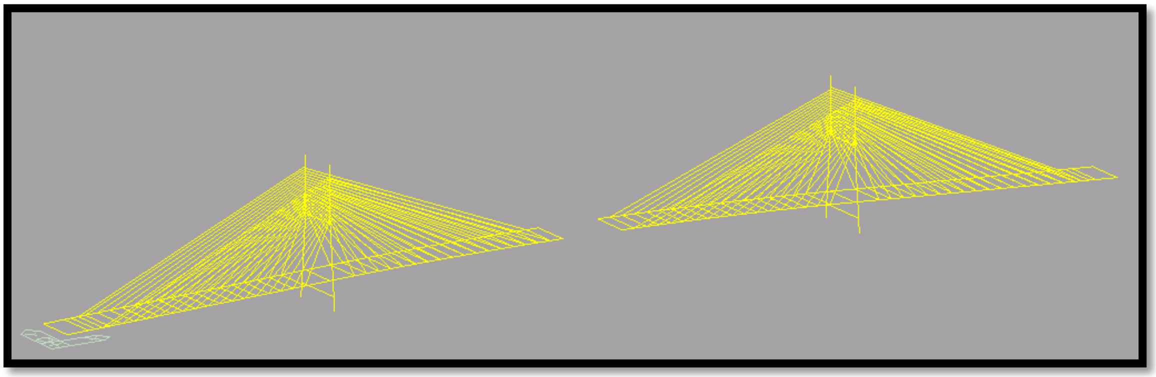
8. Construction stage BCS7

- Main Girder (19) removal



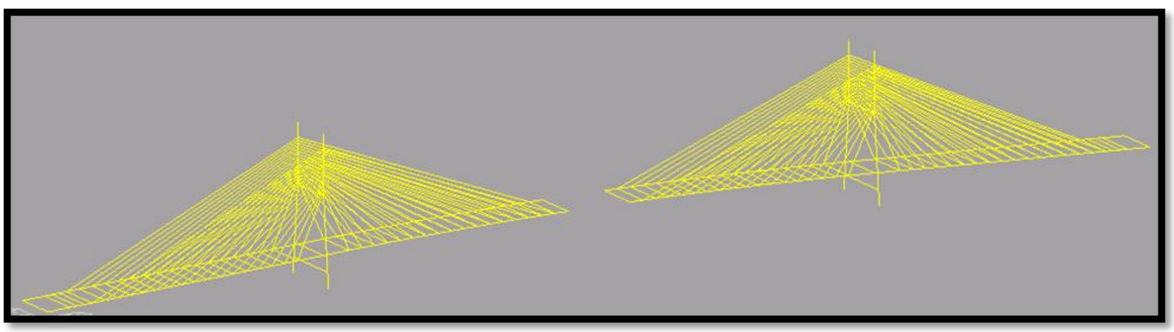
9. Construction stage BCS8

* Cable (38) removal



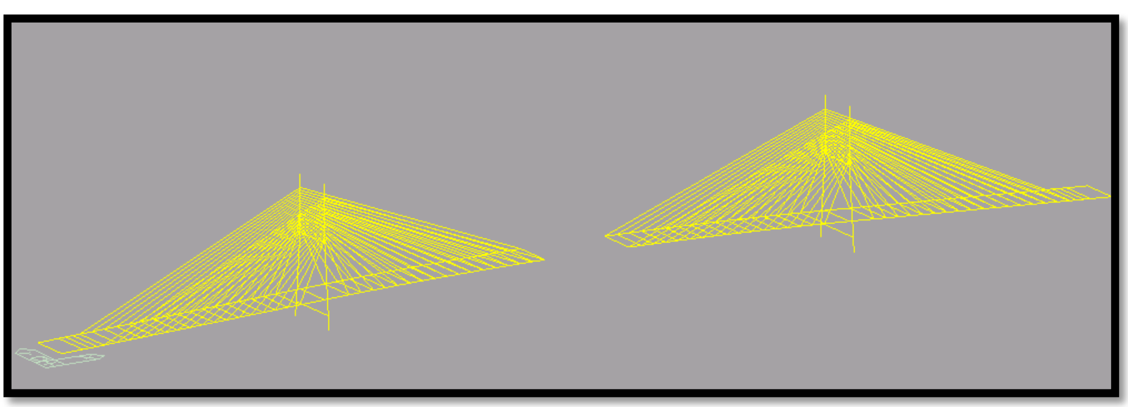
10. Construction stage BCS9

* Cable (3) removal



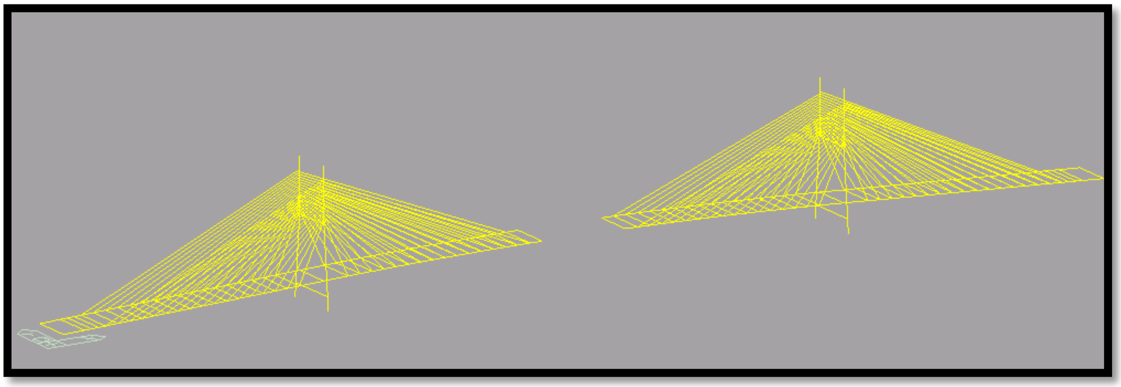
11.Construction stage BCS10

* Main girder (18) removal



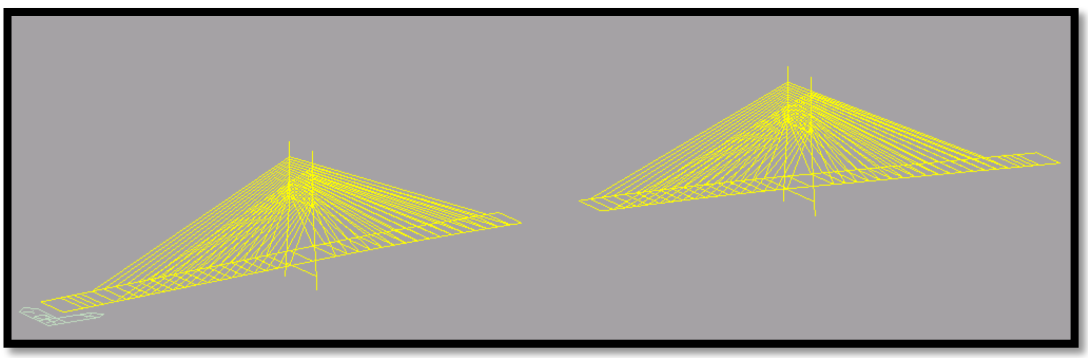
12. Construction stage BCS11

- Cable (37)removal



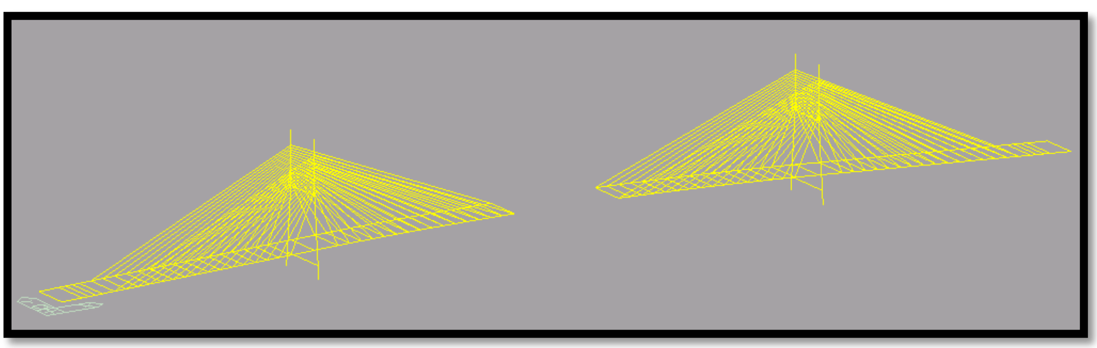
13. Construction stage BCS12

- Cable (4) removal



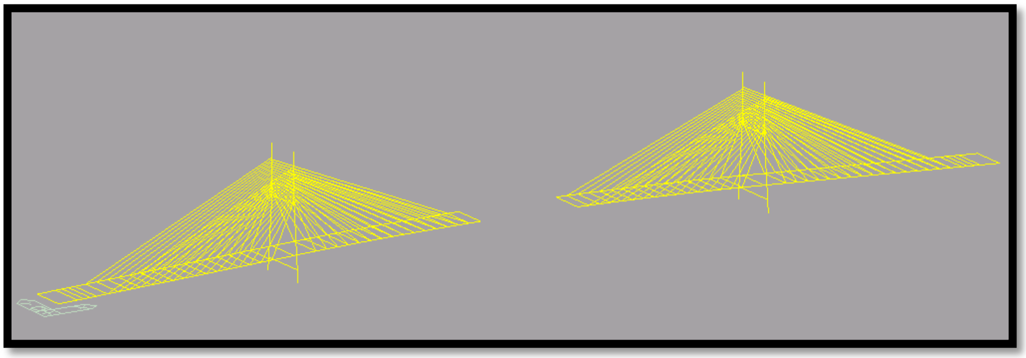
14. Construction stage CS13

- Main girder(17) removal



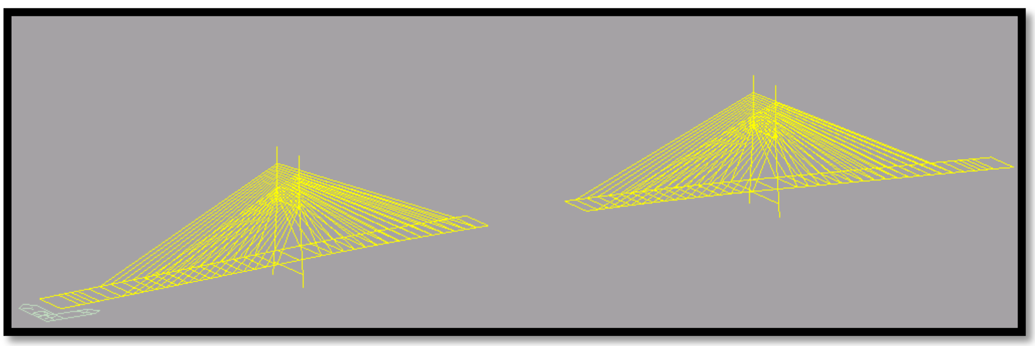
15. Construction stage CS14

- Cable (36) removal



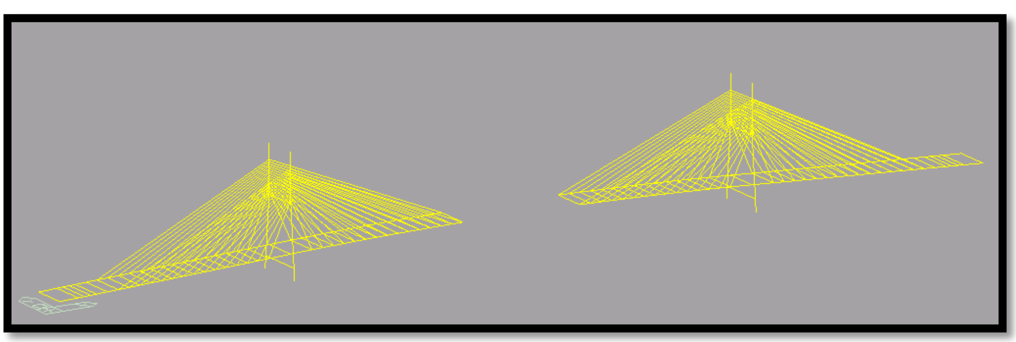
16. Construction stage BCS15

- Cable (5) removal



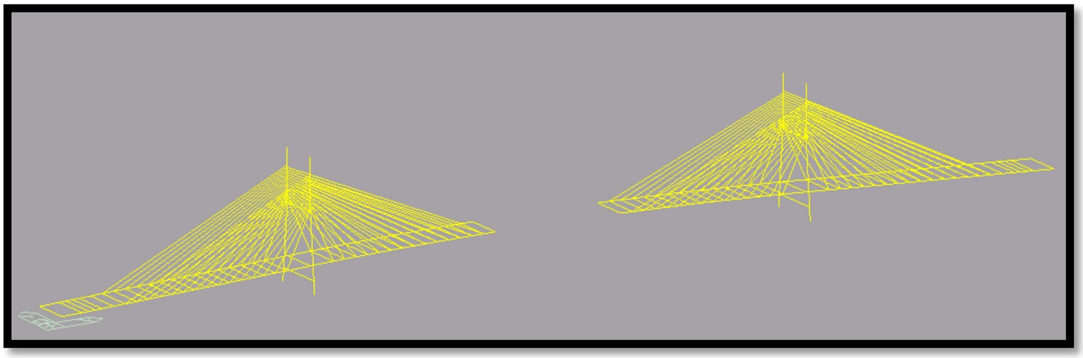
17. Construction stage BCS16

- Main girder (16) removal



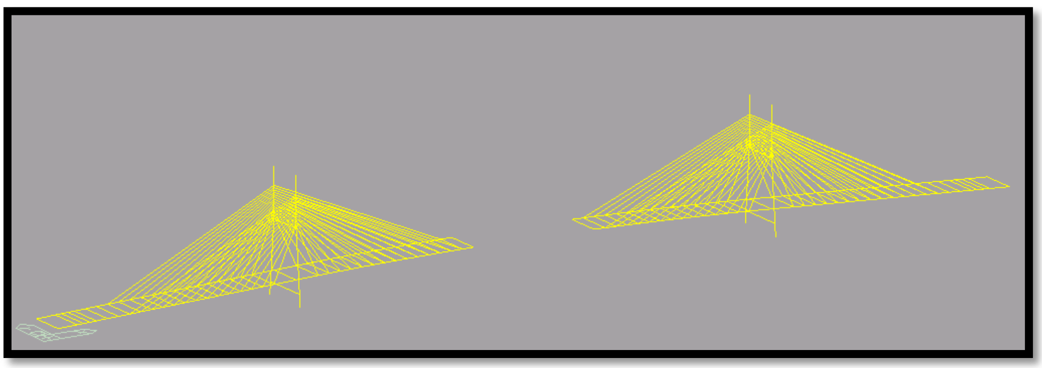
18. Construction stage BCS17

- Cable (35) removal



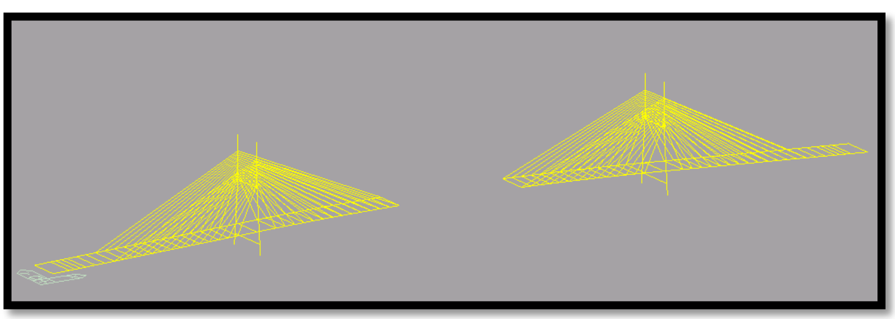
19. Construction stageBCS18

- Cable (6) removal



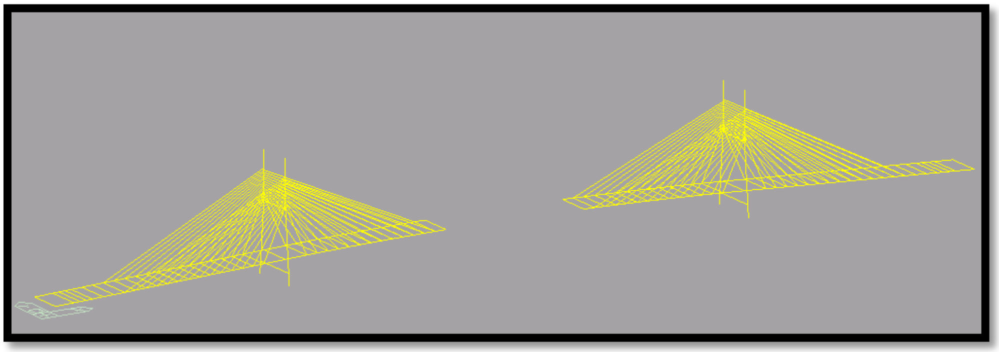
20. Construction stageBCS19

- Main girder(15) removal



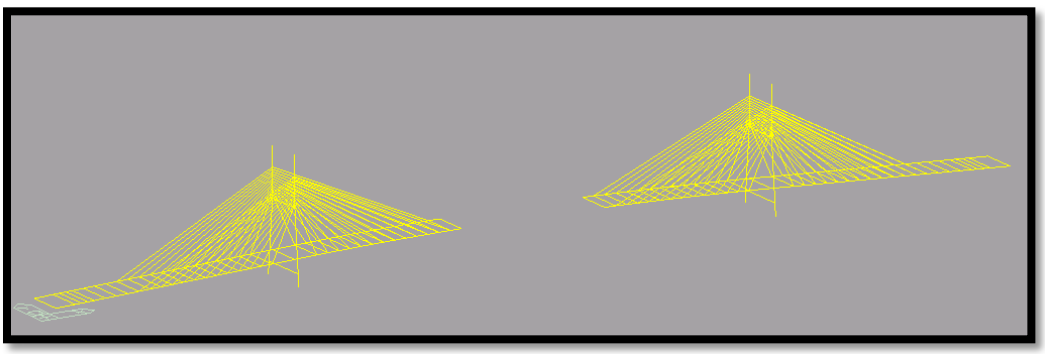
20. Construction stageBCS20

- Cable (34) removal



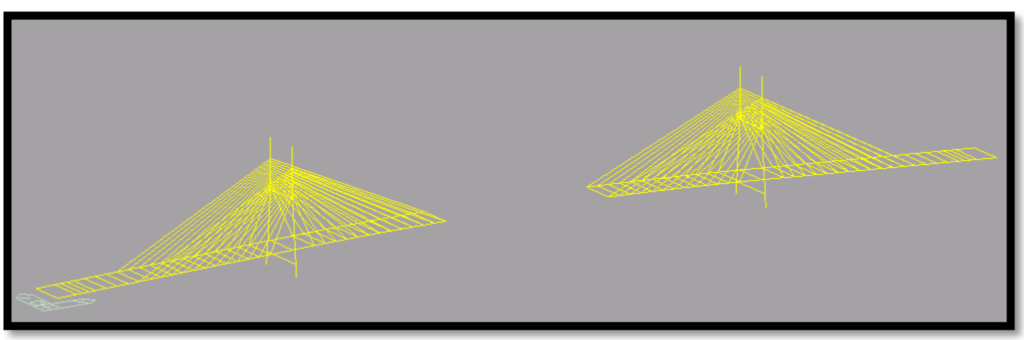
22. Construction stageBCS21

- Cable (7) removal



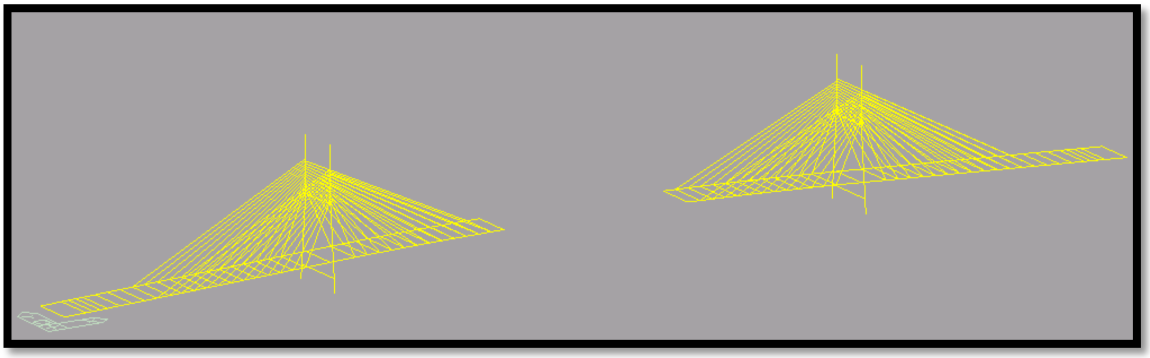
23. Construction stageBCS22

- Main girder (14) removal



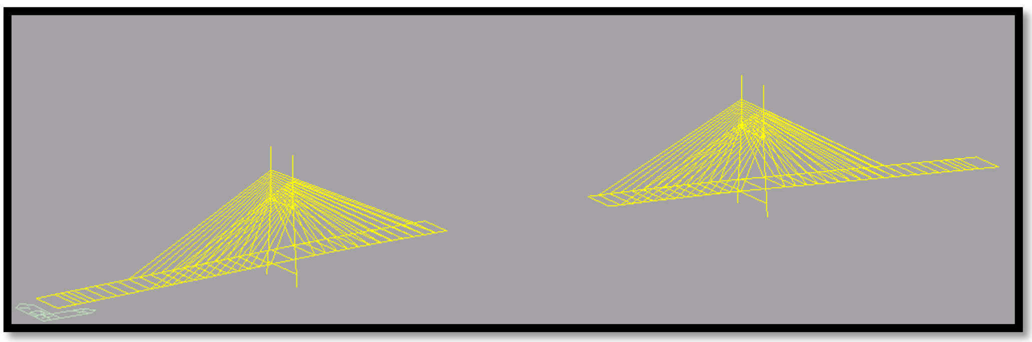
24. Construction stageBCS23

- Cable (33) removal



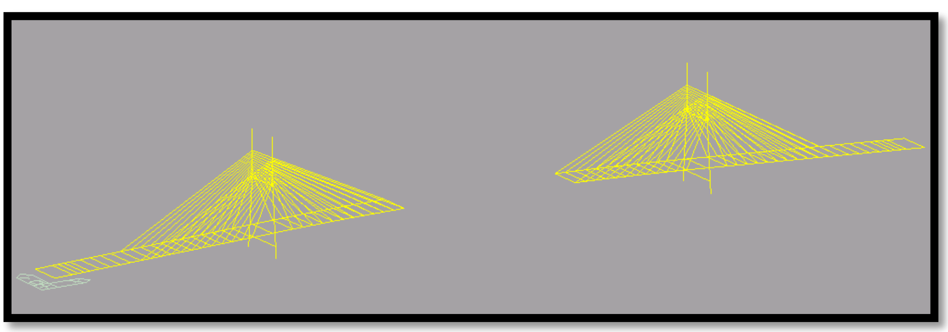
25. Construction stageBCS24

- Cable (8) removal



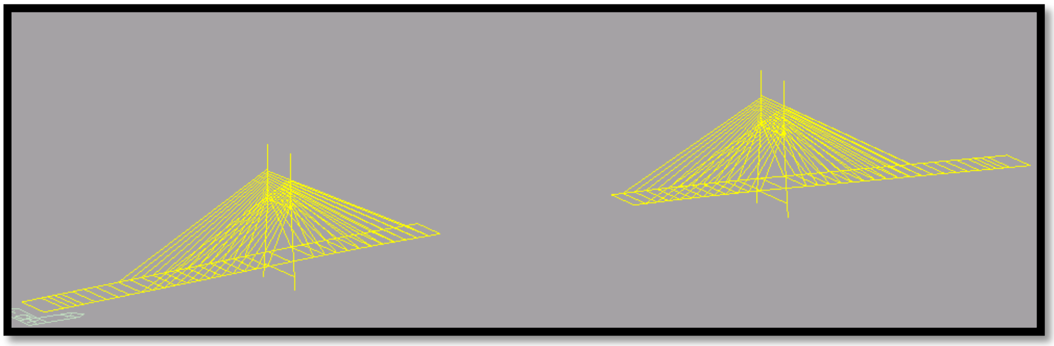
26. Construction stageBCS25

- Main girder (13) removal



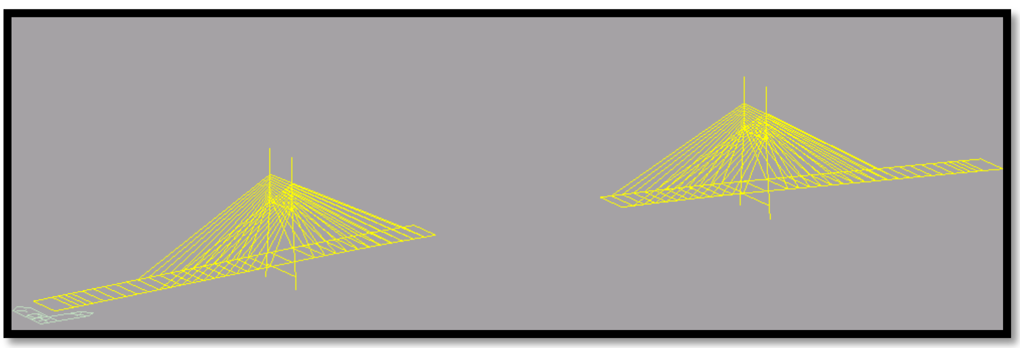
27. Construction stageBCS26

- Cable (32) removal



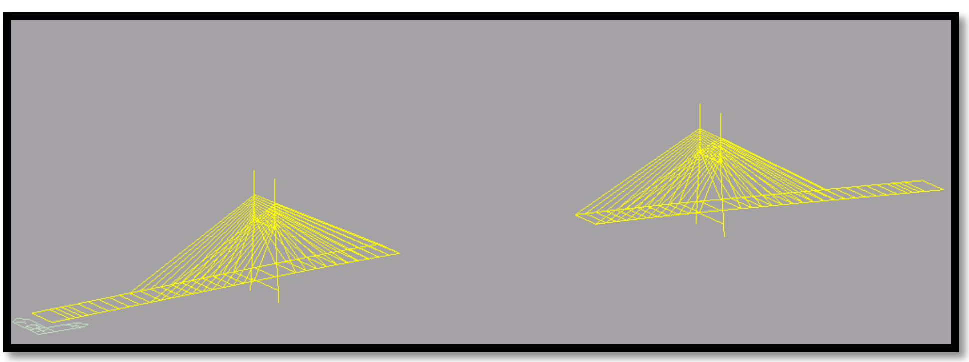
28. Construction stageBCS27

- Cable (9) removal



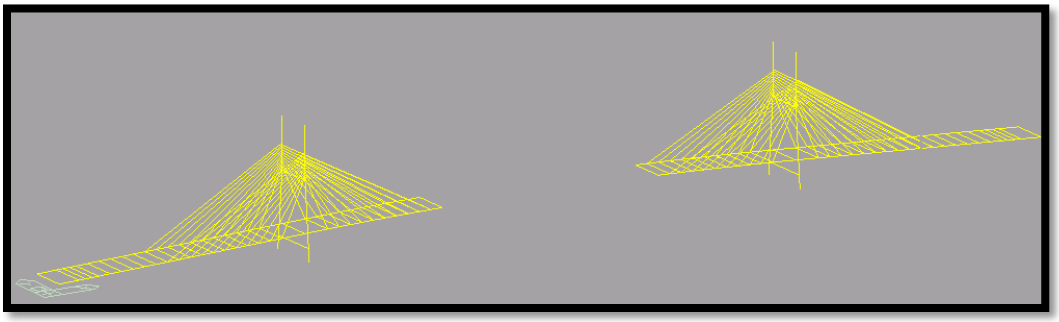
29. Construction stageBCS28

- Main girder (12) removal



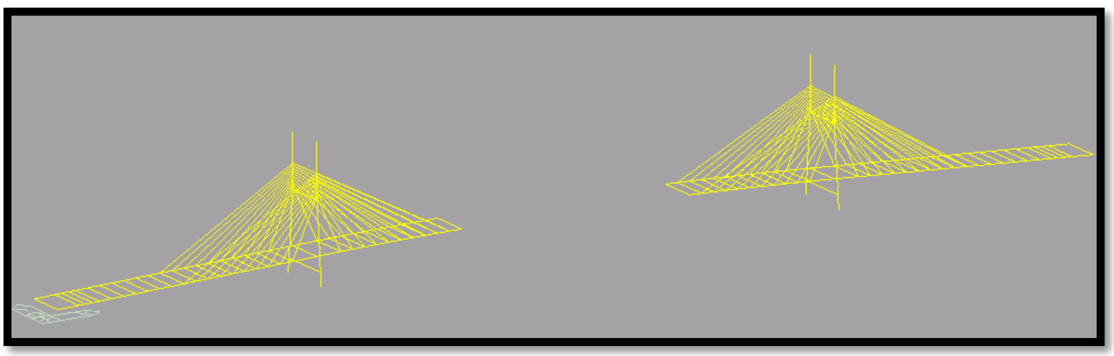
30. Construction stageBCS29

- Cable (31) removal



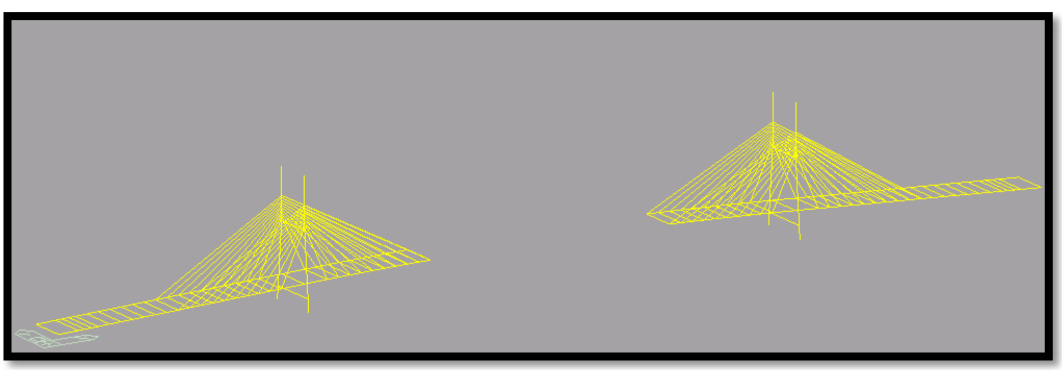
31. Construction stage BCS30

- Cable (10) removal



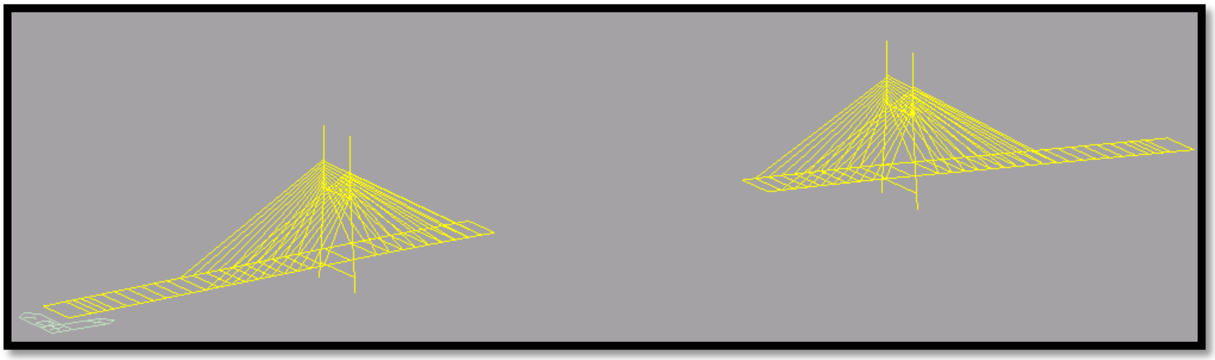
32. Construction stage BCS31

- Main girder (11) removal



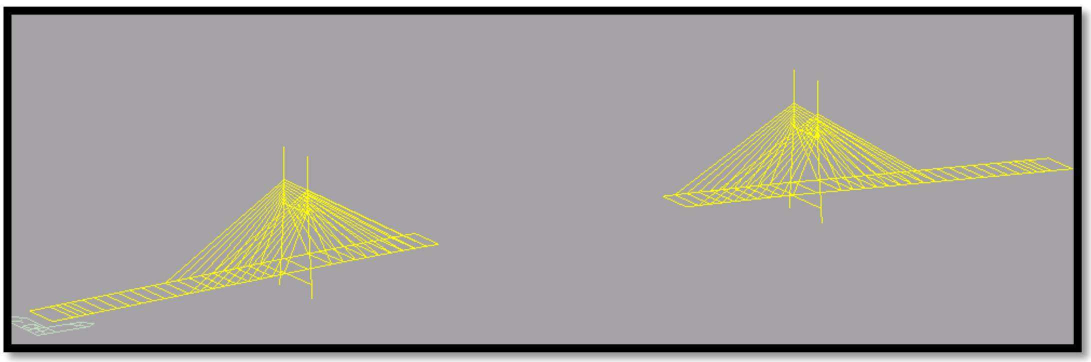
33. Construction stage BCS32

- Cable (30) removal



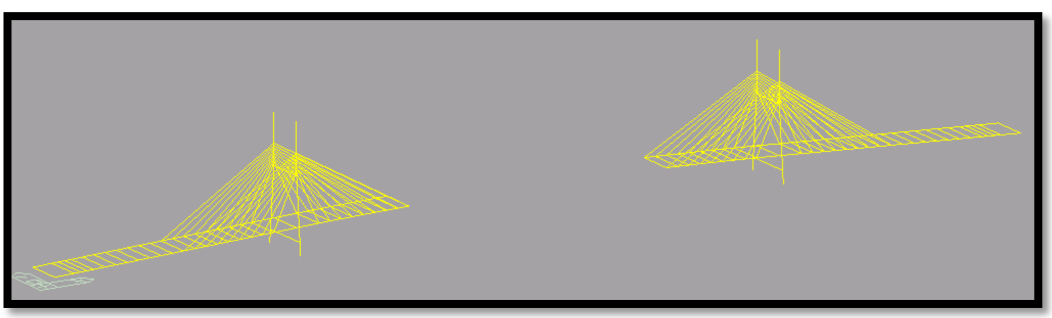
34. Construction stage BCS33

- Cable (11) removal



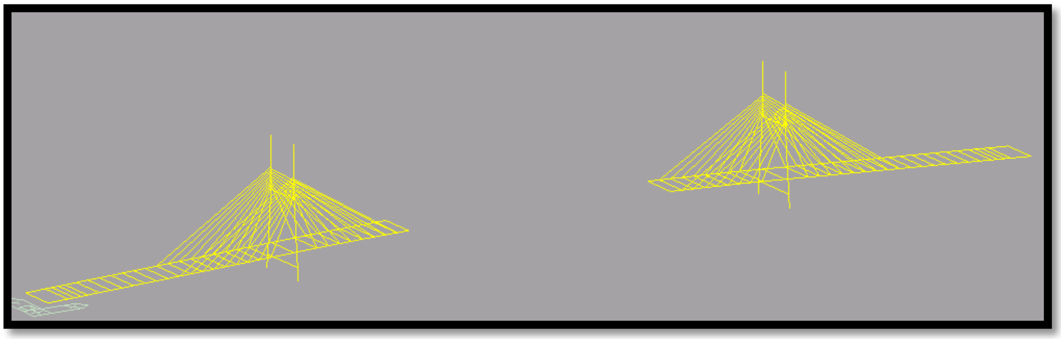
35. Construction stage B CS34

- Main girder (10) removal



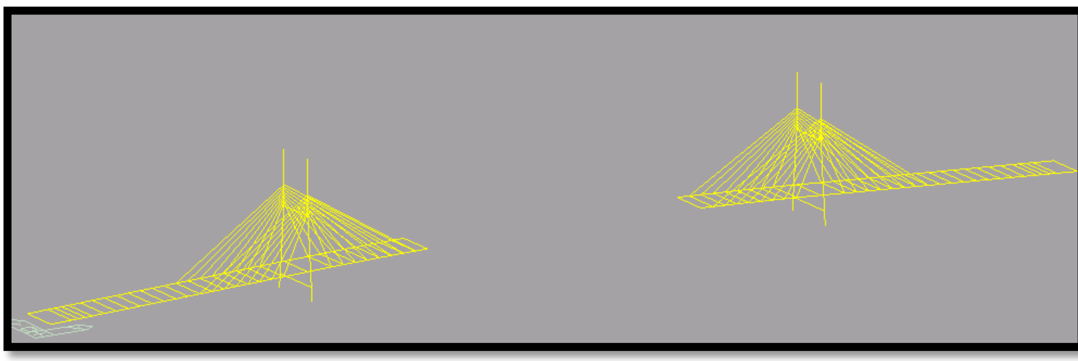
36. Construction stage B CS35

- Cable (29) removal



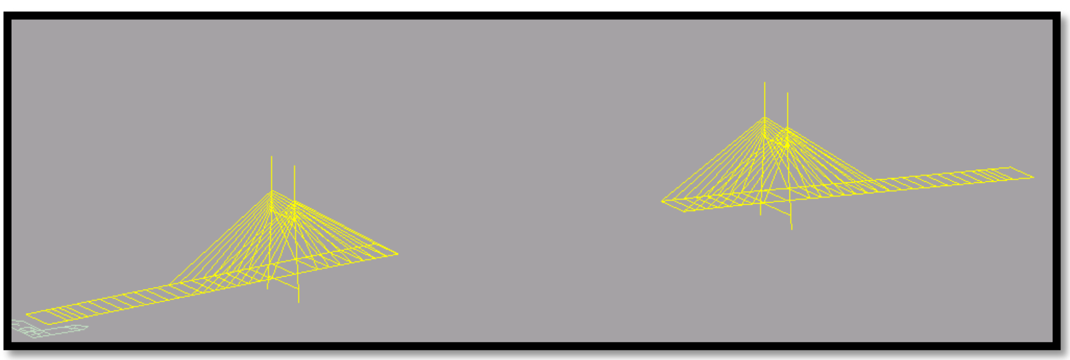
37. Construction stage B CS36

- Cable (12) removal



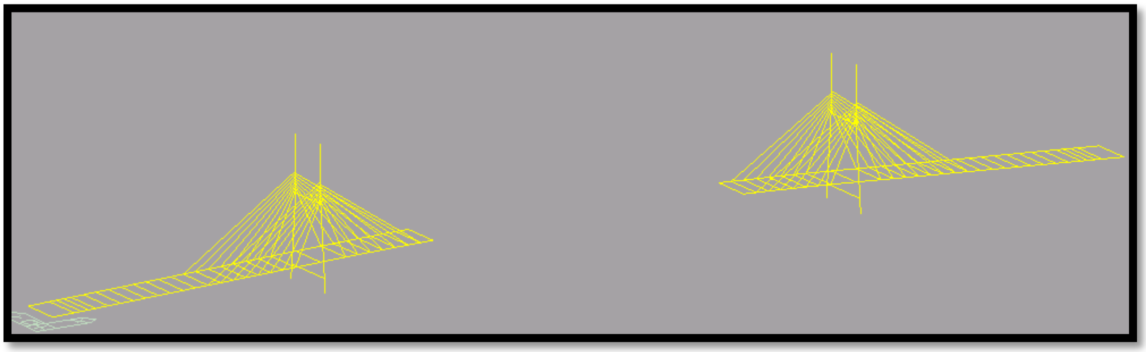
38. Construction stage B CS37

- Main girder (9) removal



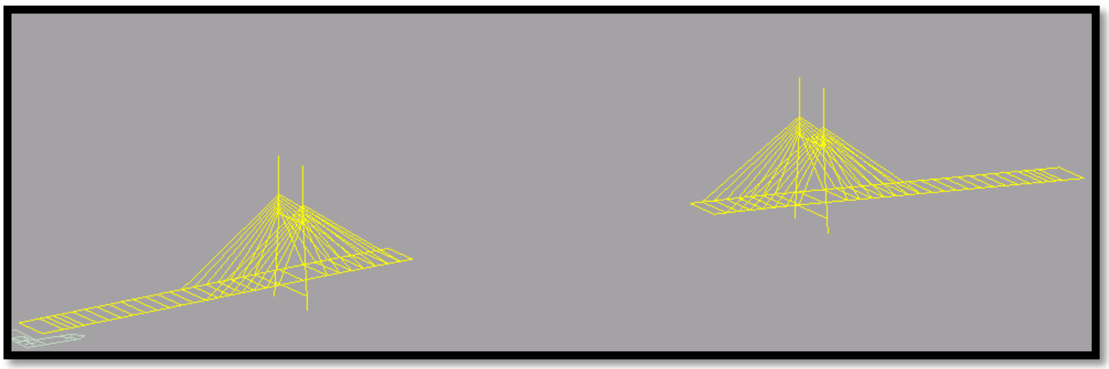
39. Construction stage B CS38

- Cable (28) removal



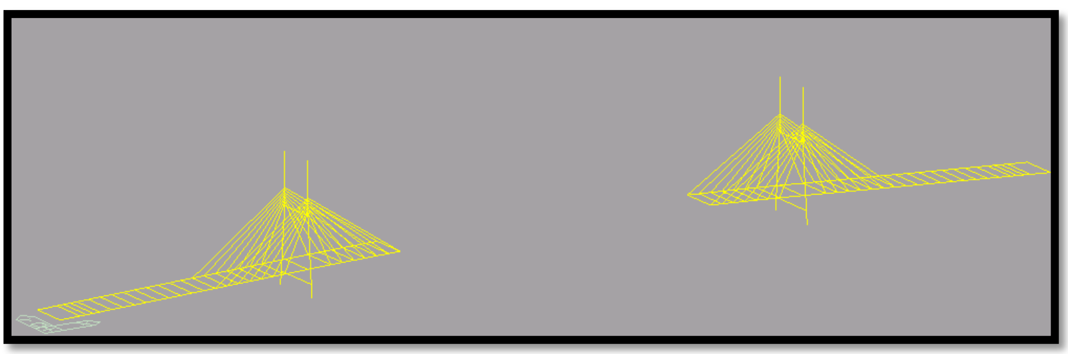
40. Construction stage B CS39

- Cable (13) removal



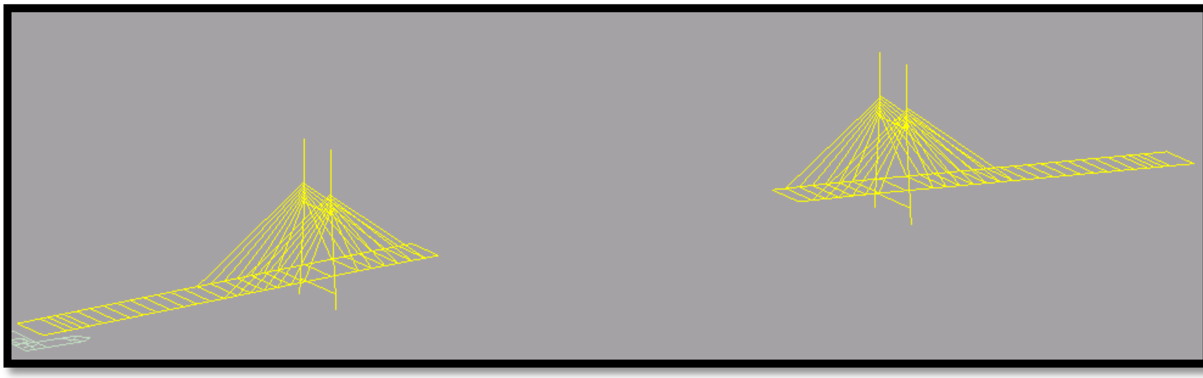
41. Construction stage B CS40

- Main girder (8) removal



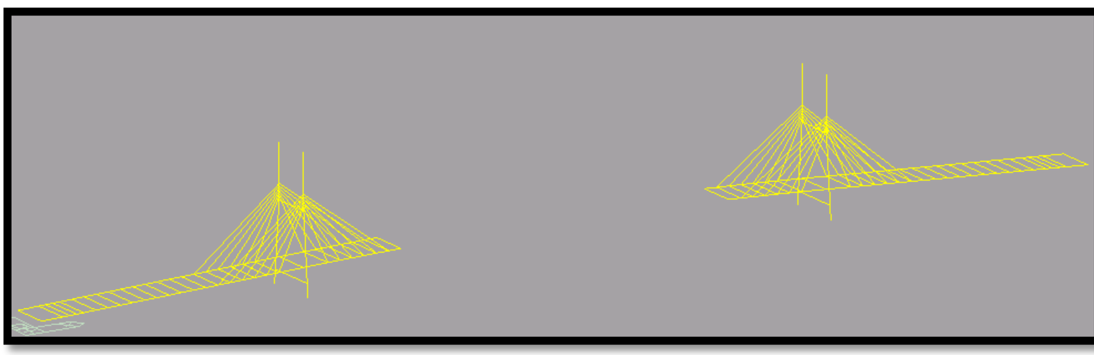
42. Construction stage B CS41

- Cable (27) removal



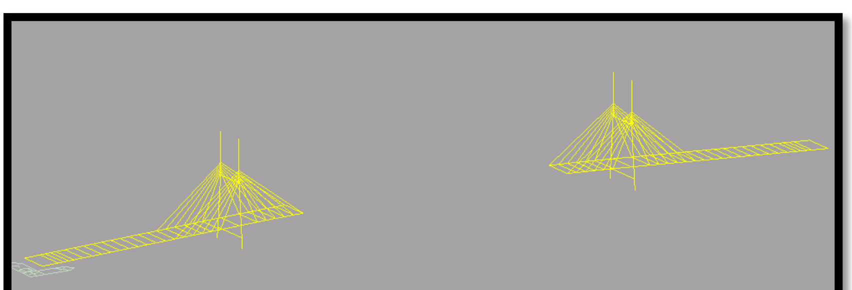
43. Construction stage B CS42

- Cable (14) removal



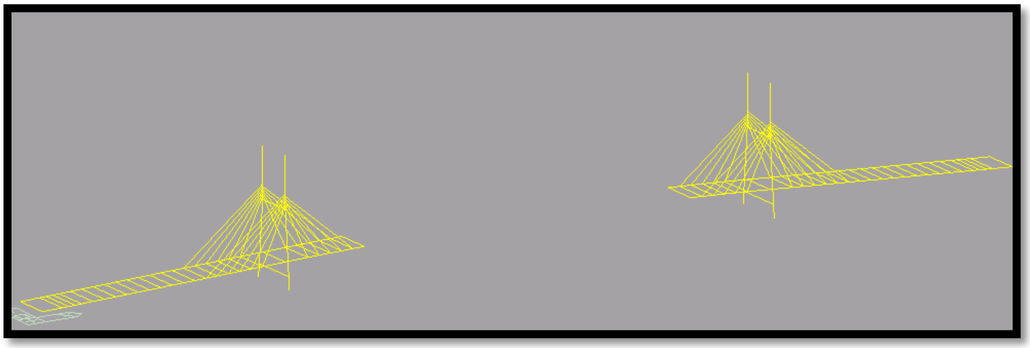
44. Construction stage B CS43

- Main girder (7) removal



45. Construction stage B CS44

- Cable (26) removal

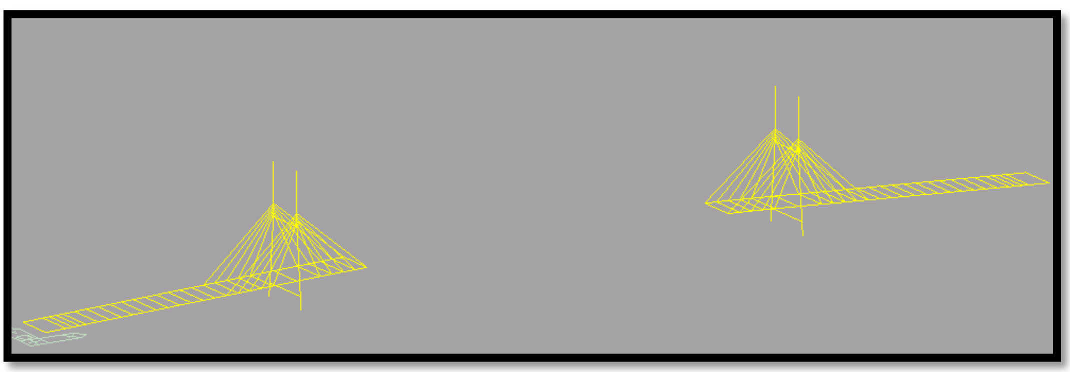


46. Construction stage B CS45

- Cable (15) removal

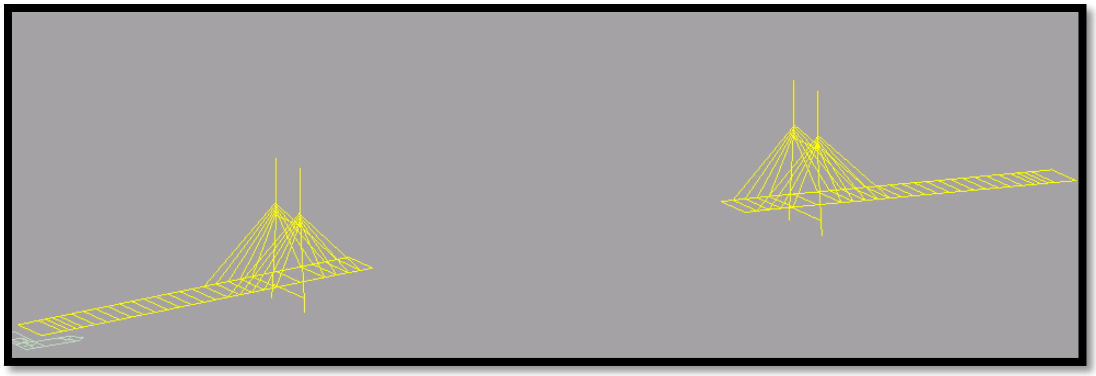
47. Construction stage B CS46

- Main girder (6) removal



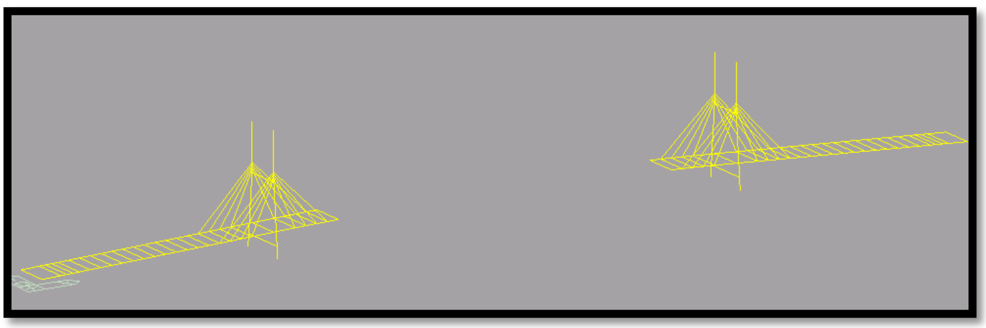
48. Construction stage B CS47

- Cable (25) removal



49. Construction stage B CS48

- Cable (16) removal

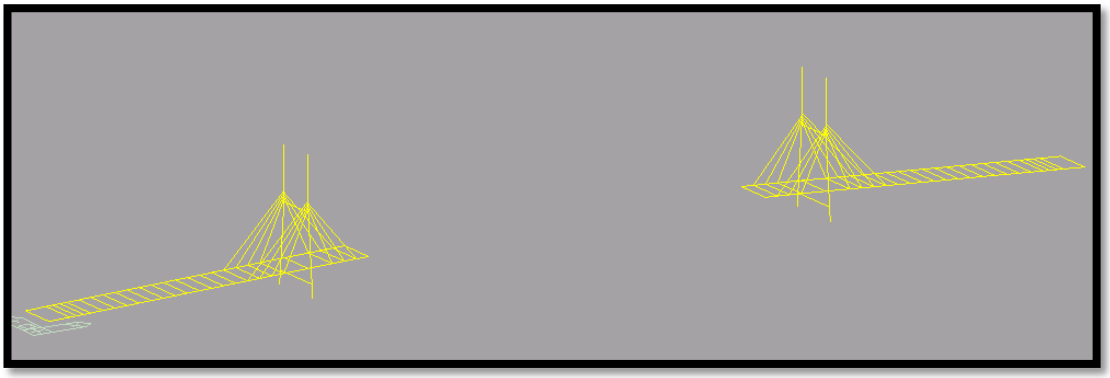


50. Construction stage BCS49

- Main girder (5) removal

51. Construction stage BCS50

- Cable (24) removal



52. Construction stage BCS51

- Cable (17) removal

53. Construction stage BCS52

- Main girder (4) removal

54. Construction stage BCS53

- Cable (23) removal

55. Construction stage BCS54

- Cable (18) removal

56. Construction stage BCS55

- Main girder (3) removal

57. Construction stage BCS56

- Cable (22) removal

58. Construction stage BCS57

- Cable (19) removal

`59. Construction stage BCS58

- Main girder (2) removal

60. Construction stage BCS59

* Cable (21) removal

61. Construction stage BCS60

* Cable (19) removal

62. Construction stage BCS61

* Main girder (1) removal

63. Construction stage BCS62

Figure (3-9)(1to 63): Analytical Sequence of Backward Construction Stages

3.5.8 Backward Construction Stage Analysis

The procedures of calculating backward analysis for cable-stayed bridges are outlined in Figure (3-10).

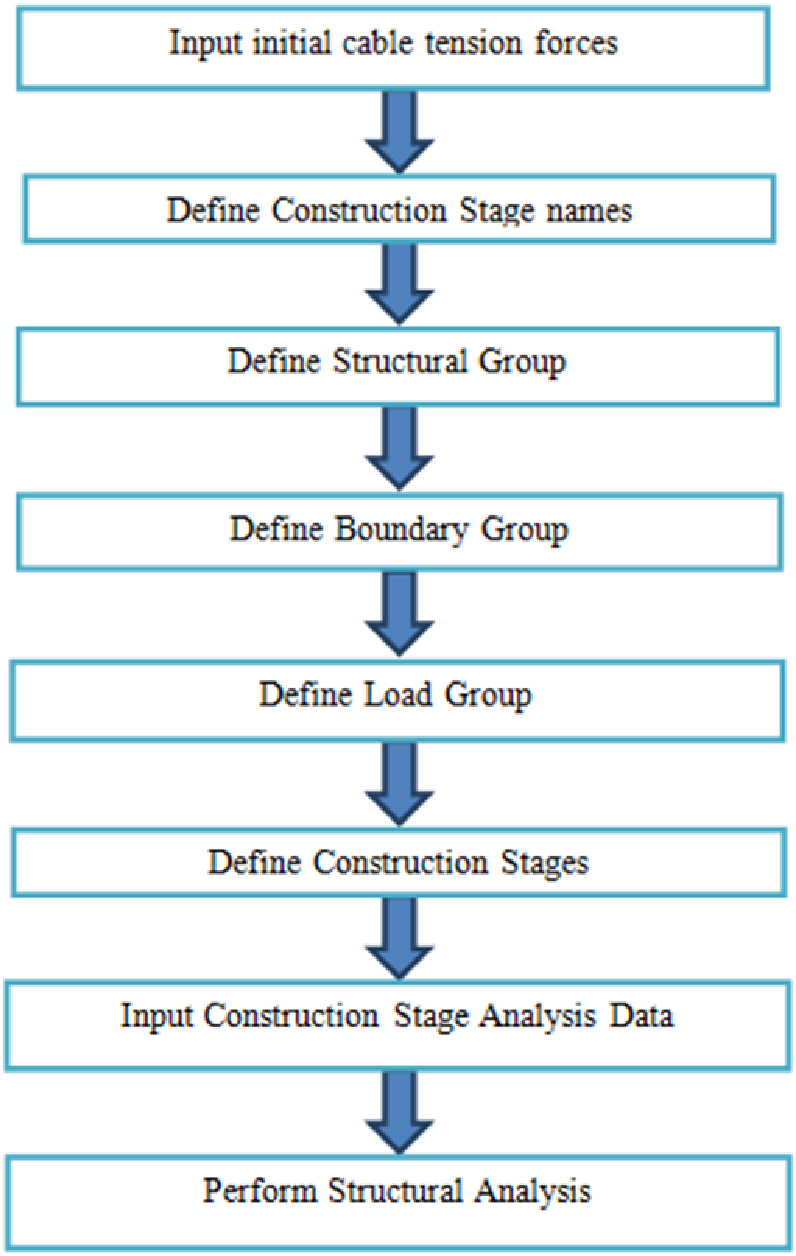


Figure (3-10): Flowchart for Construction Stage Analysis

The steps to carry out the backward analysis are as follows:

Step (1): generate a construction stage analytical model

A construction stage analytical model is generated using the model used in the final stage analysis by save the file under a different name.

Step (2): Input Initial Cable Pre-tension

The construction stage analysis model were created from the final stage model, the load combinations LCB 1 and LCB2 and unit pretension loading conditions (Tension 1 to Tension 40) deleted. Inputted the unknown load factors calculated by optimization technique as Pretension Loads and defined a new Loading case for initial pretension using the steps shown in Appendix B, (Figure B-24).

Step (3): Define Construction Stage

The each construction stage to perform backward construction stage analysis was defined. First; were assign each construction stage name in the Construction Stage dialog box using these steps shown in Appendix B, (Figure B-25).

Step (4): Define Structure Group

Define the element, which are added/deleted in each construction stage by Structure Group. After defined the name of each Structure Group, is assigned the relevant elements to the Structure Group. Using these steps shown in Appendix B, (Figure B-26).

Step (5): Define Boundary Group

Boundary conditions were defined, which become added/deleted in each construction stage, to each corresponding Boundary Group. After defined the name of each Boundary group, assigned the relevant boundary conditions to each Boundary group. Using these steps shown in Appendix B, (Figure B-27).

Step (6): Define Load Group

Define the loading conditions, which become added/deleted in each construction stage, to each corresponding Load Group. The loads considered in this backward construction stage analysis are self-weight, superimposed dead load and initial cable pretensions. First, were generated the name of each Load Group and then assign corresponding loading conditions to each Load Group, using these steps shown in Appendix B, (Figure B-28).

Step (7): Define Construction Stage

Predefined Structure Group is assign, Boundary Group and Load Group to each corresponding construction stage. First, were assigning the final stage (BCS0) to Construction Stage as the 1st stage in backward analysis. Using these steps shown in Appendix B, (Figure B-29).

Step (8): Input Construction Stage Analysis Data, using these steps shown in Appendix B, (Figure B-30).

Step (9): Perform Structural Analysis

Construction stage analysis for self-weight, superimposed dead load and initial cable pre tension was Perform, using these steps shown in Appendix B, (Figure B-31).

3.5.9 Dynamic analysis

The majority of cable- stayed bridge methods of analysis are limited to static loads

with very little information being presented concerning dynamic behavior However, under the influence of wind, seismic and traffic loads there is a dynamic response by cable –stayed bridges. Dynamic studies include the determination of the natural modes and frequencies of the bridge. The procedure of calculating the frequency of vibration for cable-stayed bridges by Midas civil are outlined in the following section

Step (1): Defined structure type using the following steps and as shown in Appendix B, Figures (B-32) and (B-33).

Step (2): convert loads to mass using the following steps and as shown in Appendix B Figure (B-34).

Step (3) eigenvalue analyses using the following steps and as shown in Appendix B, Figure (B-35).

3.6 Wind load calculations

According to AASHTO LRFD, (2010), wind load shall be assumed to be uniformly distributed on the area exposed to wind. The exposed area shall be the sum of areas of all components, including floor system and railing, as seen in elevation taken perpendicular to the assumed wind direction. This direction shall be varied to determine the extreme force effect in the structure or in its components.

The applied wind pressure on the structure had been calculated based on AASHTO CI3.8.1.2 for the different heights. The critical wind velocity selection was based on the collected information and the maximum wind velocity of 40 m/s received from the owner, was used in the calculations.

The final wind load calculation results are as show in Table (3-7)

Table (3-7): Wind load calculation

Ref	**Calculation**	**Output**
AASHTO LRFD 2010	The design wind velocity, V_{DZ}, should be adjusted according to $V_{DZ} = 2.5V_0 \left[\frac{V_{10}}{V_B}\right] ln \left[\frac{Z}{Z_0}\right]$. Design wind pressure $P_D = P_B \left[\frac{V_{DZ}}{V_B}\right]^2$	
	V_{DZ}: design wind velocity at design elevation, Z (km/hr)	
	V_{10}: wind velocity at 10 m above low ground or above design water level (km/hr).	
	Z: height of structure at which wind loads are being calculated as measured from low ground, or from water level > 10m	
C 3.8.1.2.	$V_{B:}$ base wind velocity of 160 km/hr at 10m height.	
	V_0: friction velocity, a meteorological wind characteristic taken, as specified in	
	P_B : PB = base wind pressure specified in Table 3.8.1.2.1-1	
	V_0 =17.6 km/hr V_{10} =145 km/hr	
	Z_0 = 1m V_B = 160km/hr	

	Z m	V_{DZ}(Km/hr)	P_D(MPa)
	78.8	174.13	.00426
	49	155.19	.00339
deck	14	105.23	.00104

3.7 Design Parameters:

The TUTI BAHARI cable -stayed bridge elevation as shown in Figure (3-11), calculated the Equivalent modulus of elasticity method for the model of cable – stayed bridge using the following equation

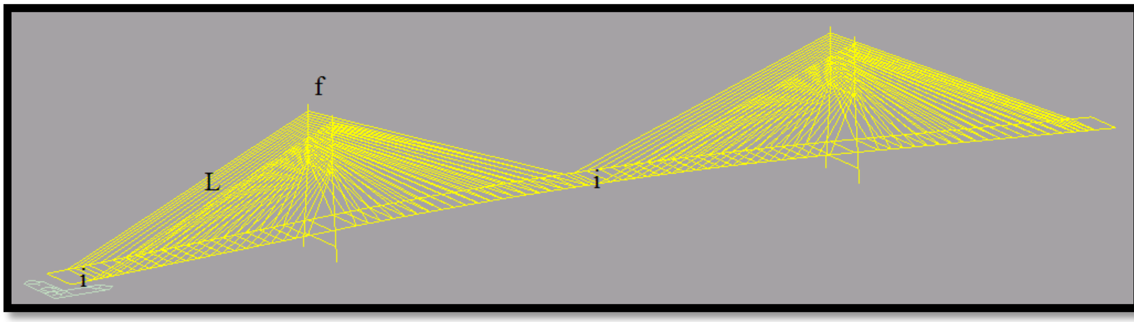


Figure (3-11): TUTI BAHARI bridge elevation view

Equivalent modulus of elasticity method

$$E_{eq} = \frac{E}{1+\left[\frac{(wL)^2\left(T_i+T_f\right)AE}{24T_i^2\,T_f^2}\right]} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.18)$$

Where:-

w : Weight of cable per unit length.

L: Horizontal length of cable.

T_i, T_f : Initial and final value of the tension.

E: Modulus of elasticity of the straight stay.

A: Cross section area of cable.

L = 138 m

W = A*γ= 1.39 kN / m

Ti = 6583.78 kN (Table (4-1))

Tf = 6670.92 kN (Table (4-1))

E = 1.97*10^8 kN/m^2

$$E_{eq} = \frac{1.97 * 10\text{^}8}{1 + \left[\frac{(1.39)^2(6583.78 + 6670.92)0.0177 * 1.97 * 10\text{^}8}{24 * 6583.78^2 * 6670.92^2}\right]}$$

Modulus of elasticity = 1.97*10^8

$$\sigma_{applied} = \frac{T_{MAX}}{A} = \frac{6670.92}{0.0177} = 376888\ kN/m2$$

3.8 Pre- tension of cable -Manual calculation

The stay forces pi can be determined by equation

$$\mathrm{p_i} = \frac{\mathrm{R_i}}{\sin\propto_\mathrm{i}} \text{..} (3.19)$$

The reaction, $\boldsymbol{R_i}$ at each cable- stayed node May simple be determined as $\boldsymbol{R_i = sw}$

3.8.1 Calculation pre –tension of cable 20

Ri = 99*10.5= 1039.5 kN

$$\mathrm{p_i} = \frac{1039.5}{\sin\propto_\mathrm{i}} = 1237.5\mathrm{kN}$$

The above equation were used to calculate the initial value of the pre-tension forces in the cables for the cable –stayed bridge as a first estimate as shown in Table (4-19) in chapter four.

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

$$A_i = \frac{R_i}{\sigma_{allow} \sin\alpha_i} \qquad (3.20)$$

CHAPTER FOUR

Analysis and Design Stage Results and Discussion

4.1 Introduction

This chapter includes the result obtained of Midas civil program under (LCB2) Dead loads (self-weight + additional load) +cables pre tensioning force applied to case1 (strand cable), case 2(New parallel-wire strand PWS) and case 3(Carbone fiber). Also, it include the results obtained under Dead loads (self-weight + additional load) + cables pre tensioning force +live loads (moving loads (LCB3). and wind load. The discussion of these results is based on using tables and graphs which are plotted for the respective parameters.

4.2 Analyses Result

4.2.1 Static analysis

Static analysis has been performed for the model cable stayed bridge for different types of loads. The dead load has great influence on the stiffness of the cable stayed bridges. Moving load analysis of a bridge structure entailed a series of analyses for all loading conditions created along the entire moving load path to find the maximum and minimum values. These are used as the results of the moving load case. The result of pre –tension force, deformation, and stress in cable obtained from analysis of TUTI BAHRI cable –stayed bridge by using Midas civil program under LCB2, LCB3 and wind load are as shown in the following sections.

1. Pre –tension force

The initial cable pre-tension forces were obtained by the Unknown Load Factor optimization function and the initial equilibrium state analysis of the complete cable–stayed bridge.

The values of cable pre –tension forces were obtained by using the three types of cable (strand cable – New parallel wire strand- Carbone fiber cable). These values obtained from the analysis of TUTI BAHRI cable –stayed bridge by using Midas civil are as shown in Tables (4-1),(4-2) and (4-3). The maximum value of the pre –tension obtained from LCB2 occur at cable number 1, and the minimum value occur at the cable number 21.

Table (4-1): Pre –tension force (case 1- strand Cable)

cable number	Forces at end I kN	Forces at end J kN
1	6583.777737	6670.924041
2	6048.393502	6133.455631
3	5448.552894	5531.530848
4	4879.247829	4960.141608
5	4288.300742	4366.901929
6	4034.761193	4111.069787
7	4079.683495	4153.699496
8	4273.157097	4344.880506
9	4309.214665	4378.645481
10	4038.891137	4106.029361
11	3716.747166	3781.592797
12	3514.369443	3576.922482
13	3397.475731	3457.736178
14	3180.529454	3238.497308
15	2979.319943	3034.995205
16	2705.764521	2759.147190
17	2545.094104	2596.184180
18	2613.256475	2662.053959
19	2384.453862	2430.958754
20	1029.698032	1073.910331
21	872.540545	914.853003
22	2129.713973	2173.403144
23	2412.148318	2457.236897
24	2322.169726	2368.680407
25	2472.041165	2519.996643
26	2764.456163	2813.879131
27	2964.259247	3015.172401
28	3160.380917	3212.806949
29	3388.374021	3442.335628
30	3584.856148	3640.376023
31	3798.158713	3855.259550
32	4009.926585	4068.631079
33	4207.725787	4268.056633
34	4414.655470	4476.635361
35	4615.341077	4678.992708
36	4770.864935	4836.211000
37	4994.477318	5061.540512
38	5287.692666	5356.495682
39	5553.123843	5623.689377
40	5539.630746	5611.981492

Table (4-2): pre –tension force (case 2- New parallel-wire strand)

Cable number	Forces at end I kN	Forces at end J kN
1	6515.259520	6604.18000
2	6014.449989	6095.784677
3	5443.486601	5522.828443
4	4911.912591	4989.261587
5	4333.946748	4409.103613
6	4055.026039	4127.990774
7	4064.287723	4135.060328
8	4227.110221	4295.690695
9	4265.842632	4332.230975
10	4043.545014	4107.741227
11	3749.264494	3811.268576
12	3537.952373	3597.764324
13	3356.735162	3414.354982
14	3178.997351	3234.425041
15	2949.835956	3003.071515
16	2724.116516	2775.159945
17	2565.961174	2614.812472
18	2613.544402	2660.203570
19	2354.627340	2399.094377
20	1046.589535	1088.864442
21	899.134072	939.592389
22	2112.137473	2153.912175
23	2382.284962	2425.397749
24	2368.671973	2413.144546
25	2461.277931	2507.131988
26	2733.248146	2780.505389
27	2986.435155	3035.117283
28	3155.913436	3206.042149
29	3358.689272	3410.286269
30	3620.139232	3673.226214
31	3796.415332	3851.013998
32	3970.628366	4026.760416
33	4207.290881	4264.978016
34	4445.215829	4504.479748
35	4563.117044	4623.979446
36	4721.317892	4783.800478
37	5017.294267	5081.418737
38	5356.606144	5422.394198
39	5568.110653	5635.583990
40	5443.129383	5512.309703

Table (4-3): Pre –tension for case 3 –Carbone fiber

Cable number	Force at end -I (kN)	Force at end -J (kN)
1	6258.982789	6276.745093
2	5801.146050	5818.483554
3	5294.892855	5311.805559
4	4793.730744	4810.218648
5	4235.920334	4251.940958
6	3920.173568	3935.726912
7	3887.757604	3902.843668
8	4052.210584	4066.829368
9	4175.017068	4189.168572
10	3991.454921	4005.139145
11	3694.011496	3707.228440
12	3473.645539	3486.395203
13	3333.186996	3345.469380
14	3099.437284	3111.252388
15	2894.900789	2906.248613
16	2710.623250	2721.503794
17	2587.652598	2598.065862
18	2573.521601	2583.467585
19	2302.328343	2311.807047
20	1103.277814	1112.289238
21	982.302747	990.926942
22	2064.793032	2073.697831
23	2326.462376	2335.652405
24	2371.333543	2380.813427
25	2482.811196	2492.585561
26	2643.134157	2653.207628
27	2963.183910	2973.561113
28	3227.771254	3238.456815
29	3286.205109	3297.203653
30	3454.309163	3465.625316
31	3640.499958	3652.138345
32	3940.364014	3952.329261
33	4160.522300	4172.819033
34	4264.022223	4276.655067
35	4366.663884	4379.637465
36	4631.297630	4644.616573
37	4935.812461	4949.481393
38	5243.418609	5257.442154
39	5312.635614	5327.018398
40	5140.472091	5155.218740

- Figures (4-1)and(4-2)show the cable pre – tension forces for each cable under LCB2 and LCB3 Table (4-4) respectively according to case 1(strand cable). The maximum cable force under LCB2 is 6670 kN at the first cable of the main girder, which is within the allowable range of the tension strength limit of the tendon. Also, the maximum cable force under LCB3 is 7229 kN at the cable (1).

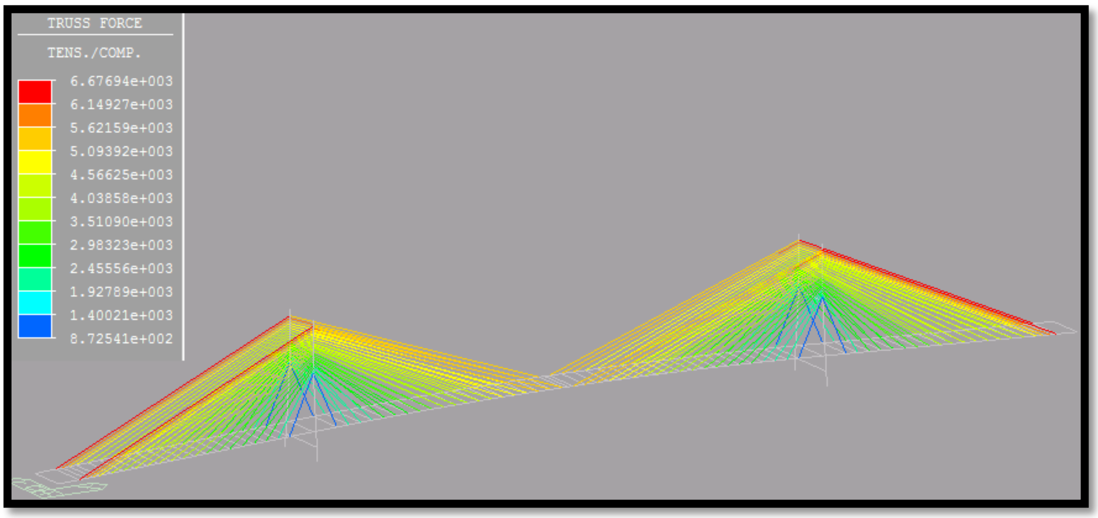


Figure (4-1): Pre – tension force under LCB2 (case 1)

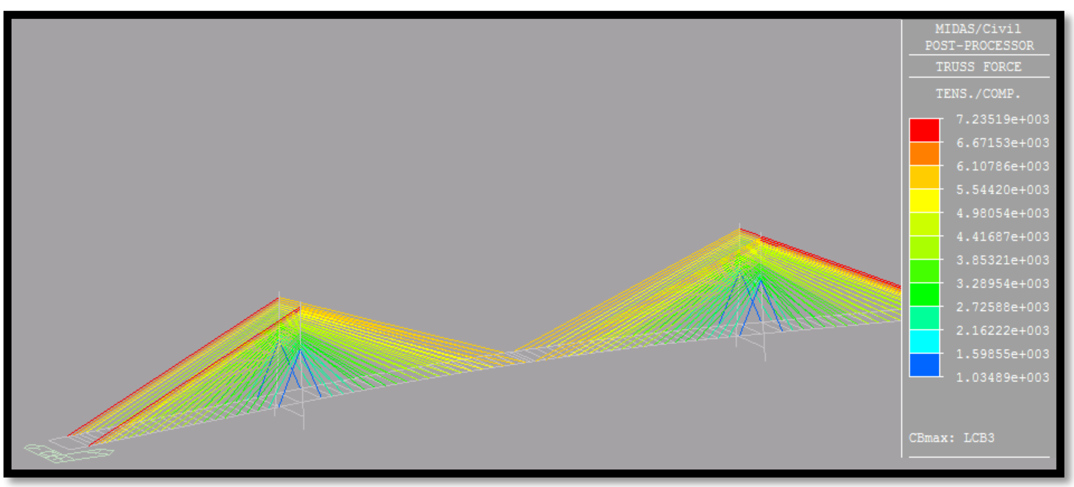


Figure (4-2): Pre -tension force under LCB3 (case 1)

- The cable pre –tension forces obtained from case 2(New parallel-wire strand PWS) and case 3(Carbone fiber) is shows in Figures (4-3) and (4-4).The maximum value of pre –tension force which occurs in first cable under LCB2 is 6604.180 kN and 6276.7451 kN for case 2 and case 3 respectively.

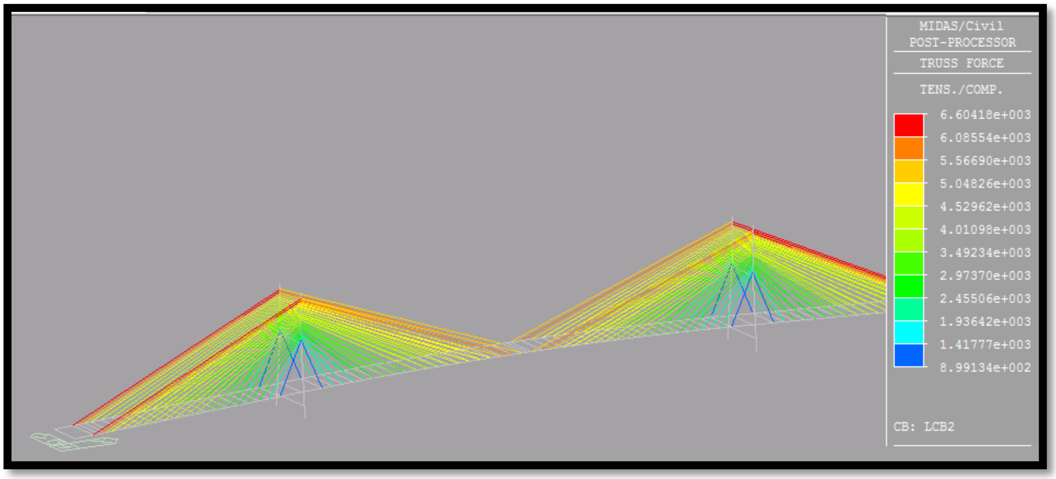


Figure (4-3): pre – tension force under LCB2 (case2)

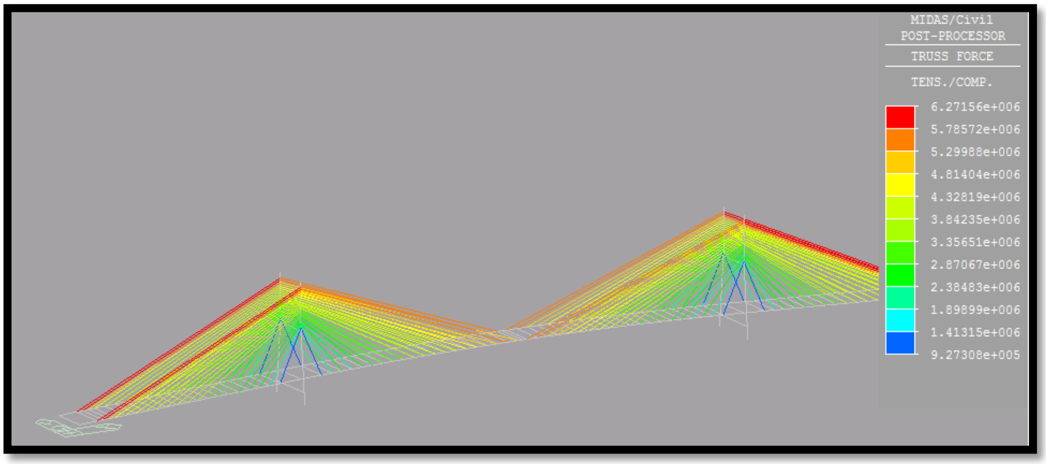


Figure (4-4): pre – tension force under LCB2 (case 3)

Table (4-4):Pre- tension force under LCB3

Cable number	Forces at end I kN	Forces at end J kN
1	7142.844988	7229.991292
2	6588.644190	6673.706319
3	5953.947519	6036.925473
4	5341.427797	5422.321576
5	4668.302398	4746.903585
6	4347.911756	4424.220350
7	4399.958026	4473.974027
8	4602.114691	4673.838100
9	4643.614759	4713.045575
10	4378.226168	4445.364392
11	4064.080603	4128.926235
12	3862.652318	3925.205357
13	3739.304293	3799.564740
14	3512.077173	3570.045027
15	3300.019818	3355.695080
16	3016.181490	3069.564159
17	2846.554229	2897.644305
18	2909.490475	2958.287959
19	2682.356050	2728.860941
20	1325.077063	1369.289362
21	1167.496608	1209.809066
22	2423.236254	2466.925425
23	2702.802037	2747.890615
24	2618.422914	2664.933595
25	2775.904446	2823.859924
26	3075.555726	3124.978694
27	3282.569091	3333.482244
28	3486.625073	3539.051106
29	3723.414365	3777.375972
30	3929.221711	3984.741585
31	4151.493525	4208.594362
32	4370.363148	4429.067642
33	4571.686631	4632.017476
34	4778.847407	4840.827298
35	4980.451609	5044.103239
36	5138.754935	5204.101000
37	5366.278724	5433.341918
38	5665.817510	5734.620526
39	5944.624780	6015.190314
40	5955.804996	6028.155742

2.Displacement:

Figures (4-5)and (4-7) show the maximum values of the vertical displacements at the main girder under LCB2and LCB3 for case 1, Figures (4-6) and (4-8) show The maximum values of the horizontal displacements at the top of the Pylons under LCB2and LCB3 respectively from case 1.

The maximum values of the horizontal displacements at the top of the Pylons under LCB2and LCB3 for case1 are 0.033308m and 0.060295m respectively. Considering the Load combinations with live loads in the central span displacement values are below the limit value of $\delta_max=H/300=66/300=0.220$m (H being the pylon height above the piles cap).Also, the maximum displacements at the center span of the main girder under LCB2 and LCB3 are 0.001m and 0.026m respectively, and are satisfactory as per the criteria (L/800=0.375 m).. .

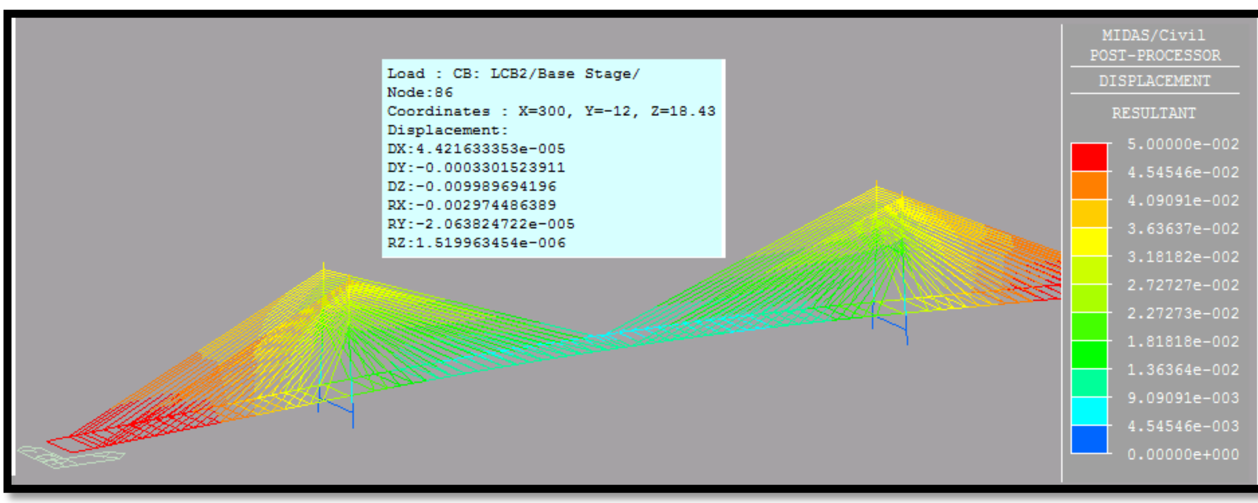


Figure (4-5): cable stayed bridge displacement under LCB2

(Main girder displacement) (Case 1)

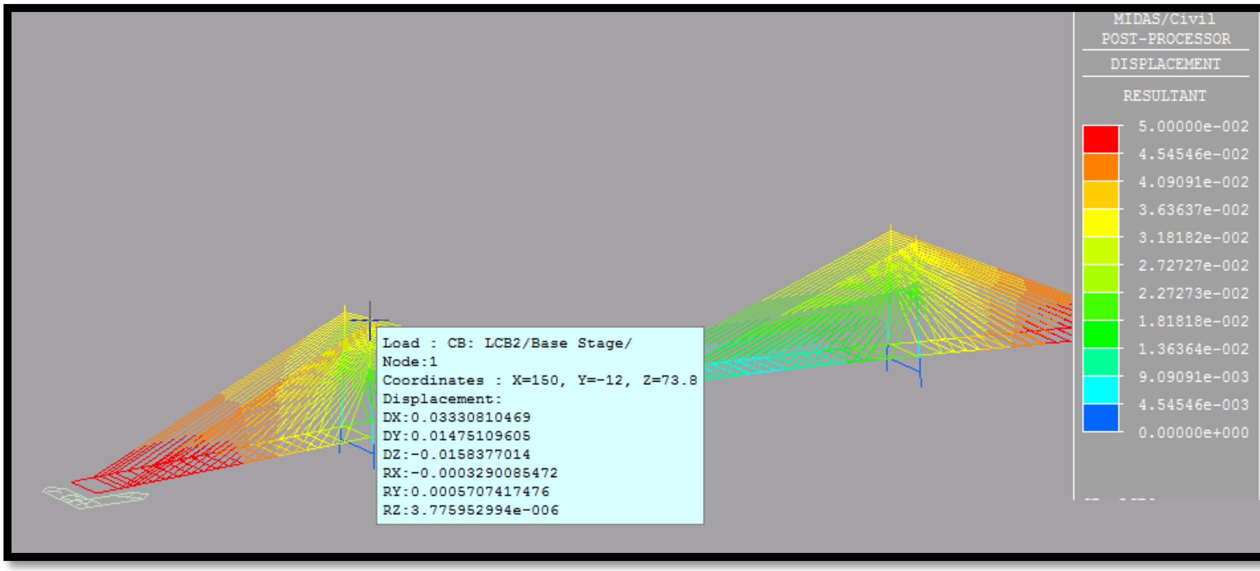


Figure (4-6): cable stayed bridge displacement under LCB2

(Tower displacement) (Case 1)

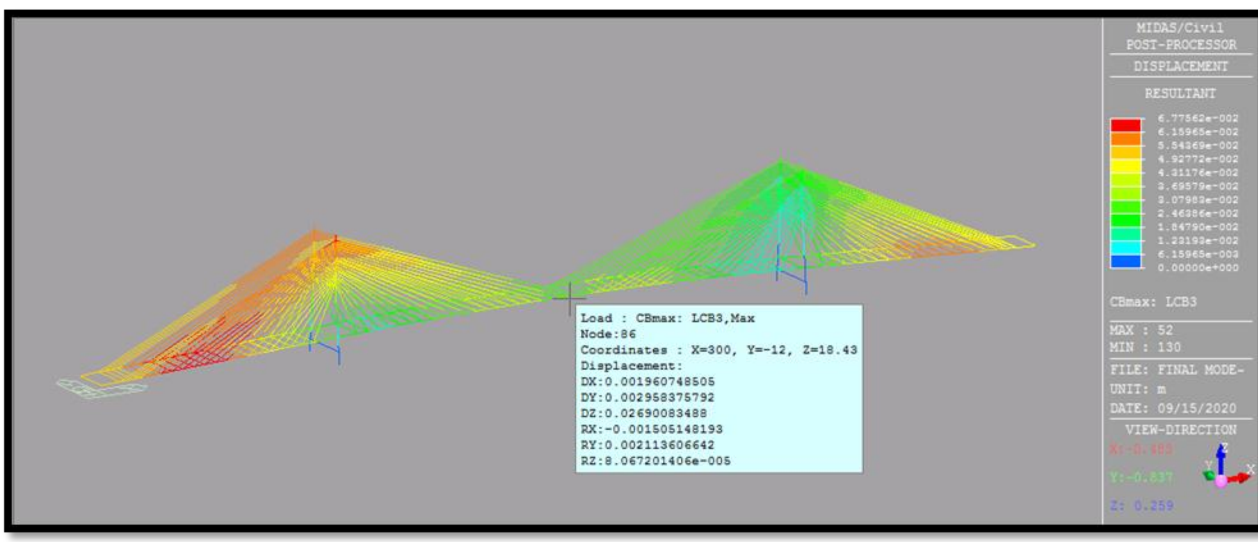


Figure (4-7): cable - stayed bridge displacement under LCB3

(Main girder displacement) (Case 1)

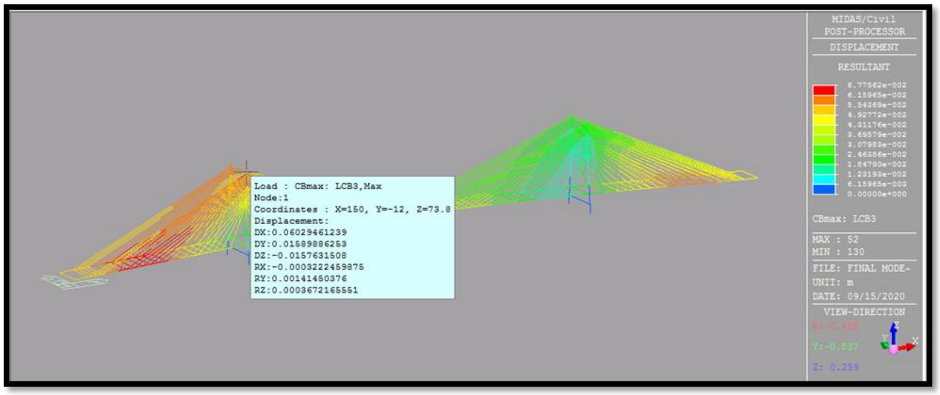


Figure (4-8): cable - stayed bridge displacement under LCB3

(Tower displacement)(Case 1)

- Figures (4-9) and (4-11), show the maximum values of the vertical displacements at the main girder under LCB2 for case 2 and case 3,Figures (4-10) and (4-12) show The maximum values of the horizontal displacements at the top of the Pylons under LCB2 for case 2 and case 3 respectively. The maximum value in the main girder is -0.009310m (case 2), -0.008173m (case 3) and the maximum value in top tower is 0.033166m (case 2) and 0.031438m (case 3).

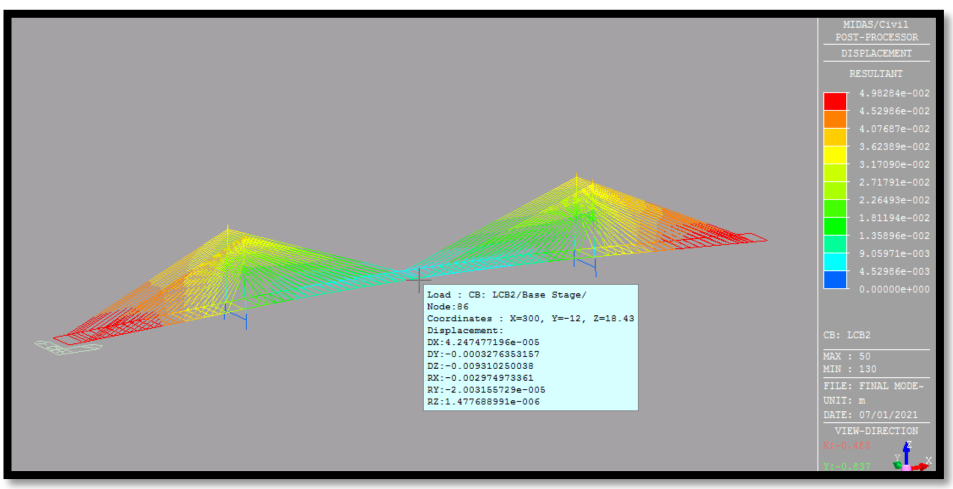


Figure (4-9): cable - stayed bridge displacement under LCB2

(Main girder displacement) (Case 2)

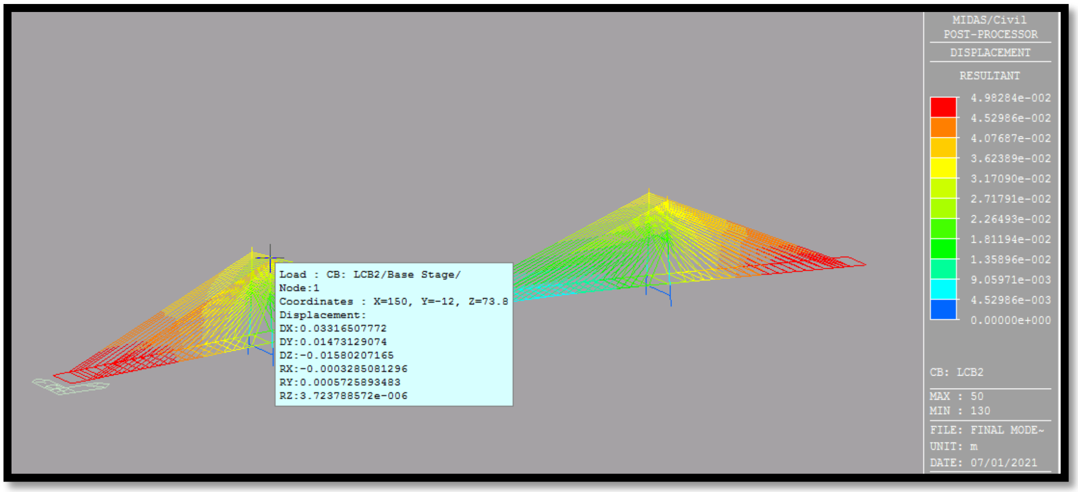


Figure (4-10): cable - stayed bridge displacement under LCB2

(Tower displacement) (Case 2)

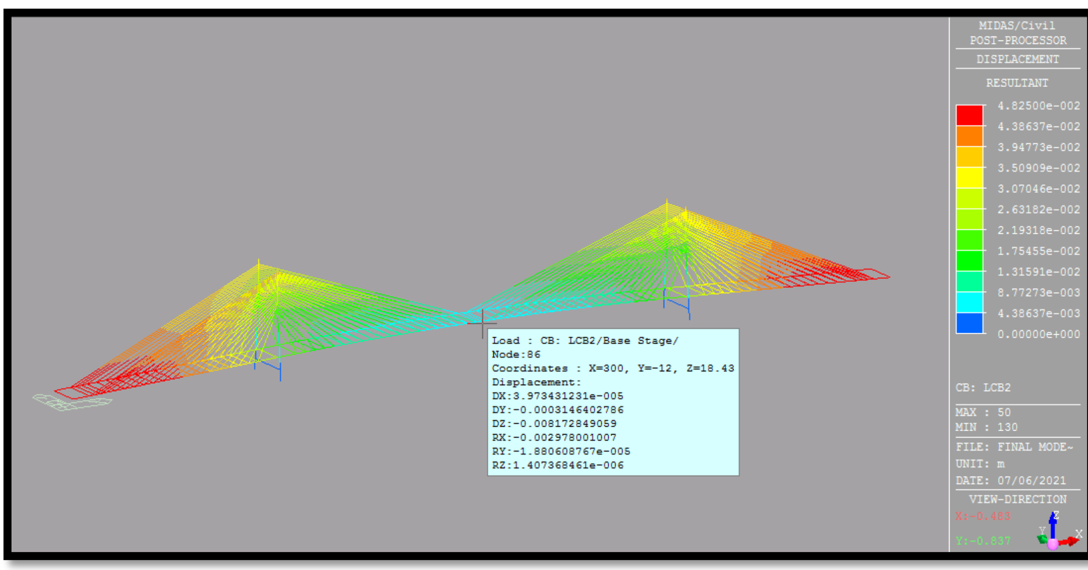


Figure (4-11): Cable - stayed bridge displacement under LCB2

(Main girder displacement) (Case 3)

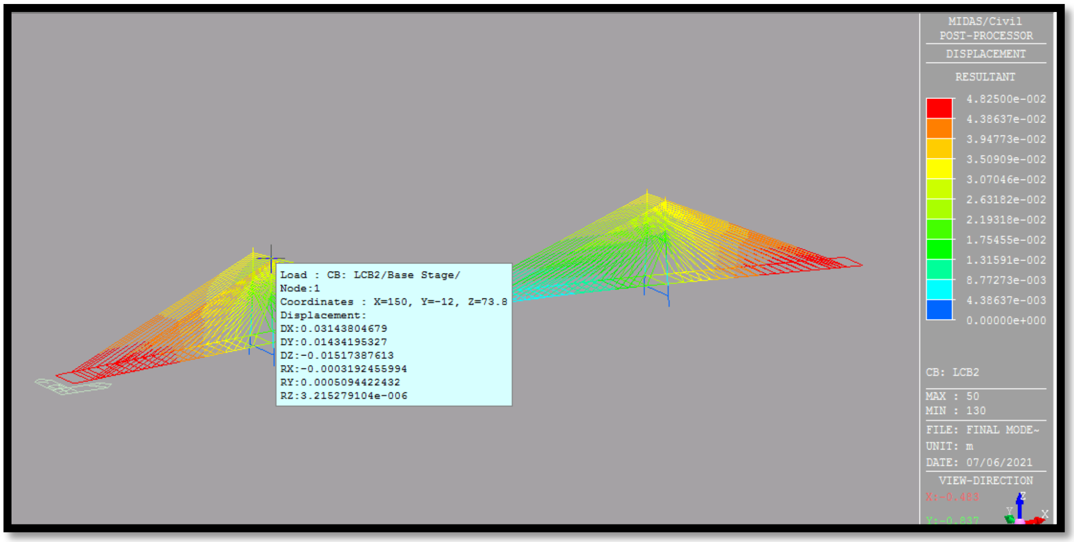


Figure (4-12): Cable - stayed bridge displacement under LCB2

(Tower displacement) (Case 3)

3. Cable stress

The various values of the cable stresses in N/mm^2 for individual cable profile under LCB2and LCB3 according to case 1 are illustrated in Figures (4-13)and (4-14) respectively. Also, the values of stress in the tower and girder are shown in Figure (4-15). Tables (4-5) and (4-6) presents the value of cable stress under LCB2 and LCB3 for case 1. The maximum cable stress for LCB2 and LCB3 equal to 377 N/mm^2 and 408.5 N/mm^2 respectively. This value occurs in the two long stay cables with maximum pre-tensions.

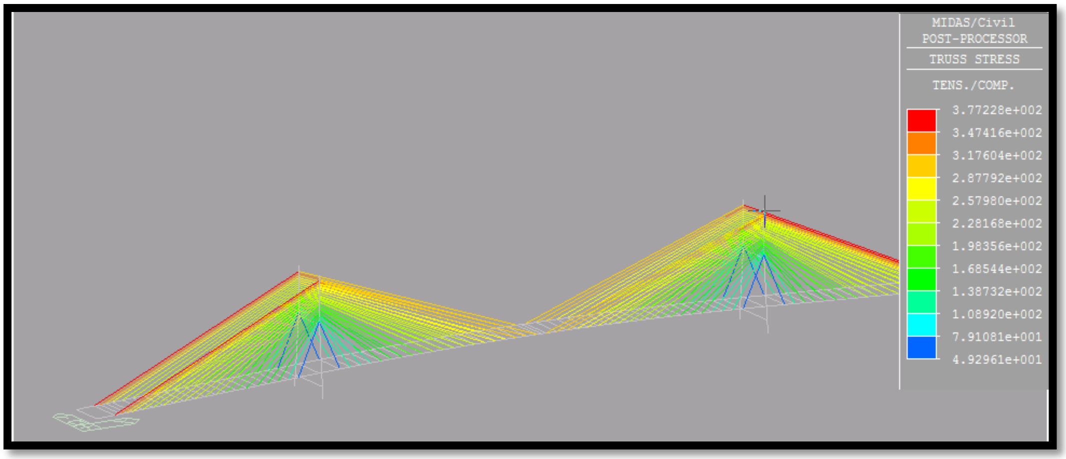


Figure (4-13): Cable stress under LCB2 (case 1)

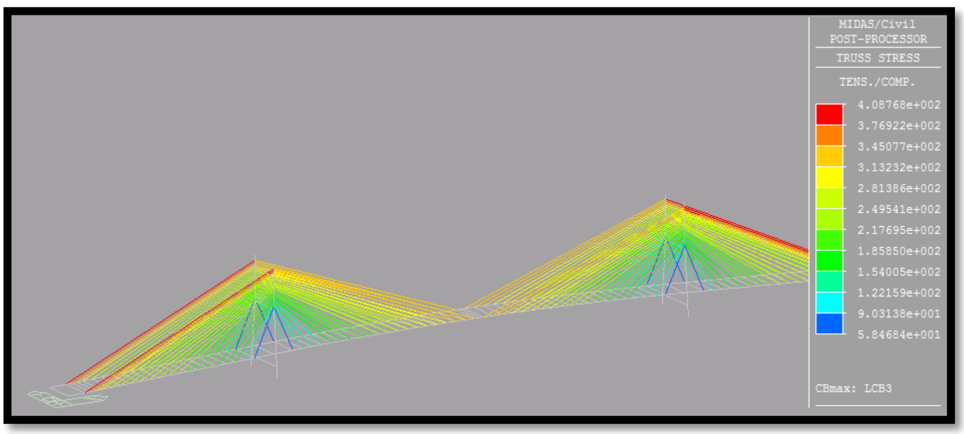
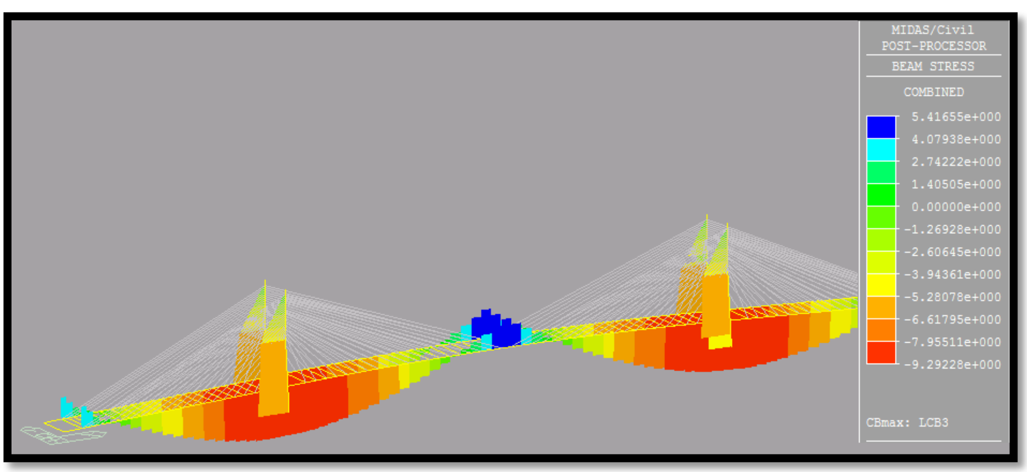


Figure (4-14): Cable stress under LCB3 (case 1)



`Figure (4-15): Tower and girder stress under LCB3 (case 1)

Table (4-5): Cable stresses (case 1) under LCB2

Cable number	Stress at end I N/mm^2	Stress at end J N/mm^2
1	3.720e+002	3.769e+002
2	3.417e+002	3.465e+002
3	3.078e+002	3.125e+002
4	2.757e+002	2.802e+002
5	2.423e+002	2.467e+002
6	2.280e+002	2.323e+002
7	2.305e+002	2.347e+002
8	2.414e+002	2.455e+002
9	2.435e+002	2.474e+002
10	2.282e+002	2.320e+002
11	2.100e+002	2.136e+002
12	1.986e+002	2.021e+002
13	1.919e+002	1.954e+002
14	1.797e+002	1.830e+002
15	1.683e+002	1.715e+002
16	1.529e+002	1.559e+002
17	1.438e+002	1.467e+002
18	1.476e+002	1.504e+002
19	1.347e+002	1.373e+002
20	5.818e+001	6.067e+001
21	4.930e+001	5.169e+001
22	1.203e+002	1.228e+002
23	1.363e+002	1.388e+002
24	1.312e+002	1.338e+002
25	1.397e+002	1.424e+002
26	1.562e+002	1.590e+002
27	1.675e+002	1.703e+002
28	1.786e+002	1.815e+002
29	1.914e+002	1.945e+002
30	2.025e+002	2.057e+002
31	2.146e+002	2.178e+002
32	2.265e+002	2.299e+002
33	2.377e+002	2.411e+002
34	2.494e+002	2.529e+002
35	2.608e+002	2.643e+002
36	2.695e+002	2.732e+002
37	2.822e+002	2.860e+002
38	2.987e+002	3.026e+002
39	3.137e+002	3.177e+002
40	3.130e+002	3.171e+002

Table (4-6): Cable stress under LCB3

Cable number	Stress at end I N/mm^2	Stress at end J N/mm^2
1	4.036e+002	4.085e+002
2	3.722e+002	3.770e+002
3	3.364e+002	3.411e+002
4	3.018e+002	3.063e+002
5	2.637e+002	2.682e+002
6	2.456e+002	2.500e+002
7	2.486e+002	2.528e+002
8	2.600e+002	2.641e+002
9	2.624e+002	2.663e+002
10	2.474e+002	2.512e+002
11	2.296e+002	2.333e+002
12	2.182e+002	2.218e+002
13	2.113e+002	2.147e+002
14	1.984e+002	2.017e+002
15	1.864e+002	1.896e+002
16	1.704e+002	1.734e+002
17	1.608e+002	1.637e+002
18	1.644e+002	1.671e+002
19	1.515e+002	1.542e+002
20	7.486e+001	7.736e+001
21	6.596e+001	6.835e+001
22	1.369e+002	1.394e+002
23	1.527e+002	1.552e+002
24	1.479e+002	1.506e+002
25	1.568e+002	1.595e+002
26	1.738e+002	1.766e+002
27	1.855e+002	1.883e+002
28	1.970e+002	1.999e+002
29	2.104e+002	2.134e+002
30	2.220e+002	2.251e+002
31	2.345e+002	2.378e+002
32	2.469e+002	2.502e+002
33	2.583e+002	2.617e+002
34	2.700e+002	2.735e+002
35	2.814e+002	2.850e+002
36	2.903e+002	2.940e+002
37	3.032e+002	3.070e+002
38	3.201e+002	3.240e+002
39	3.359e+002	3.398e+002
40	3.365e+002	3.406e+002

- The values of cable stress for LCB2 according to case 2 and case 3 are shown in Figures (4-16) and (4-17) and Tables (4-7) and (4-8) respectively. The maximum value of cable stress obtained from case 2 is 407 N/mm^2 occurring in the first cable .Also, the maximum value obtained from case 3 is 355 N/mm^2 in cable 1.

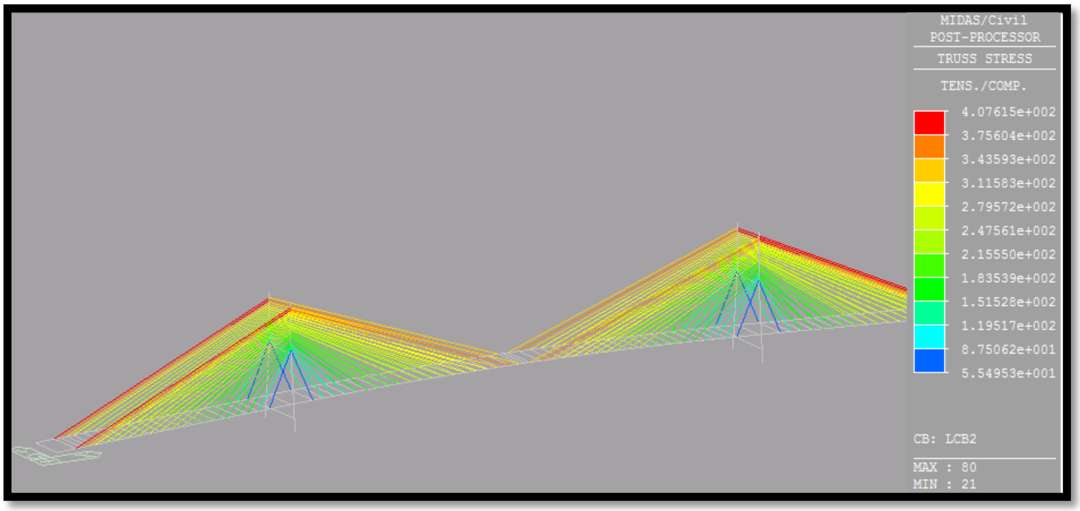


Figure (4-16): Cable stress under LCB2 (case 2)

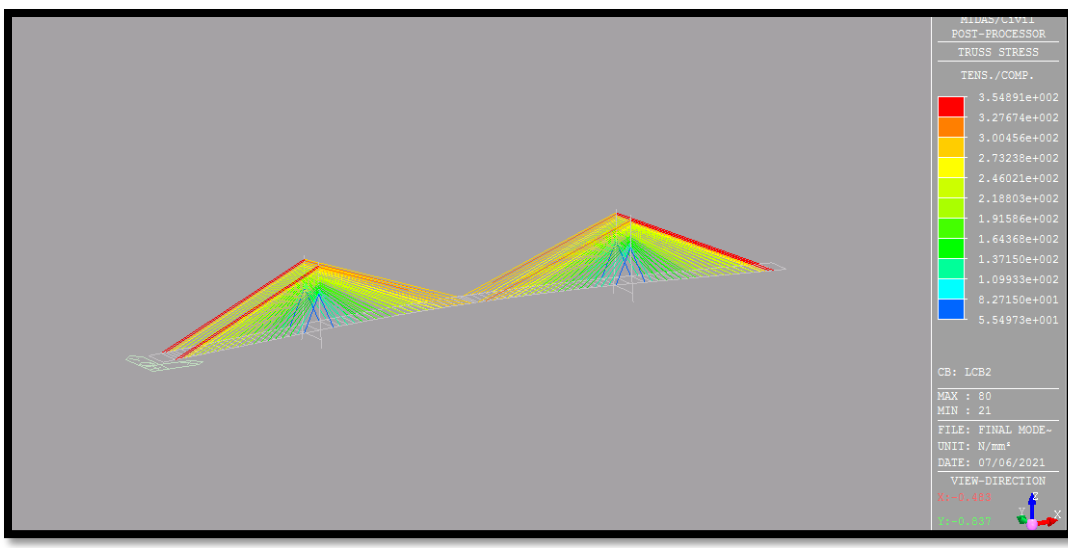


Figure (4-17): Cable stress under LCB2 (case 3)

Table (4-7): Cable stresses (case 2) under LCB2

Cable number	Stress at end I N/mm^2	Stress at end J N/mm^2
1	4.021e+002	4.073e+002
2	3.712e+002	3.762e+002
3	3.360e+002	3.409e+002
4	3.032e+002	3.079e+002
5	2.675e+002	2.721e+002
6	2.503e+002	2.548e+002
7	2.509e+002	2.552e+002
8	2.609e+002	2.651e+002
9	2.633e+002	2.674e+002
10	2.496e+002	2.535e+002
11	2.314e+002	2.352e+002
12	2.184e+002	2.221e+002
13	2.072e+002	2.107e+002
14	1.962e+002	1.996e+002
15	1.821e+002	1.854e+002
16	1.681e+002	1.713e+002
17	1.584e+002	1.614e+002
18	1.613e+002	1.642e+002
19	1.453e+002	1.481e+002
20	6.460e+001	6.721e+001
21	5.550e+001	5.799e+001
22	1.304e+002	1.329e+002
23	1.470e+002	1.497e+002
24	1.462e+002	1.489e+002
25	1.519e+002	1.547e+002
26	1.687e+002	1.716e+002
27	1.843e+002	1.873e+002
28	1.948e+002	1.979e+002
29	2.073e+002	2.105e+002
30	2.234e+002	2.267e+002
31	2.343e+002	2.377e+002
32	2.451e+002	2.485e+002
33	2.597e+002	2.632e+002
34	2.744e+002	2.780e+002
35	2.816e+002	2.854e+002
36	2.914e+002	2.953e+002
37	3.097e+002	3.136e+002
38	3.306e+002	3.347e+002
39	3.437e+002	3.478e+002
40	3.360e+002	3.402e+002

Table (4-8): Cable stresses (case 3) under LCB2

Cable number	Stress at end I N/mm2	Stress at end J N/mm^2
1	3.536e+002	3.546e+002
2	3.277e+002	3.287e+002
3	2.991e+002	3.001e+002
4	2.708e+002	2.718e+002
5	2.393e+002	2.402e+002
6	2.215e+002	2.224e+002
7	2.196e+002	2.205e+002
8	2.289e+002	2.298e+002
9	2.359e+002	2.367e+002
10	2.255e+002	2.263e+002
11	2.087e+002	2.094e+002
12	1.963e+002	1.970e+002
13	1.883e+002	1.890e+002
14	1.751e+002	1.758e+002
15	1.636e+002	1.642e+002
16	1.531e+002	1.538e+002
17	1.462e+002	1.468e+002
18	1.454e+002	1.460e+002
19	1.301e+002	1.306e+002
20	6.233e+001	6.284e+001
21	5.550e+001	5.598e+001
22	1.167e+002	1.172e+002
23	1.314e+002	1.320e+002
24	1.340e+002	1.345e+002
25	1.403e+002	1.408e+002
26	1.493e+002	1.499e+002
27	1.674e+002	1.680e+002
28	1.824e+002	1.830e+002
29	1.857e+002	1.863e+002
30	1.952e+002	1.958e+002
31	2.057e+002	2.063e+002
32	2.226e+002	2.233e+002
33	2.351e+002	2.358e+002
34	2.409e+002	2.416e+002
35	2.467e+002	2.474e+002
36	2.617e+002	2.624e+002
37	2.789e+002	2.796e+002
38	2.962e+002	2.970e+002
39	3.001e+002	3.010e+002
40	2.904e+002	2.913e+002

4. Reactions Force

Figures (4-18) and (4-19) and Table (4-9) and (4-10), show the reactions obtained from LCB2 and LCB3 respectively. The maximum reaction value occurs at node 130.

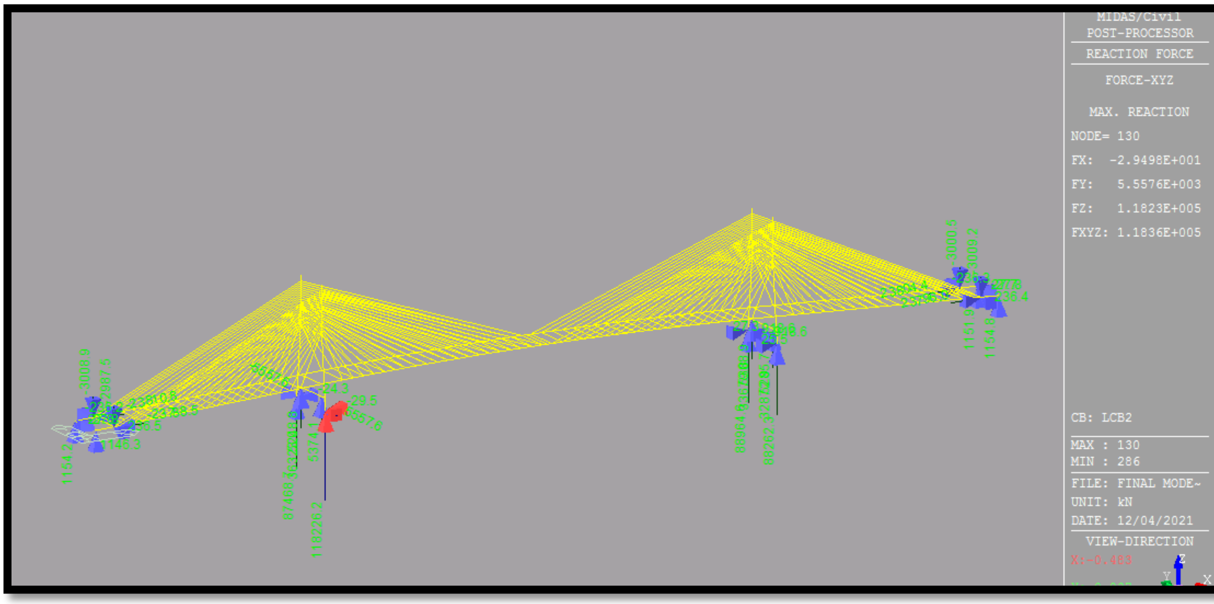


Figure (4-18): Reaction under LCB2

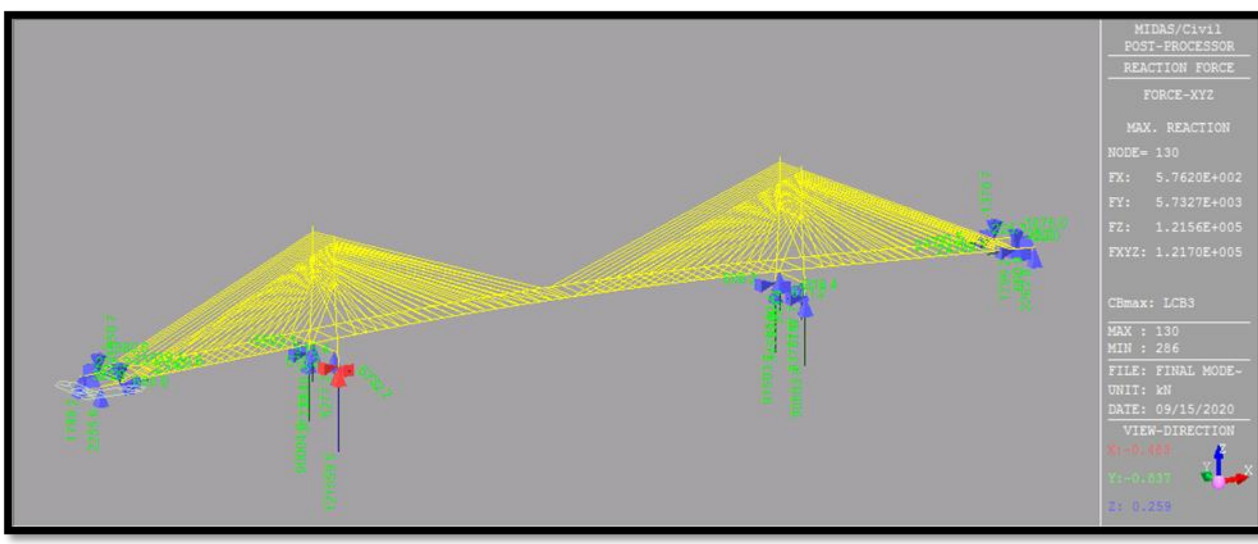


Figure (4-19): Reaction under LCB3

Table (4-9): Reaction under LCB2

Node	Load	FX (kN)	FY (kN)	FZ (kN)	MX (kN·m)	MY (kN·m)	MZ (kN·m)
43	LCB2	0.000000	-27.849898	1146.343448	4183.879236	0.000000	0.109010
129	LCB2	0.000000	-27.815974	1154.753297	4194.706797	0.000000	0.491235
130	LCB2	-29.498443	5557.557042	118226.168038	48150.50390	-4667.05803	-40.488323
131	LCB2	25.537211	-918.615418	88262.346883	9764.255331	4624.160777	22.872343
256	LCB2	0.000000	27.624923	1154.186605	-4201.71697	0.000000	0.659207
260	LCB2	0.000000	27.658847	1151.889753	-4190.88941	0.000000	-0.058962
266	LCB2	-24.324381	-5557.557042	87468.696413	28313.57368	-4594.80988	14.701840
268	LCB2	27.309482	918.615418	88964.552827	-10899.6748	4649.048406	-33.808622
270	LCB2	0.000000	0.000000	32872.885652	0.000000	0.000000	0.000000
271	LCB2	0.000000	0.000000	36323.026488	0.000000	0.000000	0.000000
272	LCB2	0.000000	0.000000	33679.824389	0.000000	0.000000	0.000000
276	LCB2	0.000000	0.000000	5374.149376	0.000000	0.000000	0.000000
277	LCB2	0.000000	0.000000	5248.845260	0.000000	0.000000	0.000000
278	LCB2	0.000000	0.000000	5285.712927	0.000000	0.000000	0.000000
279	LCB2	0.000000	0.000000	5308.917796	0.000000	0.000000	0.000000
284	LCB2	-23788.533924	236.472264	-2987.523260	0.000000	0.000000	0.000000
285	LCB2	23795.966864	236.409417	-3009.220080	0.000000	0.000000	0.000000
286	LCB2	-23810.829552	-236.218365	-3008.919342	0.000000	0.000000	0.000000
287	LCB2	23804.372743	-236.281212	-3000.533764	0.000000	0.000000	0.000000

Table (4-10): Reaction under LCB3

Node	Load	FX (kN)	FY (kN)	FZ (kN)	MX (kN·m)	MY (kN·m)	MZ (kN·m)
43	LCB3(ma	0.000000	-21.931397	2255.578573	17887.81123	0.000000	407.842854
129	LCB3(ma	0.000000	-22.007026	2262.849672	17898.73979	0.000000	346.233485
130	LCB3(ma	576.204744	5732.676136	121559.536288	50087.09090	9010.699961	1100.626052
131	LCB3(ma	671.230399	-918.361487	90803.872633	10654.99964	19929.22777	1079.697593
256	LCB3(ma	0.000000	35.238751	1799.188917	-3973.57304	0.000000	31.764427
260	LCB3(ma	0.000000	35.163748	1796.179441	-3962.64014	0.000000	319.322101
266	LCB3(ma	515.369244	-5503.095558	90004.916913	29878.56881	8190.478113	1196.127215
268	LCB3(ma	608.203607	923.353998	91503.249327	-10392.7657	19096.60640	1184.477253
270	LCB3(ma	0.000000	0.000000	33787.035464	0.000000	0.000000	0.000000
271	LCB3(ma	0.000000	0.000000	37232.093300	0.000000	0.000000	0.000000
272	LCB3(ma	0.000000	0.000000	34595.846389	0.000000	0.000000	0.000000
276	LCB3(ma	0.000000	0.000000	6277.157438	0.000000	0.000000	0.000000
277	LCB3(ma	0.000000	0.000000	5640.317447	0.000000	0.000000	0.000000
278	LCB3(ma	0.000000	0.000000	6187.719177	0.000000	0.000000	0.000000
279	LCB3(ma	0.000000	0.000000	5700.425609	0.000000	0.000000	0.000000
284	LCB3(ma	-23400.571518	660.869045	-1050.669760	0.000000	0.000000	0.000000
285	LCB3(ma	24799.059739	660.871385	-1076.001580	0.000000	0.000000	0.000000
286	LCB3(ma	-23539.061833	-225.155773	-1380.596342	0.000000	0.000000	0.000000
287	LCB3(ma	24730.530305	-225.108452	-1370.704389	0.000000	0.000000	0.000000

4.2.2 Dynamic analysis

In dynamic analysis, Eigen value analysis method was used to find the natural frequencies of the cable stayed bridge. Modal analysis was conducted based on linear Eigen value analysis, where the initial conditions were assigned to estimate as suggested by Vikas et al, (2013).

Figures (4-20) to (4-39) show the variation in fundamental frequency, the minimum natural frequency was obtained at the first mode, while maximum natural frequency in the structure occurred at 20 mode .Also the maximum value of the natural period was in mode 1 while the minimum value was obtained at mode 20.

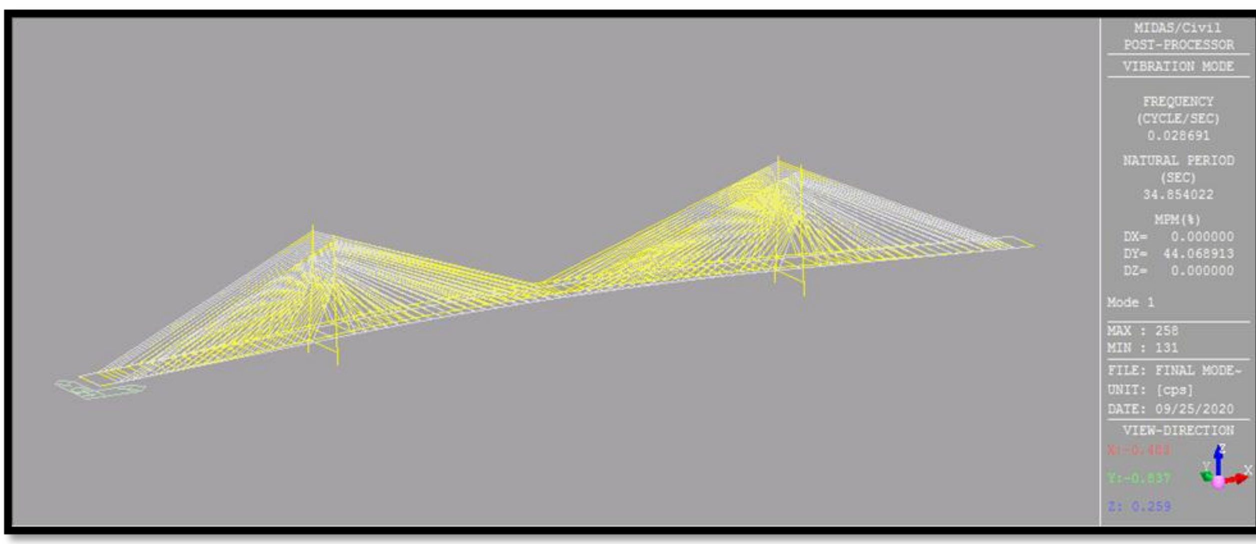


Figure (4-20): variation mode (mode 1)

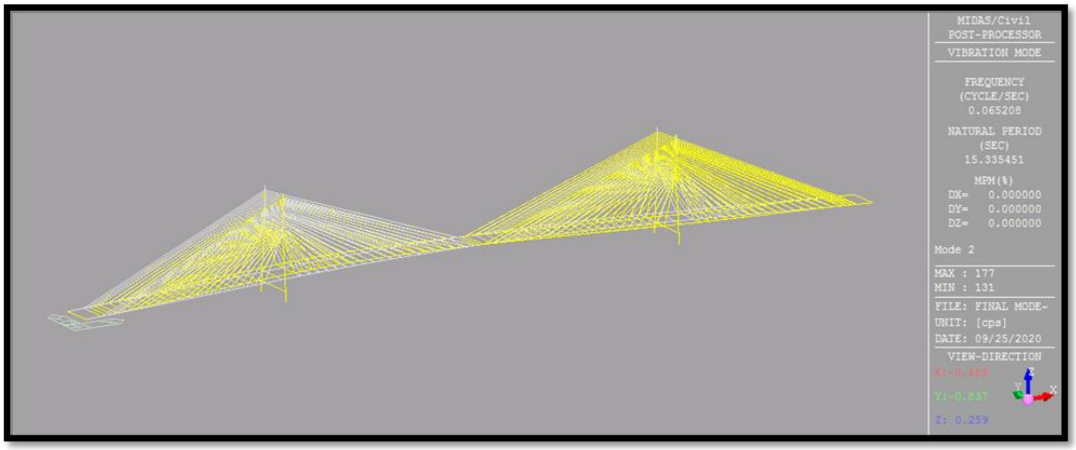


Figure (4-21): variation mode (mode 2)

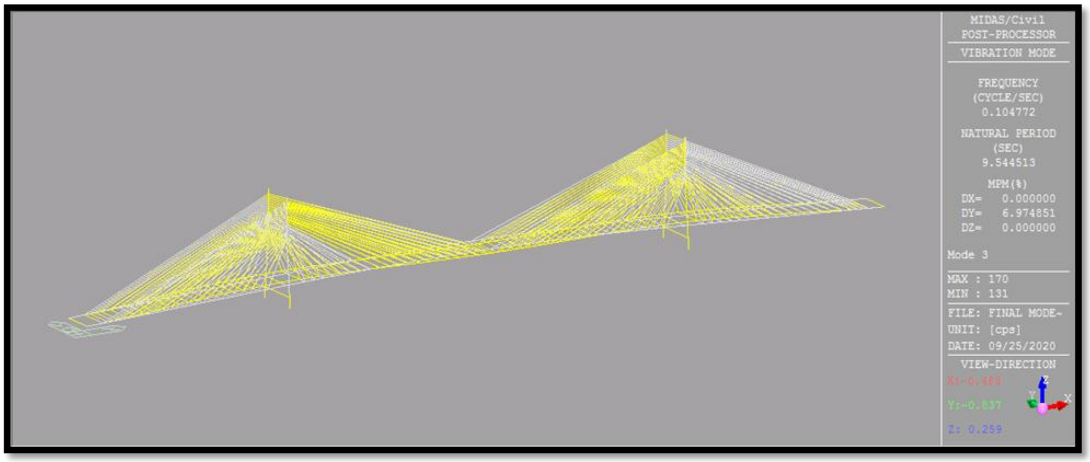


Figure (4-22): variation mode (mode 3)

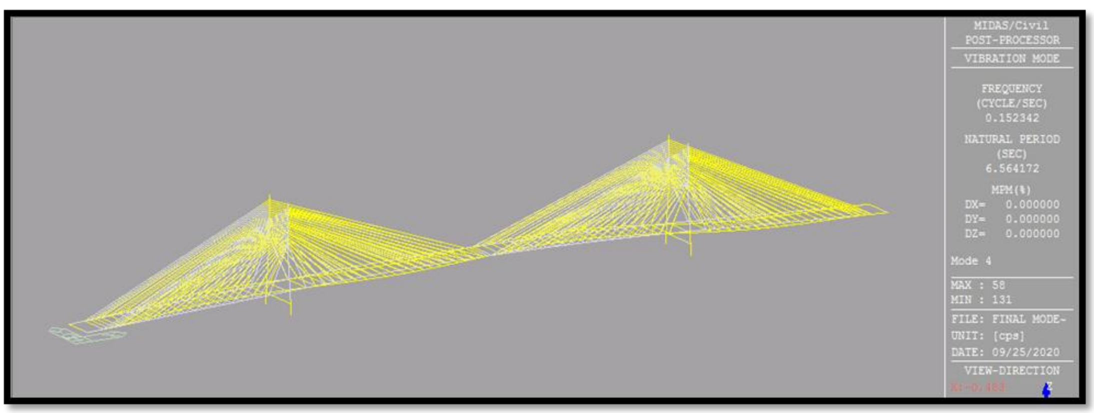


Figure (4-23): variation mode (mode 4).

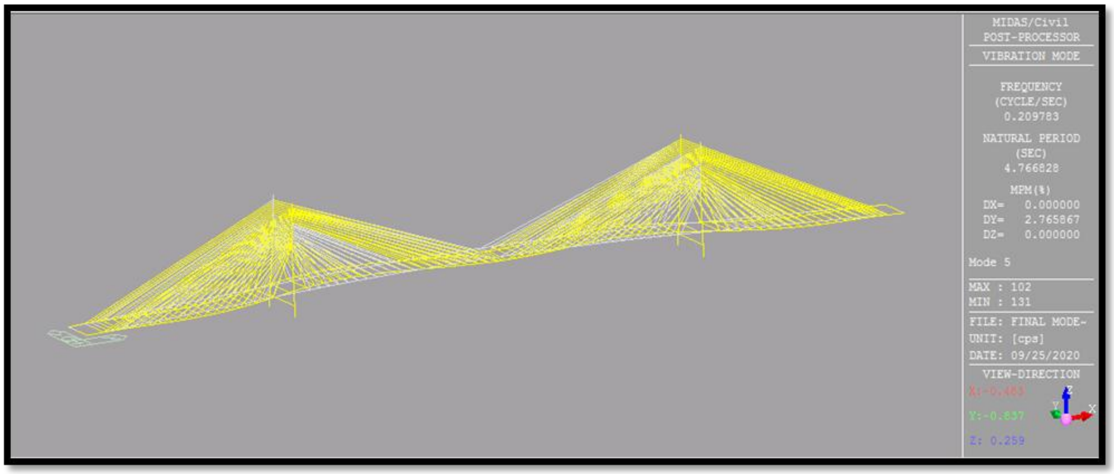


Figure (4-24): variation mode (mode 5)

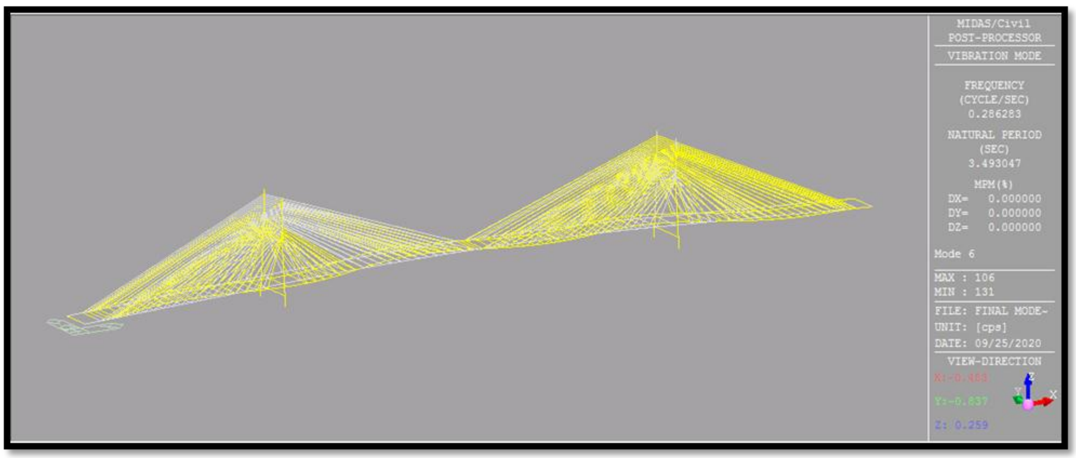


Figure (4-25): variation mode (mode 6)

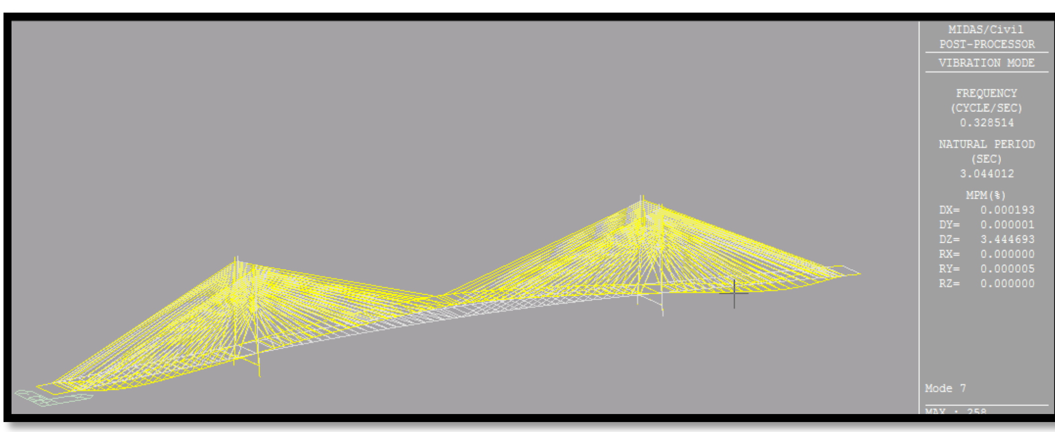


Figure (4-26): variation mode (mode 7)

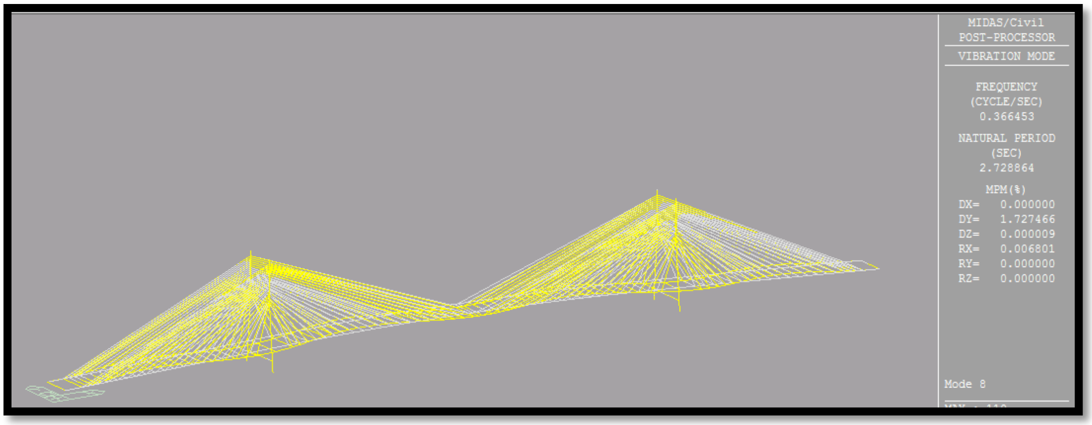


Figure (4-27): variation mode (mode 8)

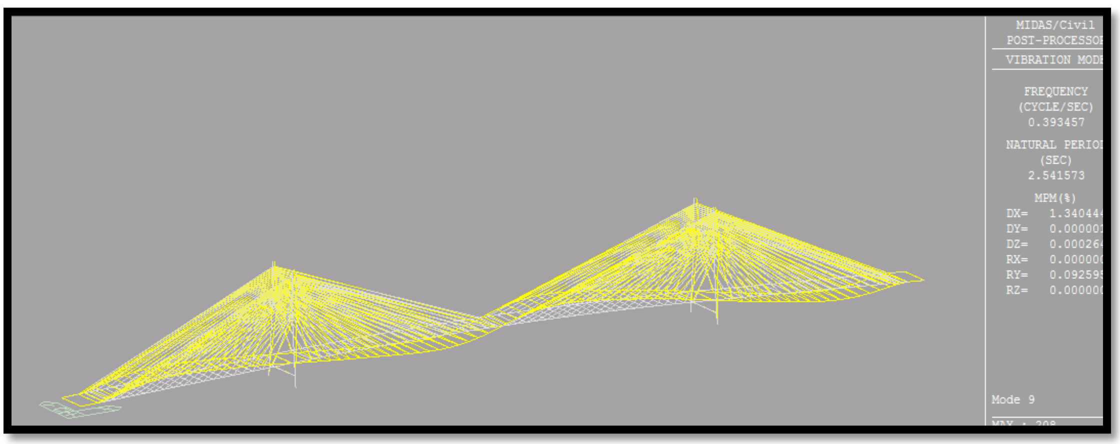


Figure (4-28): variation mode (mode 9)

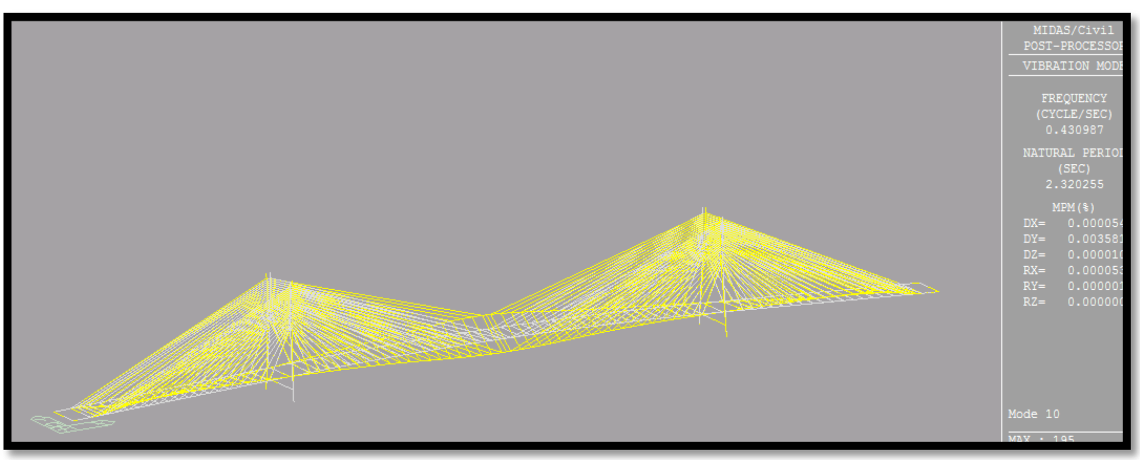


Figure (4-29): variation mode (mode 10)

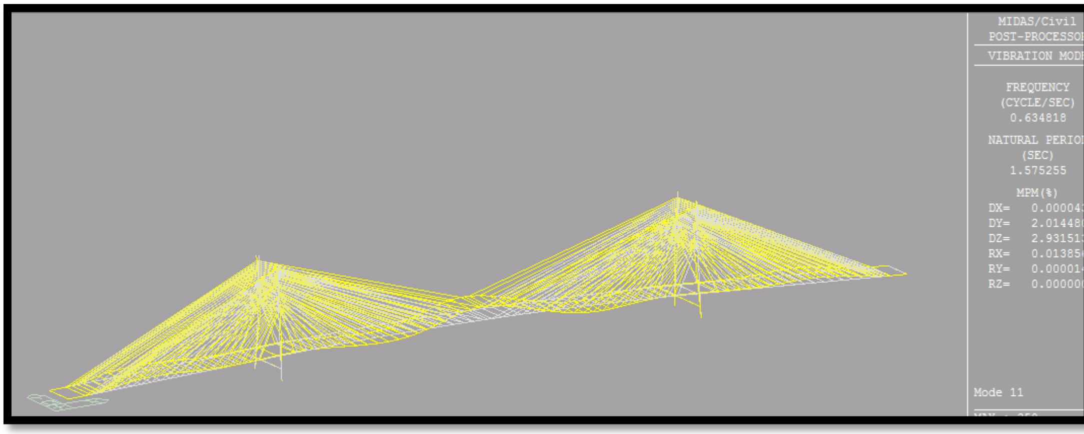


Figure (4-30): variation mode (mode 11)

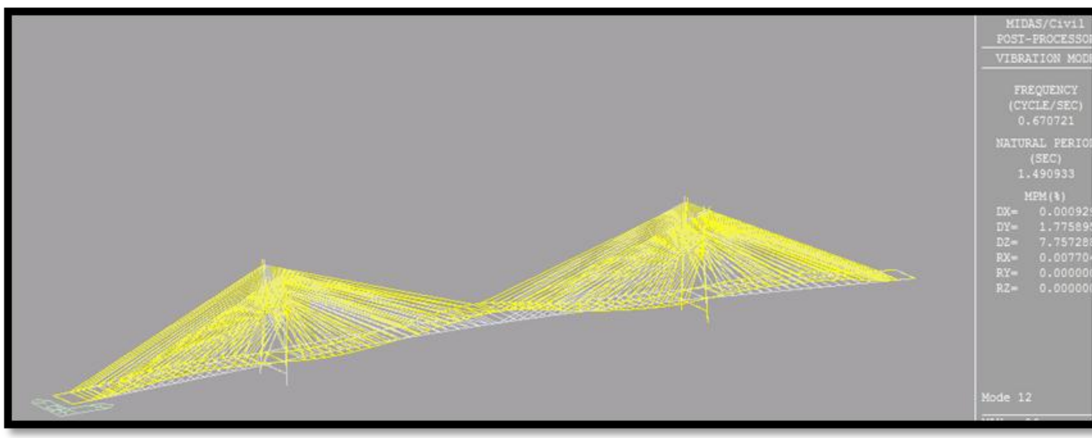


Figure (4-31): variation mode (mode 12)

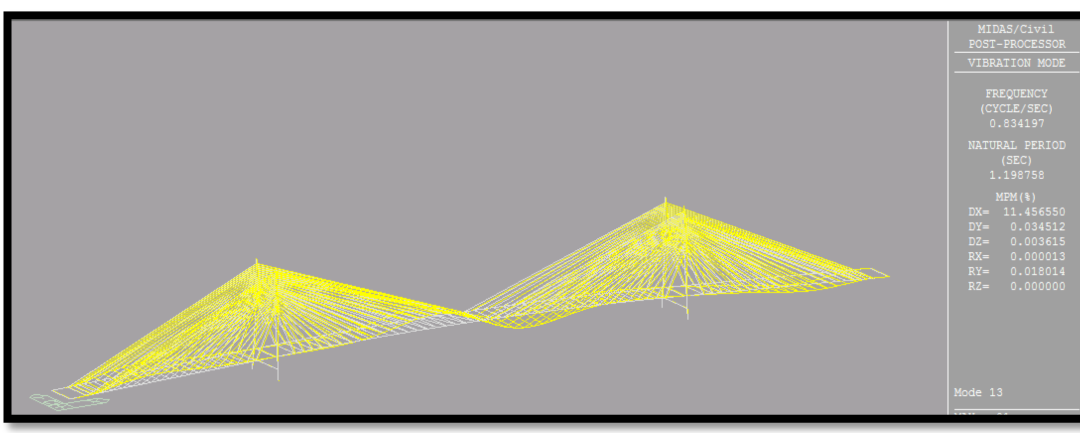


Figure (4-32): variation mode (mode 13)

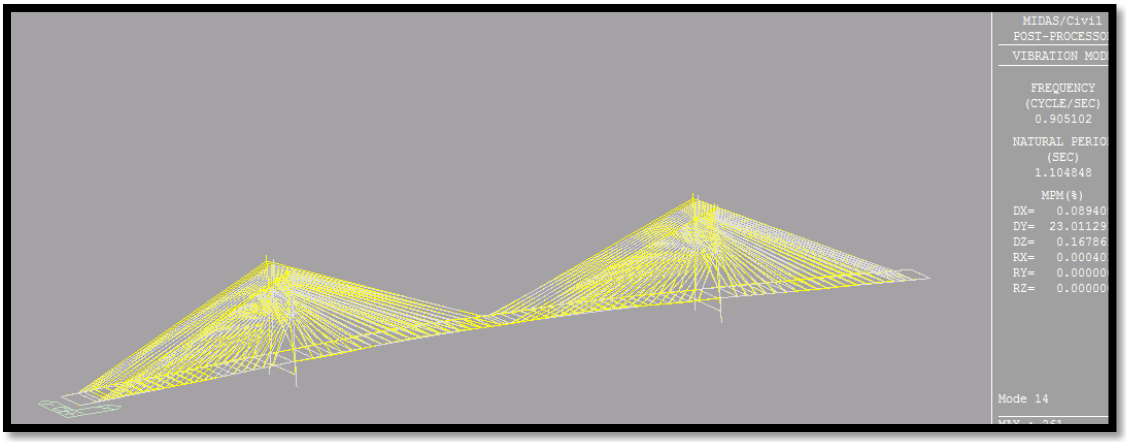


Figure (4-33): variation mode (mode 14)

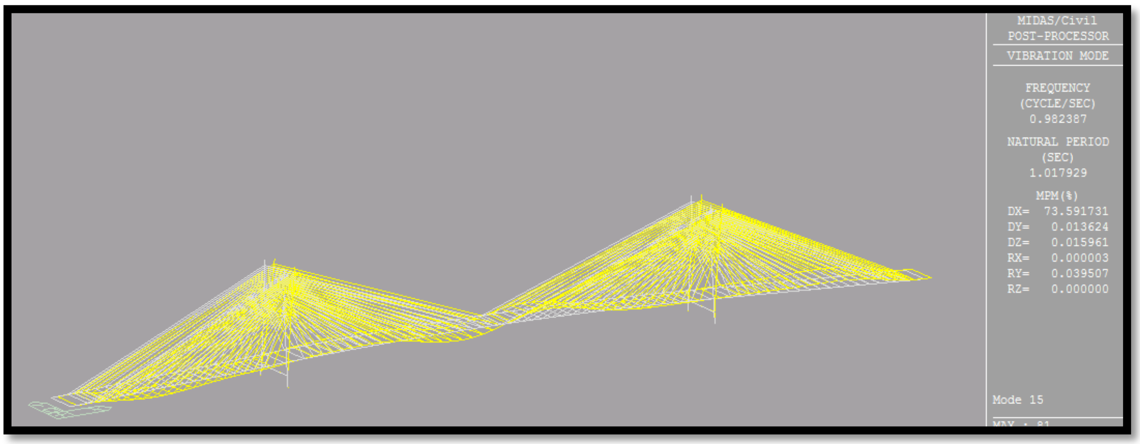


Figure (4-34): variation mode (mode 15)

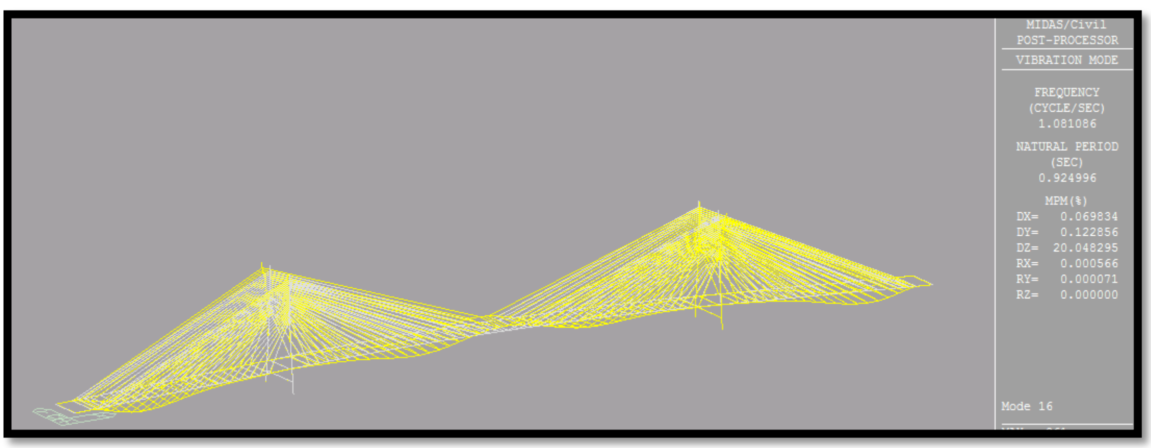


Figure (4-35): variation mode (mode 16)

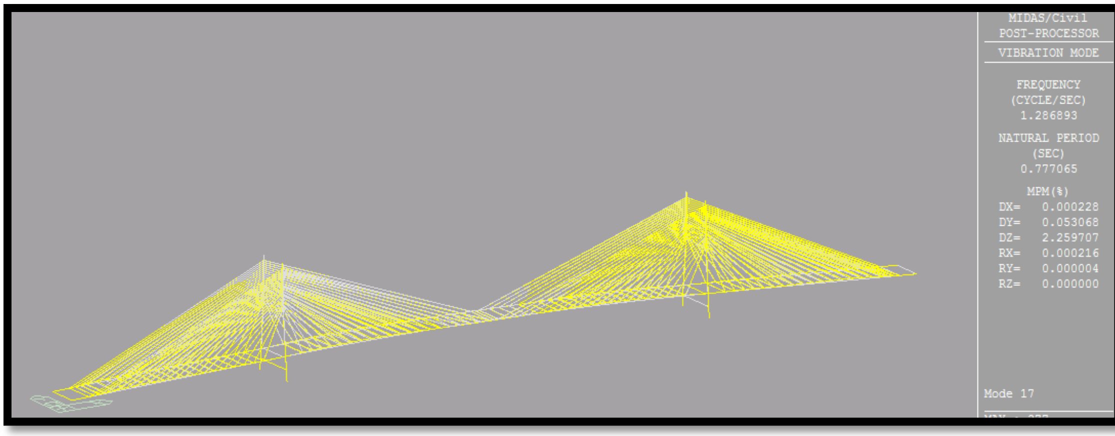


Figure (4-36): variation mode (mode 17)

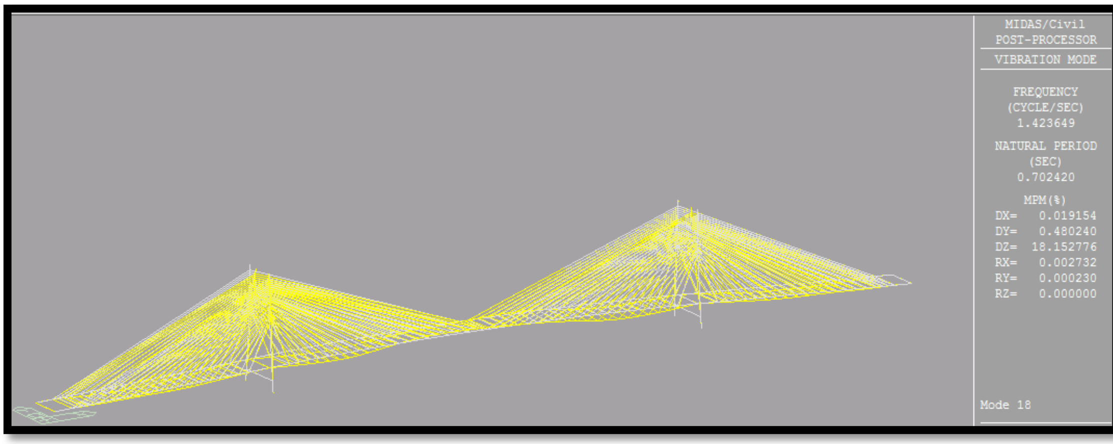


Figure (4-37): variation mode (mode 18)

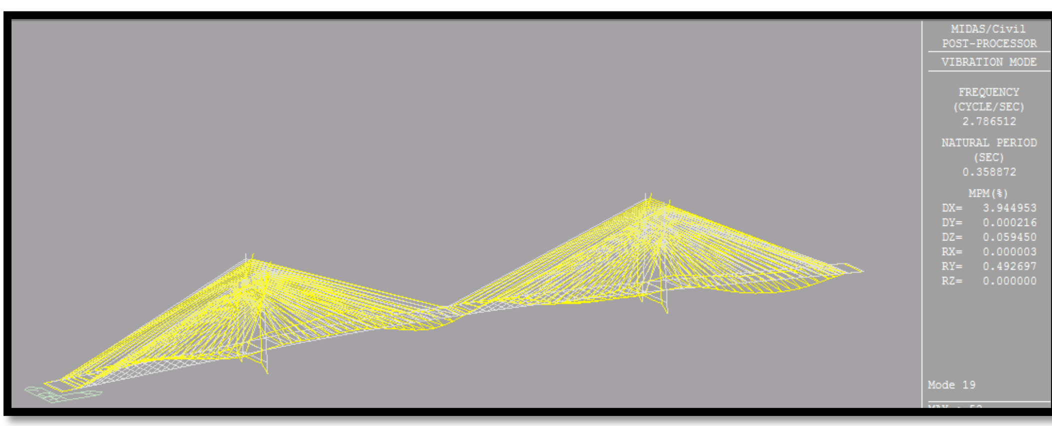


Figure (4-38): variation mode (mode 19)

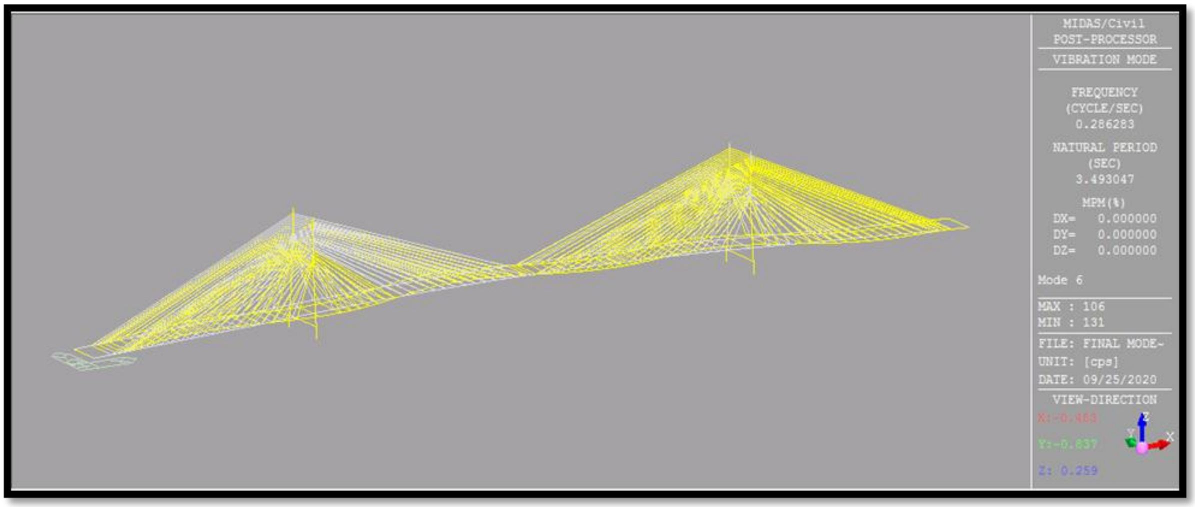


Figure (4-39): variation mode (mode 20)

- Table (4-11), shows the variation of the frequency (cycle /sec) corresponding to the variation of natural period (sec) for the numbers from 1 to 20. The maximum value of the frequency 36.7896 (cycle /sec) occurring at mode 20 while the minimum value was 0.02869 (cycle /sec) occur at mode 1. Also, the value of the natural period is varies from 34.854022 (sec) at mode 1to 0.027182 (sec) at mode 20.It is noticeable that the obtained values showed that the damping is small and can be ignored.

Table (4-11): Mode frequency and period (case1)

Mode No	Frequency(cycle/sec)	Period(sec)
1	0.028691	34.854022
2	0.065208	15.335431
3	0.104772	9.544512
4	0.152343	6.564124
5	0.209784	4.766812
6	0.289046	3.459655
7	0.328514	3.044012
8	0.366453	2.728864
9	0.393457	2.541573
10	0.430987	2.320255
11	0.634818	1.575255
12	0.670721	1.490933
13	0.834197	1.198758
14	0.905102	1.104848
15	0.982387	1.017929
16	1.081086	0.924996
17	1.286893	0.777065
18	1.423649	0.702420
19	2.786512	0.358872
20	36.789682	0.027182

4.2.3 Wind load

In wind load condition, the critical wind velocity was based on the collected information from the internet and the maximum wind velocity of 40 m/s received from the owner, was used in the calculations.

The wind load analysis had been carried out using the same software. The input calculations for wind loads on the cable-stayed bridge model were done manually. The initial pretension analysis for the same model was carried out using MIDAS CIVIL by using the Unknown Load Factor method. These values of the initial pretension are used as an input for the model used for carrying out modal analysis and wind analysis.

Figure (4-40) shows the deformed shape under wind load, Figure (4-41) shows the displacement in the center of girder under wind load and Figure (4-42) shows the displacement in the top tower under wind load

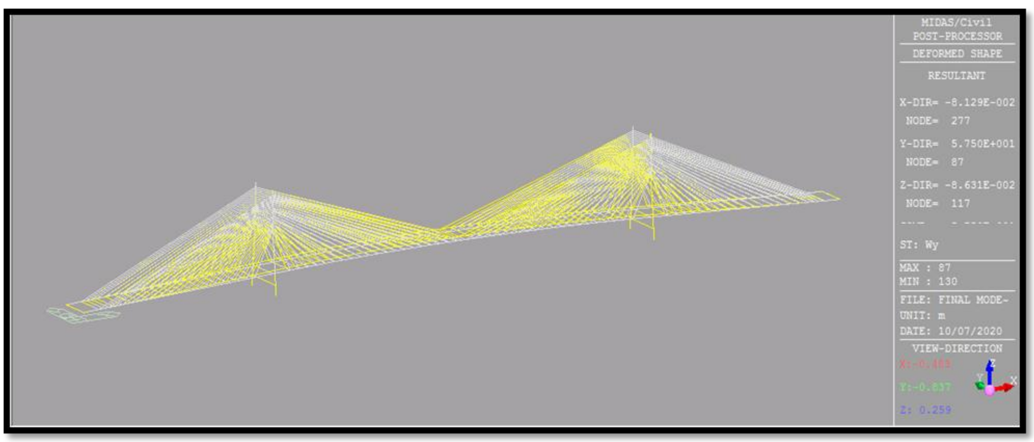


Figure (4-40): Deformed shape under wind load

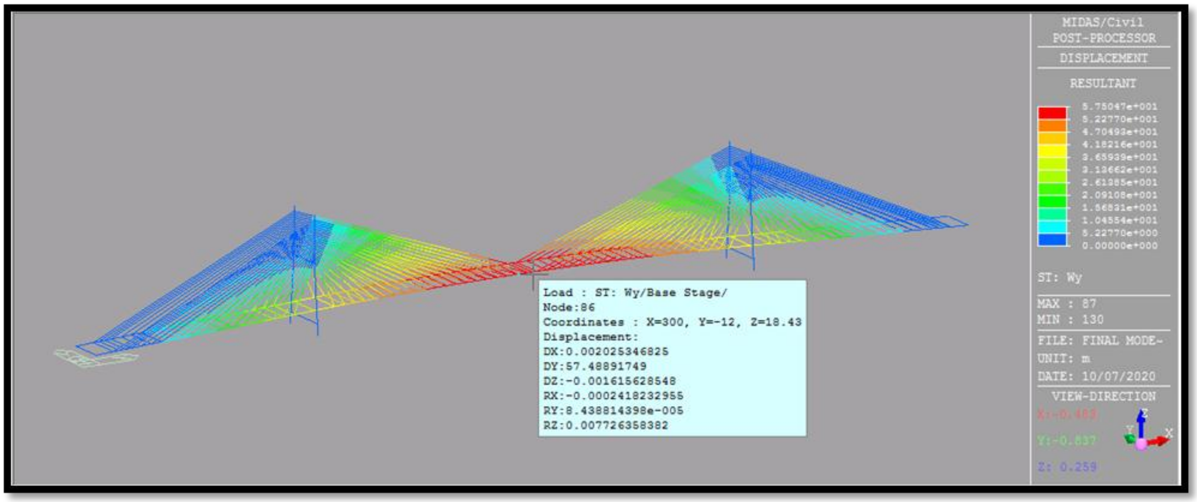


Figure (4-41): Displacement in the center of girder under wind load

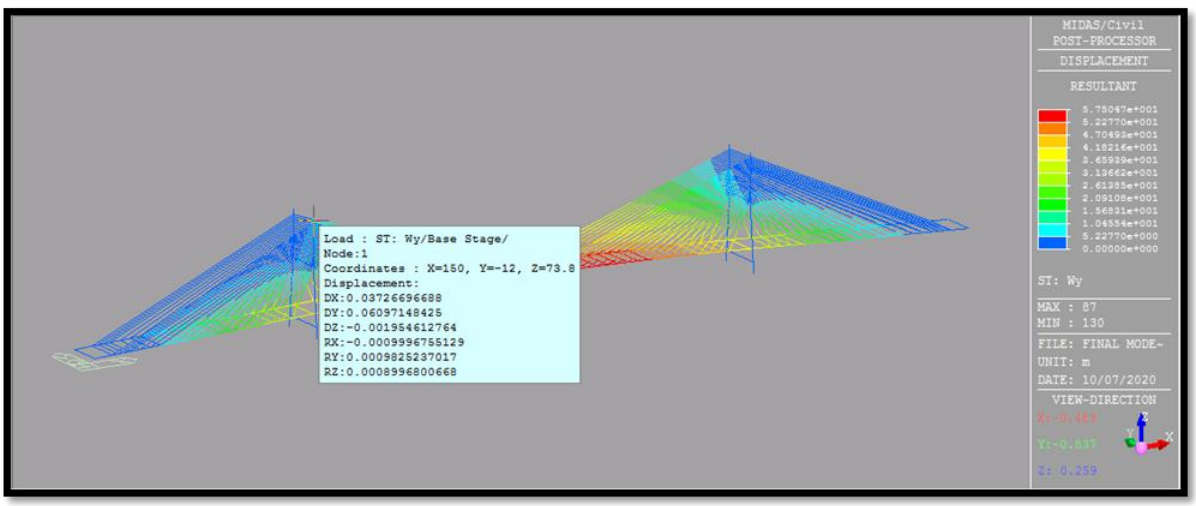


Figure (4-42): Displacement in the top tower under wind load

4.3 Discussion of Results

4.3.1 Cable pretension forces

Figures (4-43) and (4-44) show the pre –tension force under LCB2and LCB3 from case1 respectively. The maximum pre- tension forces are 6670.92 kN, 7229.99 kN in the first outer cable (cable 1) for LCB2 and LCB3 respectively.

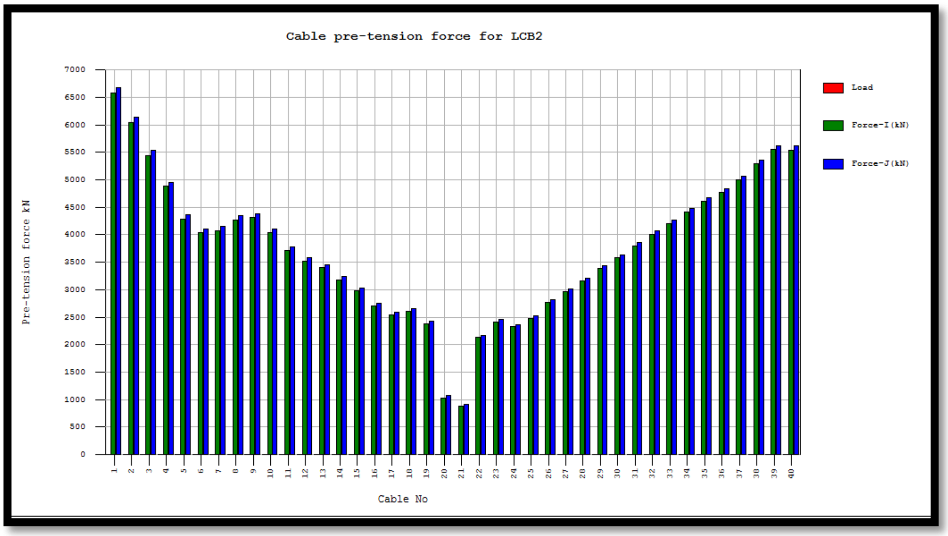


Figure (4-43): Cable pre-tension Force Variation for LCB2 (case 1)

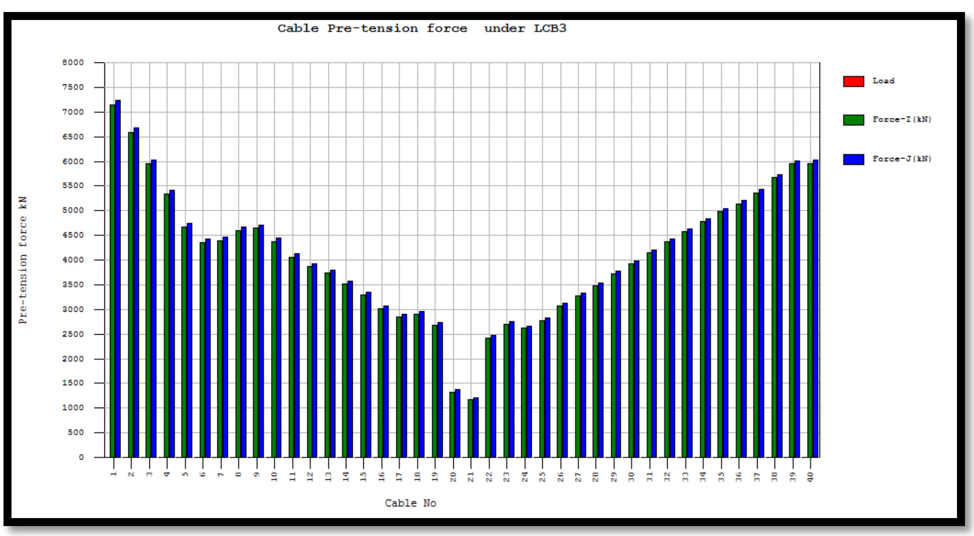


Figure (4-44): Cable pre-tension Force Variation for LCB3(case1)

* In Figures (4-45) and (4-46) the cable pre –tension force Variation Graph under LCB2 for case 2 and case 3 are shown. The maximum value of pre –tension force is 6604.180 kN for case 2 and 6276.7451 kN for case 3.

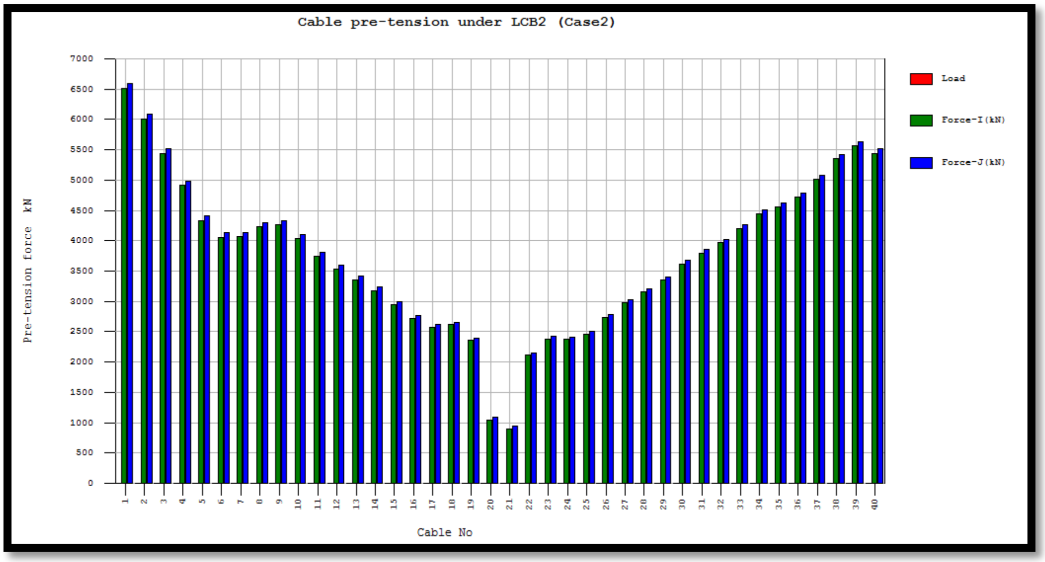


Figure (4-45): Cable pre-tension Force Variation for LCB2 (case 2)

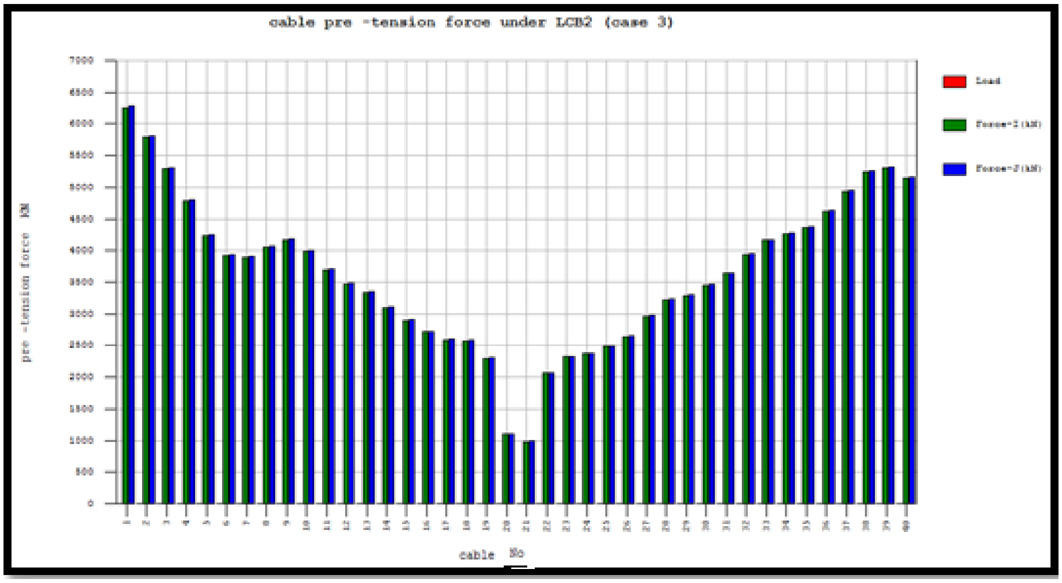


Figure (4-46): Cable pre-tension Force Variation for LCB2 (case 3)

4.3.2 Cable stress

The cable stresses under LCB2 for case 1, case 2and case 3 and under LCB3 for case 1, as shown in Figures (4-47) to (4-50) .The maximum value obtained is 377 N/mm^2 from case 1, is 407 N/mm^{2}from case 2 and' is 355 N/mm^2 from case3and 408.5 N/mm^2 from LCB3. As can be observed form these Figures, the maximum cable stress occurs at the cable with maximum value of pre –tension force.it is noticeable that the value of the stress increases as the value of the pre-tension forces in the cables increases.

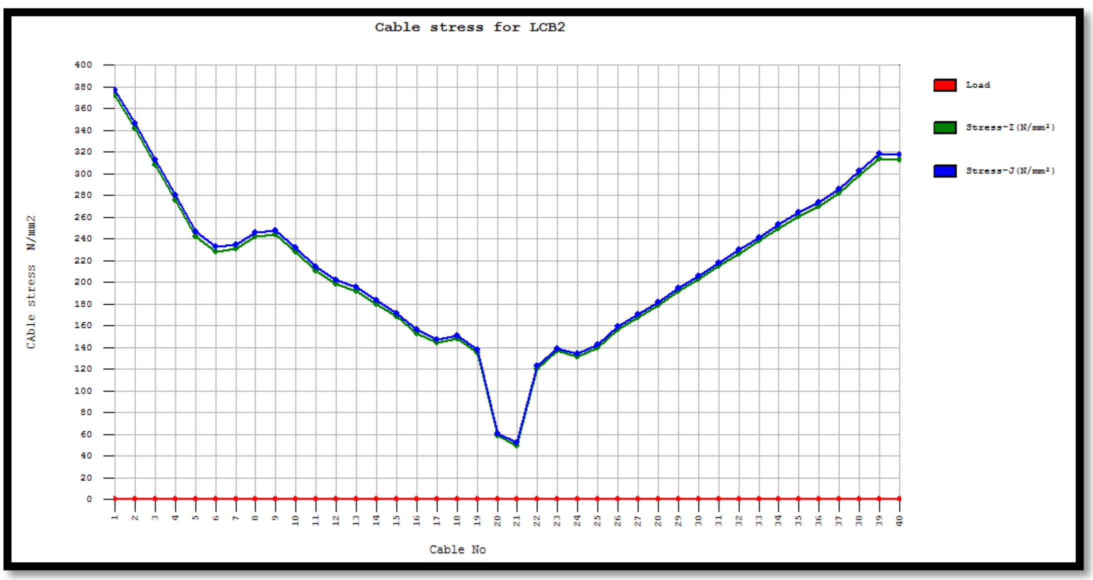


Figure (4-47): Cables stress variation graphic under LCB2 (Case 1)

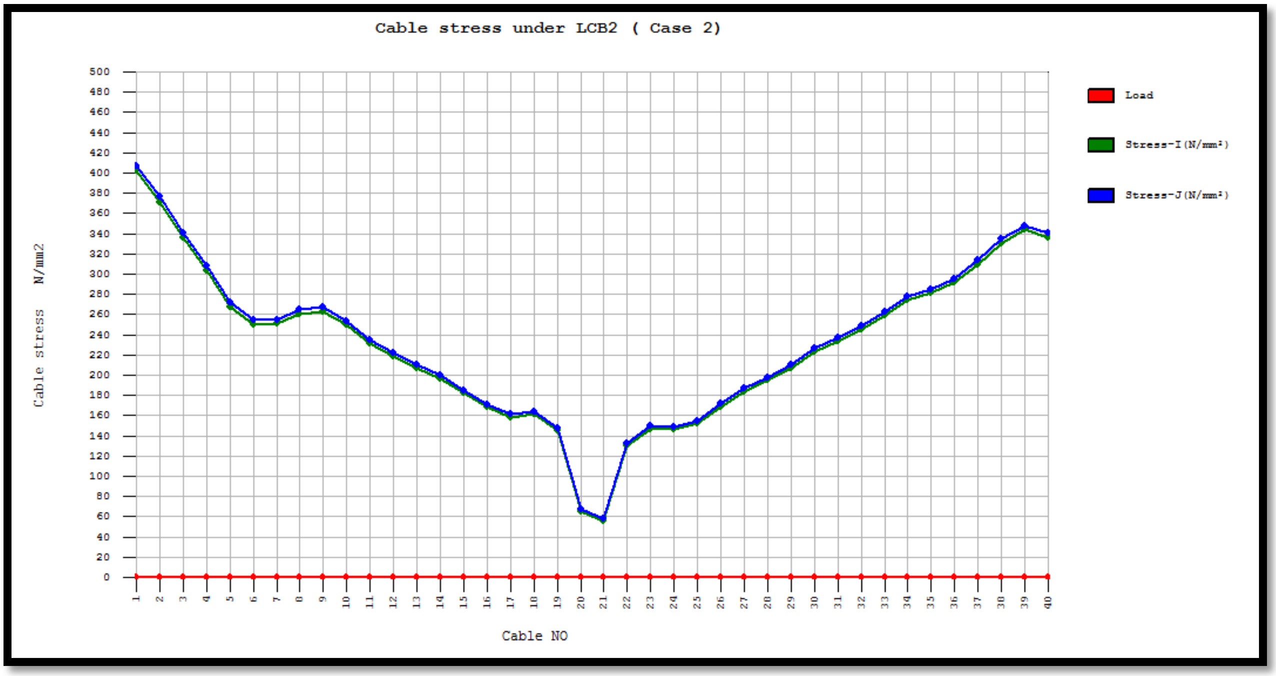


Figure (4-48): Cables stress variation graphic under LCB2 (Case2)

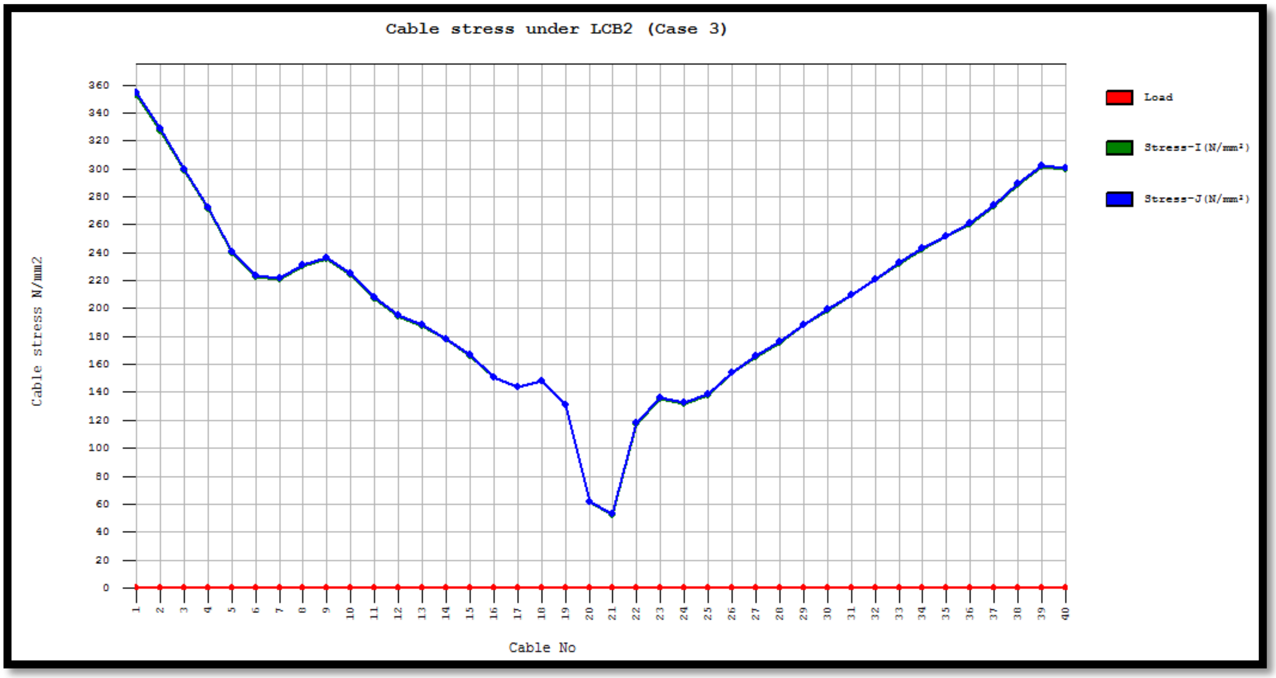


Figure (4-49): Cables stress variation graphic under LCB2 (Case 3)

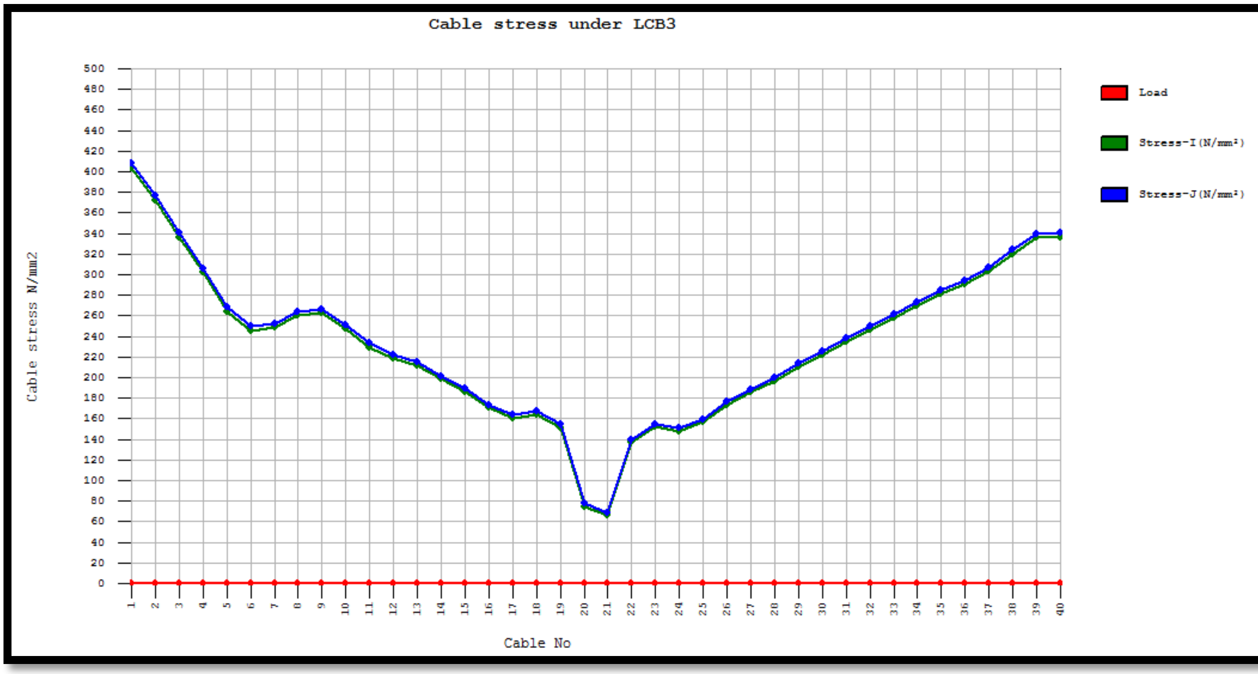


Figure (4-50): Cables stress variation graphic under LCB3

4.3.3 Displacement

Tables (4-12),(4-13),(4-14) and (4-15) show the displacement at top tower, tower-deck joint and center of main girder in X ,y and Z directions under LCB2 for the three cables and under LCB3 for case 1.

Table (4-12): Maximum displacement under LCB2 in the X,y and Z direction (Case 1)

Location	**X (m)**	**Y (m)**	**Z (m)**
Top of tower	0.033308	0.014751	-0.015838
Tower- deck joint	0.022861	-0.000262	0.000000
Center of main girder	0.000044	-0.000330	-0.009990

Table (4-13): Maximum displacement under LCB2 in the X,y and Z direction

(Case 2)

Location	X (m)	Y (m)	Z (m)
Top of tower	0.033166	0.014731	-0.015880
Tower- deck joint	0.022821	-0.000258	0.000000
Center of main girder	0.000044	-0.000327	-0.009310

Table (4-14): Maximum displacement under LCB2 in the X,y and Z direction

(Case 3)

Location	X (m)	Y (m)	Z (m)
Top of tower	0.031438	0.014342	-0.015174
Tower- deck joint	0.017705	-0.000269	-0.008342
Center of main girder	0.000040	-0.000315	-0.008173

Table (4-15): Maximum displacement under LCB3 in the X,y and Z direction

(Case 1)

Location	X (m)	Y (m)	Z (m)
Top of tower	0.060295	0.015899	-0.015763
Tower- deck joint	0.025693	0.001631	0.000000
Center of main girder	0.001961	0.002958	0.026901

4.3.4 Eigenvalue analysis

The minimum natural frequency was obtained at the first mode, while maximum natural frequency in the structure occurred at mode 20. Figure (4-51) shows the frequency of all modes. From Figure (4 -52) which shows the period of mode it can be observed that the maximum value of period is obtained from mode 1 and the minimum value from mode 20.

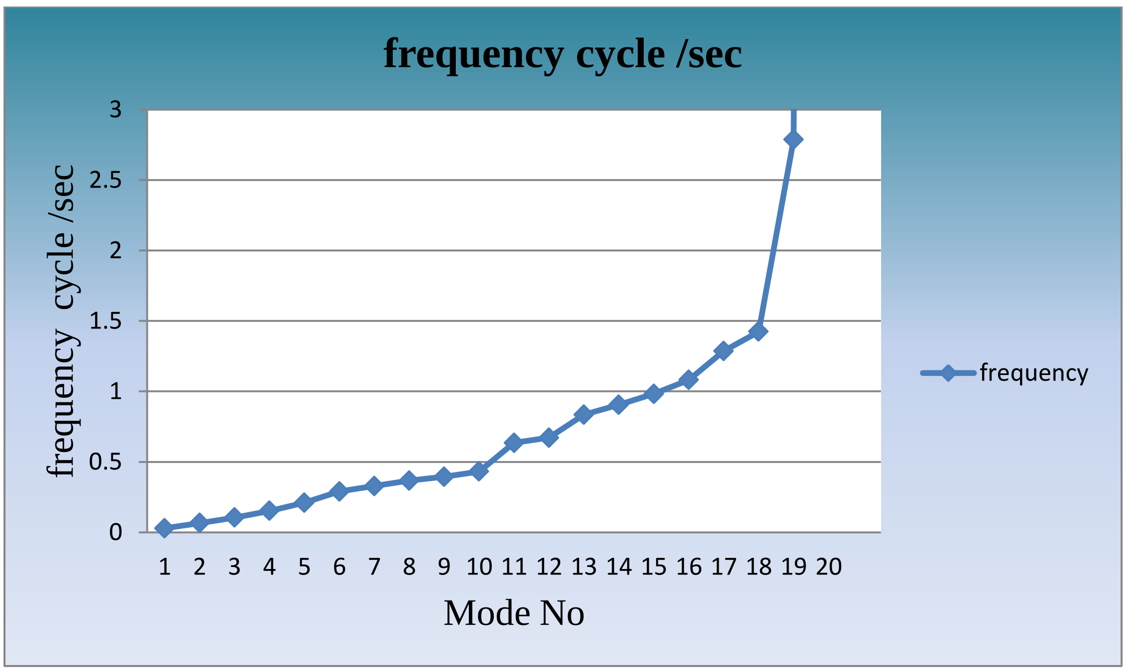


Figure (4-51): Vibration mode – frequency Relation

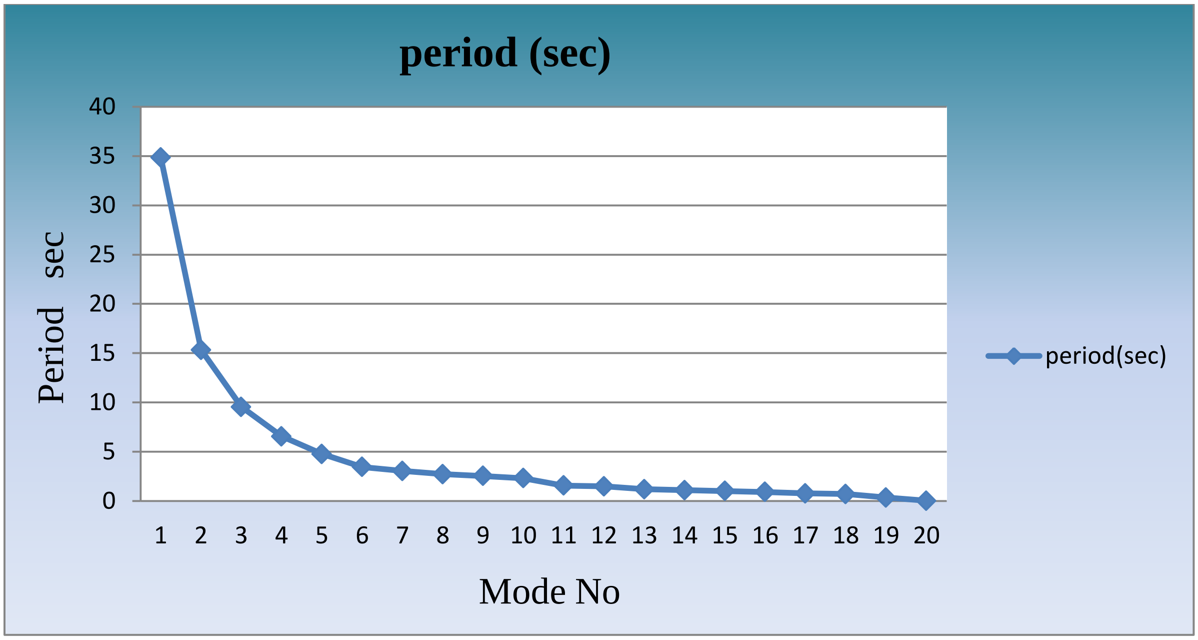


Figure (4-52): Vibration mode – period Relation

4.3.5 Result of wind load analysis

The maximum displacement under applied wind pressure on the structure is as Shawn in Table (4-16).

Table (4-16): Maximum displacement under wind load

Location	**Type of Load**	**X (m)**	**z (m)**
Top of tower	Wind load dir.- y	0.0372	
Center of main girder	Wind load dir.- y		-0.0016

4.4 Summary of Results:

4.1.4 Cable pre- tension force

Figure (4-53) shows the various values of cable pre –tension corresponding to the cable number from case 1and case 2.The maximum value obtained from case 1 is 6670.924041 kN and from case2 is 6604.180 kN. It should be noted that these value are similar. The percentage difference between these values is 1 %.(almost identical).

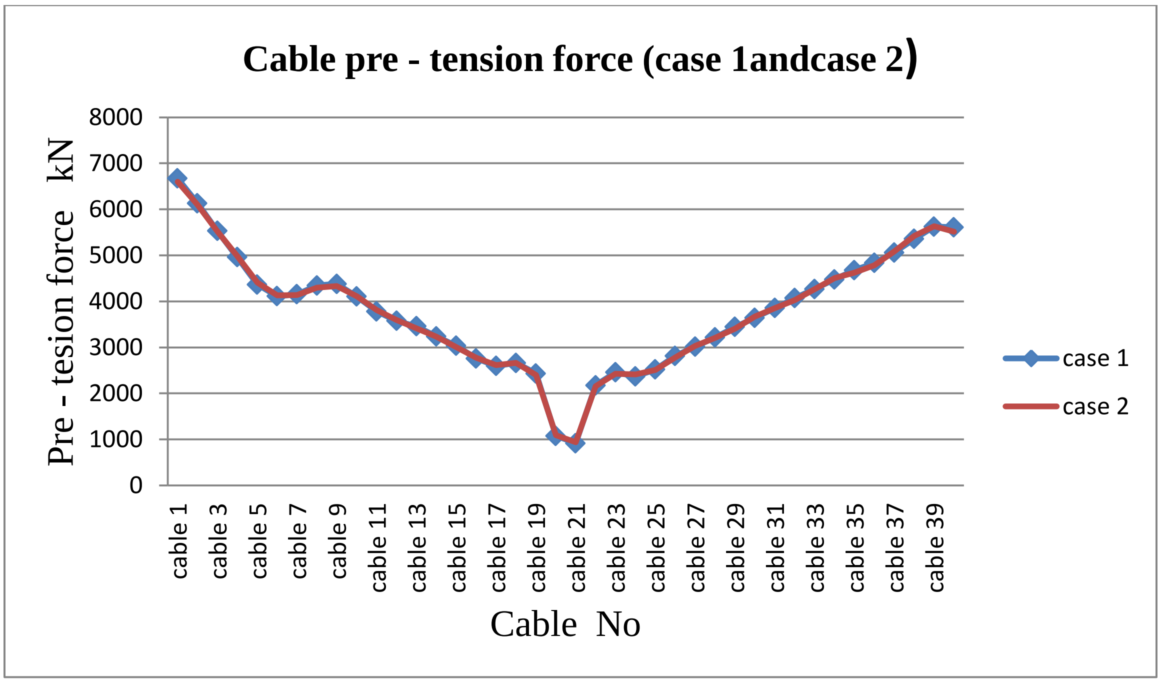


Figure (4-53): Cable pre –tension force (Case 1and Case2)

- The values of cable pre –tension force obtained from case 1and case 3 are illustrated in Figure (4-54).The maximum value of cable pre –tension is 6670.924041 kN from case 1 and 6276.7451 kN from case 3.It is noticeable that these values are obtained differently. The percentage difference between these values is 6%.

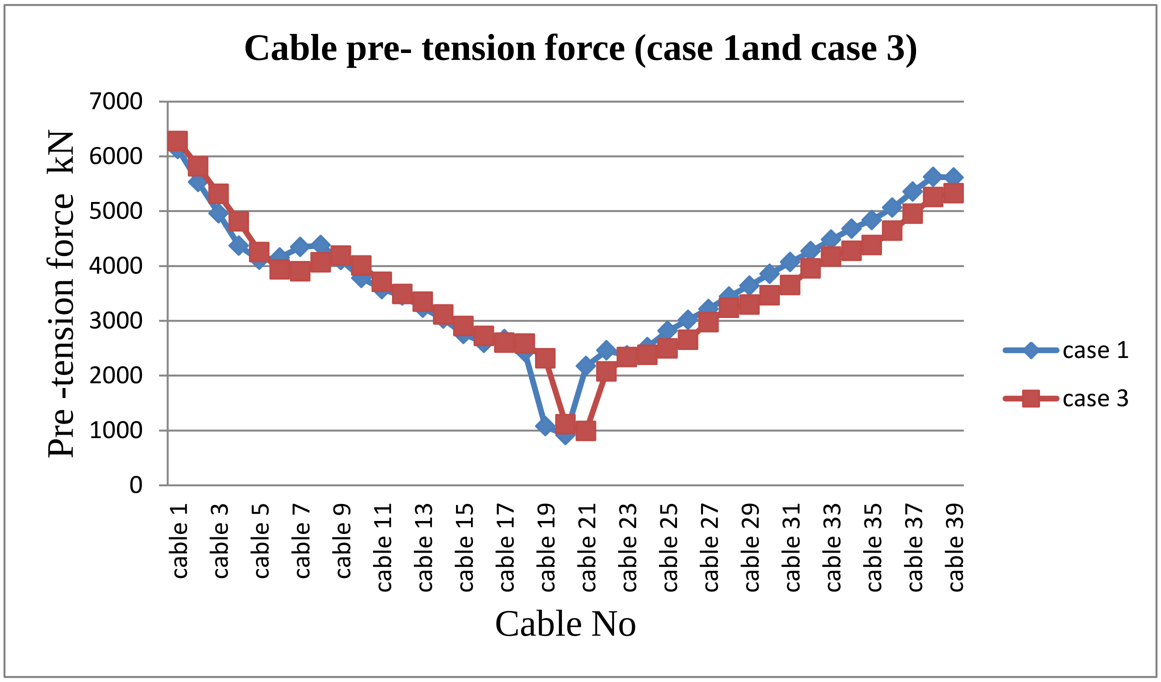


Figure (4-54): Cable pre –tension force (Case 1and Case3)

4.4.1 Result of displacement

Table (4-17), shows the value of maximum displacement under LCB2 for case 1, case 2and case 3, under LCB3 and under wind load. The obtained value of displacement in the tower Top and main girder is compared with the allowable values in AASHTO 2010.From the comparison it is found that the obtained value within the allowable range.

- Figure (4-55); shows the various values of maximum displacement for case 1, case2 and case3, corresponding to the certain node. It can be observed from this figure, that the displacement obtained from case 3is less than the value obtained from case 1and case 2. The percentage difference between case1and case2is 4%, case 1and case3is 6%, case2andcase3is 5% for top tower.

case1and case2is -7%, case1andcase3 is -18%, case2 and case3is -12%, for main girder.

Table (4-17): Summary Result of Maximum Displacement

Type of loads	Top Tower x(m)	Main girder z (m)	Maximum Allowable Value AASHTO 2010 main girder	Maximum Allowable Value AASHTO 2010 Tower
LCB2(case 1)	0.033308	-0.009990	L/800 = 0.375 m	H/300= 0.22m
LCB2(case2)	0.033166	-0.009310		
LCB2(case 3)	0.031438	-0.008173		
LCB3	0.060	0.026		
Wind load	0.037	-0.0016		

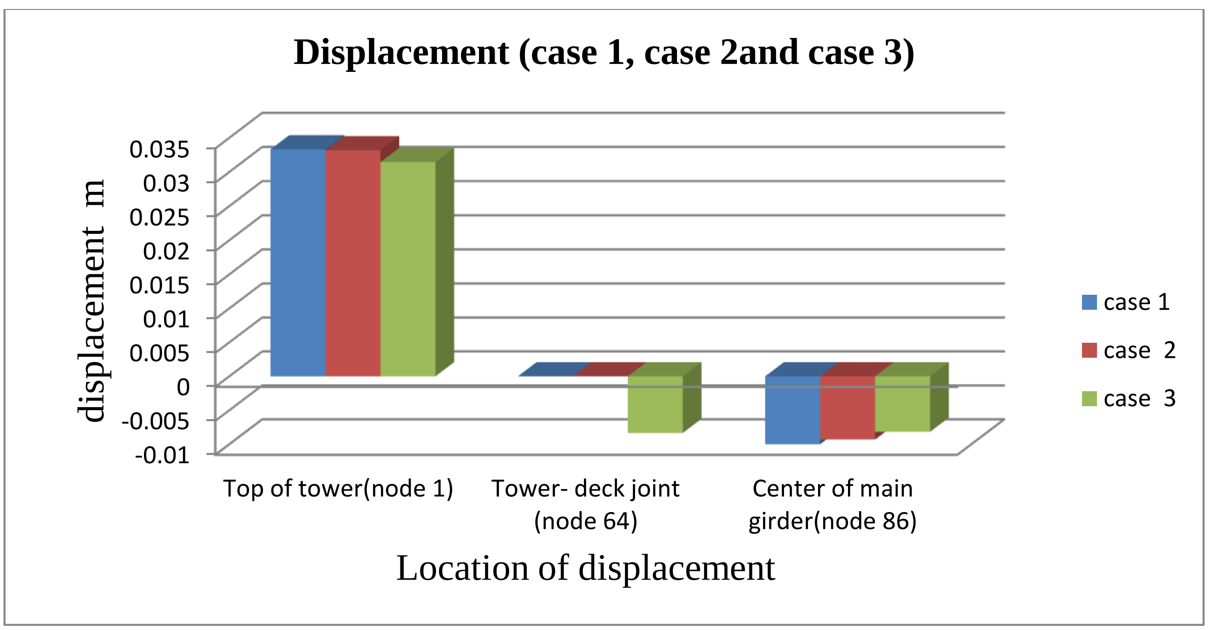


Figure (4-55): Displacement (Case 1, Case2 and Case 3)

4.4.2 Result of stresses

Table (4-18), Figures (4-56) and (4-57), illustrate the maximum value of cable stress under LCB2 for case 1, case 2 and case 3, and under LCB3. This value of cable stress is compared with the allowable range in AASHTO 2010.

Table (4-18): Summary Result of Maximum Allowable stresses

Type of loads	**Cable stress**	**Maximum Allowable Value AASHTO 2010**
LCB2(case1)	372 N/mm^2	837 N/mm^2
LCB2(case 2)	407 N/mm^2	
LCB2(case 3)	355 N/mm^2	
LCB3	408.5 N/mm^2	

- Figure (4-56), shows the value of stress (N/mm^2) in each cable obtained from case 1and case 2.The maximum value of cable stress is 372 N/mm^2 from Case 1and 407 N/mm^2 from case 2. Clearly, the value obtained from case 2 is greater than value from case 1. The percentage difference between these values is 8.5%.
- The various values of cable stress from case 1and case 3 are as shown in Figure (4-57). The maximum value from case 1 which is 372 N/mm^2 is more than maximum value obtained from case 3 which is 355 $N/mm^{2.}$. The percentage difference between these values is 5%.

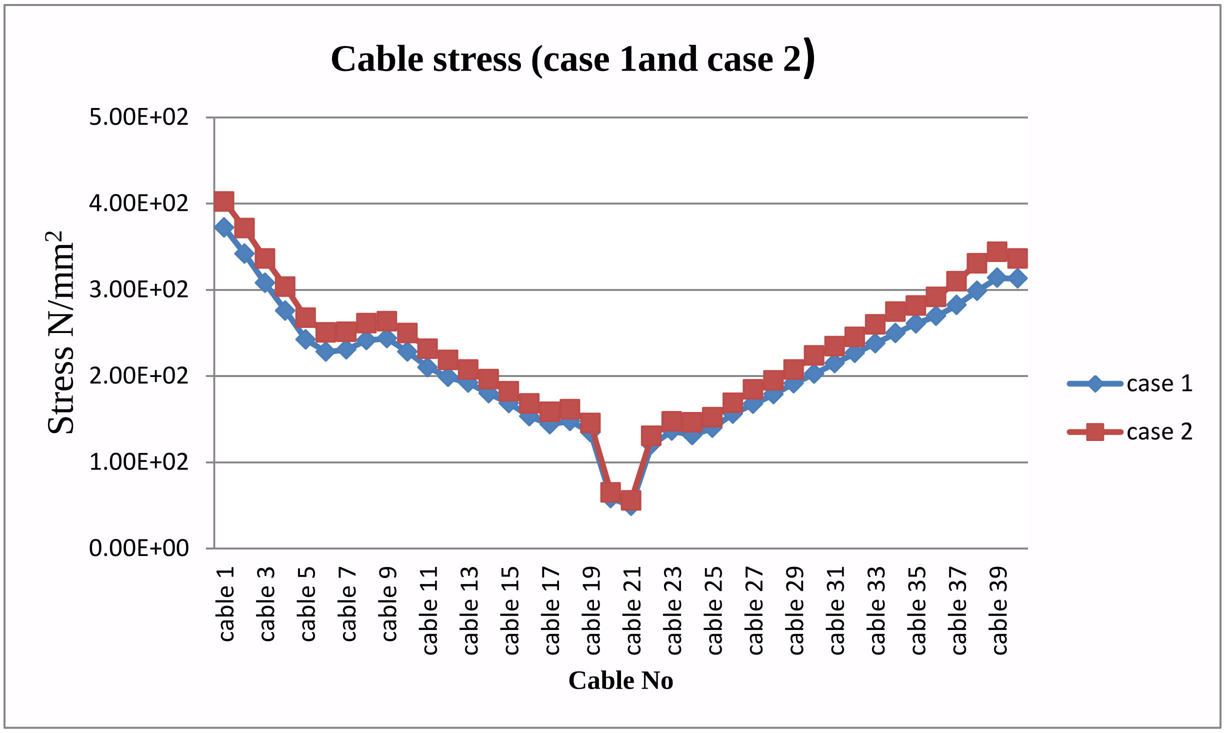


Figure (4-56): Cable stress (Case 1and Case2)

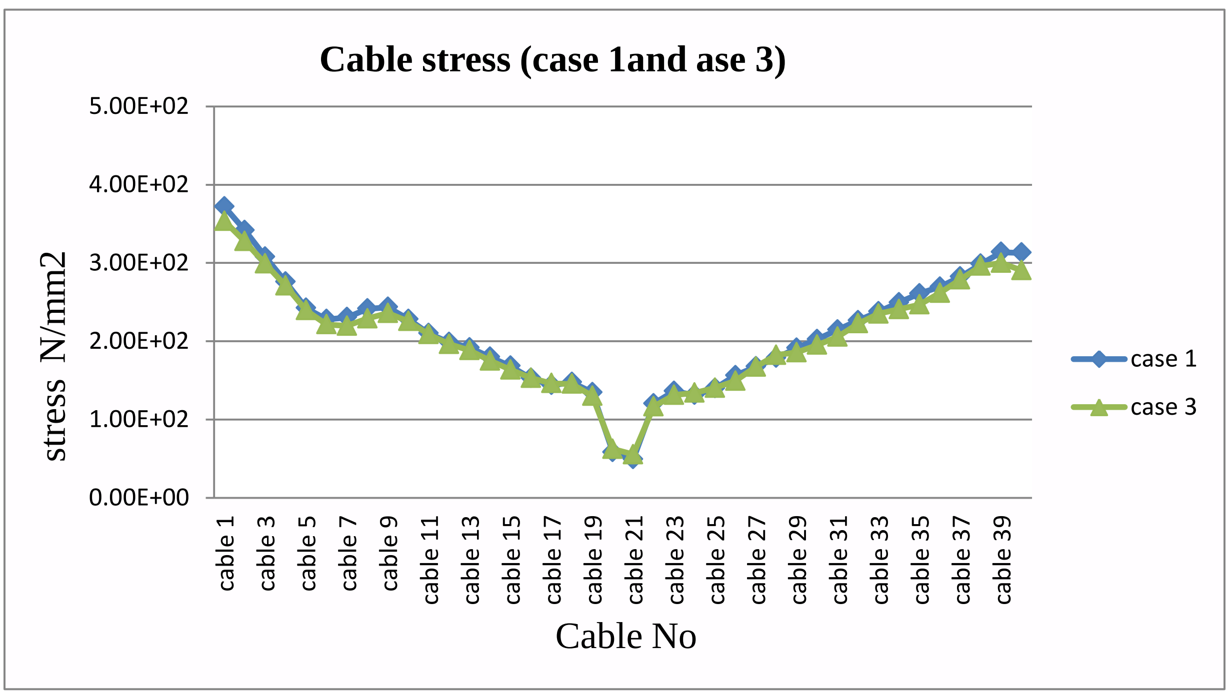


Figure (4-57): Cable stress (case 1and case3)

- **Comparison between Midas Civil and Preliminary Manual calculation**

Table (4-19), shows the comparison value of pre –tension force between the Midas Civil and preliminary Manual calculation.

Table (4-19): Comparison between Midas Civil and Manual calculation

Cable number	Calculated by Midas Civil	Manual calculation	%Different
1	6670.924	4889.355	26 %
2	6133.456	4868.339	21%
3	5531.531	4846.318	12%
4	4960.142	4822.958	3%
5	4366.902	4712.002	-7.3%
6	4111.070	4595.089	-11%
7	4153.700	4471.362	-7.1%
8	4344.881	4341.628	0.07%
9	4378.645	4202.978	4%
10	4106.029	4057.166	1.2%
11	3781.593	3902.953	-3%
12	3576.922	3739.852	-4.5%
13	3457.736	3567.656	-2%
14	3238.497	3386.175	-4.5%
15	3034.995	3195.675	-5%
16	2759.147	2997.217	-8%
17	2596.184	2792.932	-7%
18	2662.054	2587.247	3%
19	2430.959	2388.217	2%
20	1073.910	1237.5	-13%

CHAPTER FIVE

Construction Stage Results and Discussion

5.1 Introduction

This chapter contains the results obtained of Midas /civil program under backward construction stage analysis, and discussion of these results. The results are presented in tables graphs plotted for the parameters.

5.2 Result and discussion

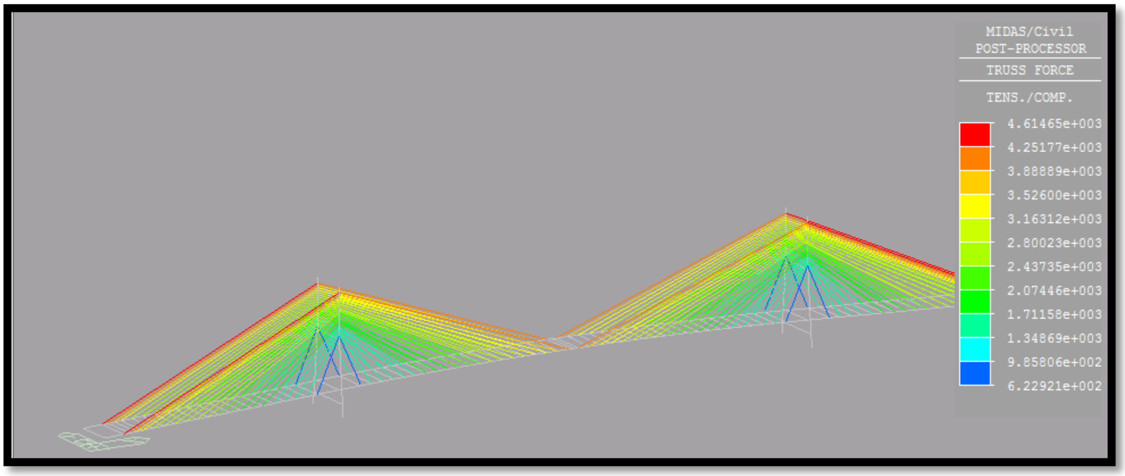
The result of the Construction Stage Analysis for each construction process requires detailed information on the existing partial structure to determine the actual structure state, investigate the deflection, and thus meet design guidelines. For TUTI BAHARI cable-stayed bridge, the criteria in construction stage were based on the design specification and construction scheme. Specifically, a zero allowable tension for the concrete tower was guaranteed in the construction stages analysis; each cable tension stress needed to be less than 0.45 times the cable design stress according to Podolny and B.scalzi,(1986). These limitations played an important role in the construction stages to ensure that the erected structure was in a safe state.

The results obtained of analysis model of cable –stayed bridge by using Midas civil program under backward construction stage analysis are shows in the following section.

5.2.1 Cable pre –tension force

The cable pre-tensioning forces, which are introduced during the construction of a cable–stayed bridge, were calculated by backward analysis from the final stage.

The axial force for each cable in each construction stage was evaluation sample of these calculation are shown in Figure (5-1)to Figure (5-12) .



Figure(5-1): Cable Pre tension force during construction (BCS0).

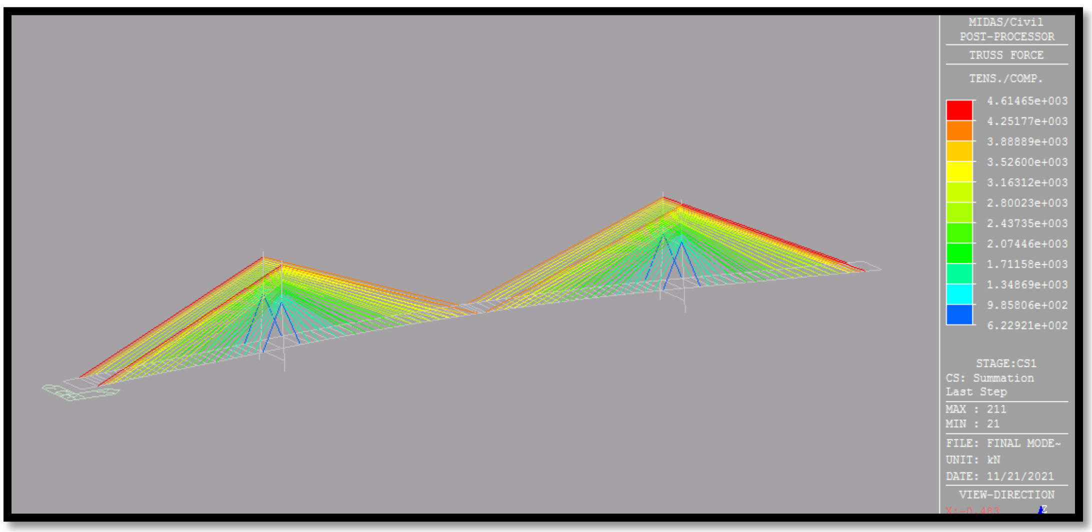


Figure (5-2): Cable Pre tension force during construction (BCS1).

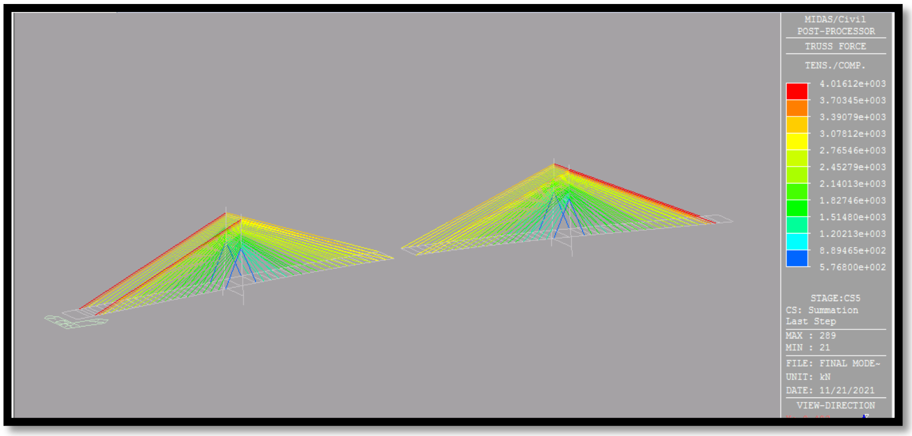


Figure (5-3): Cable Pre tension force during construction (BCS5).

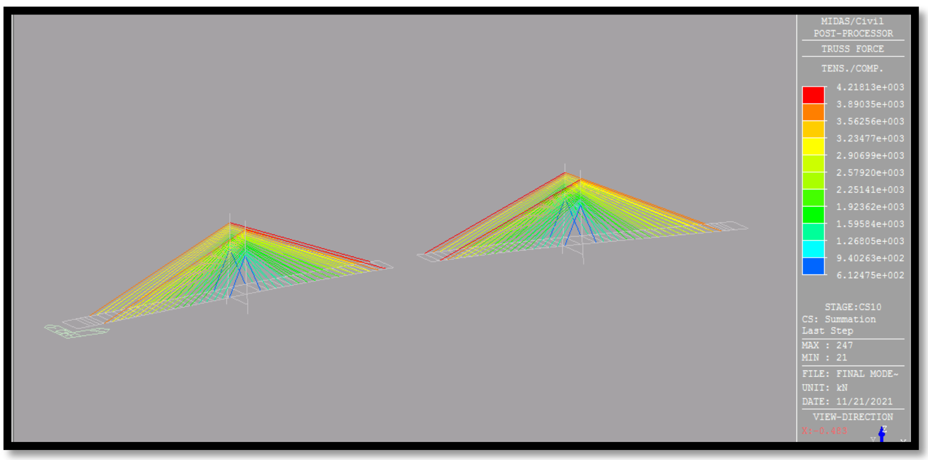


Figure (5-4): Cable Pre tension force during construction (BCS10).

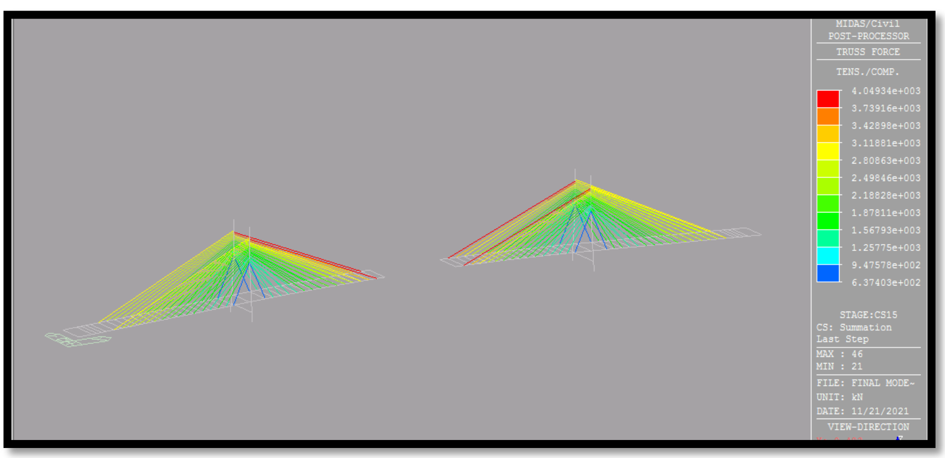


Figure (5-5): Cable Pre tension force during construction (BCS15).

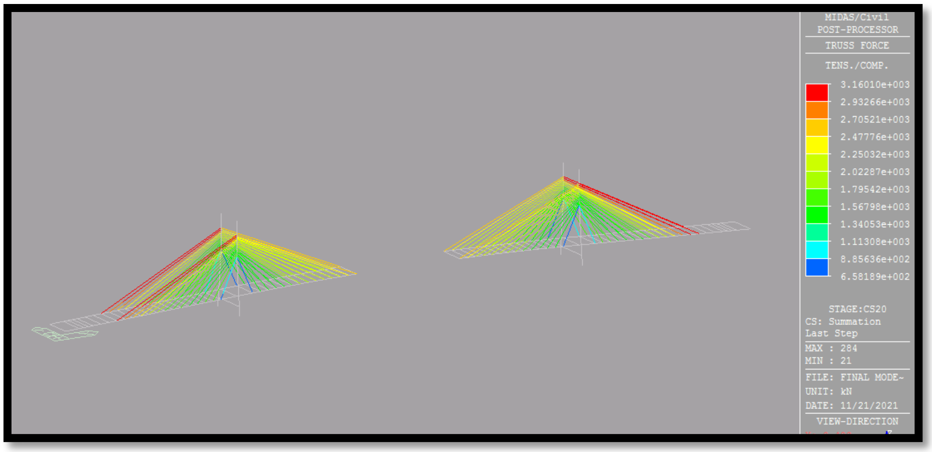


Figure (5-6): Cable Pre tension force during construction (BCS20).

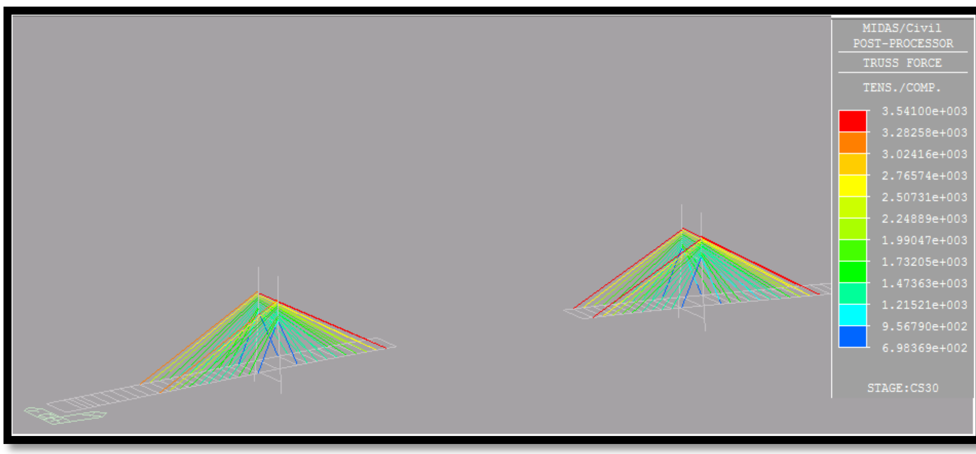


Figure (5- 7): Cable Pre tension force during construction (BCS30).

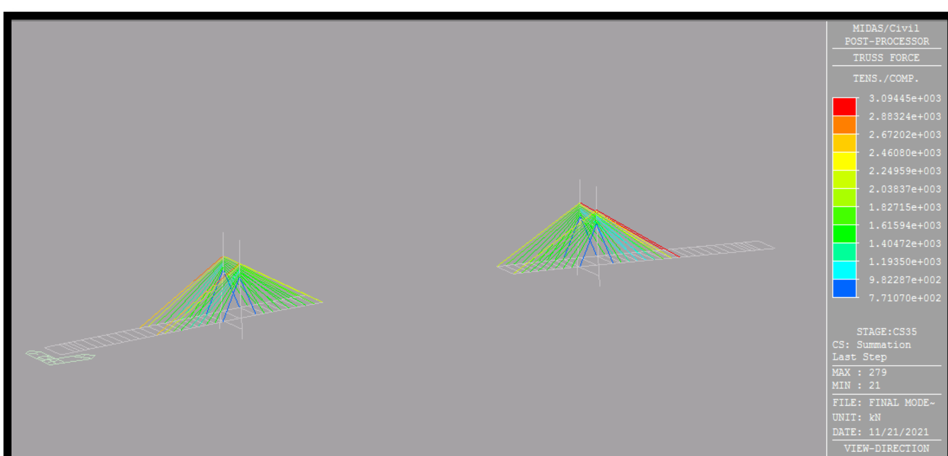


Figure (5-8): Cable Pre tension force during construction (BCS35).

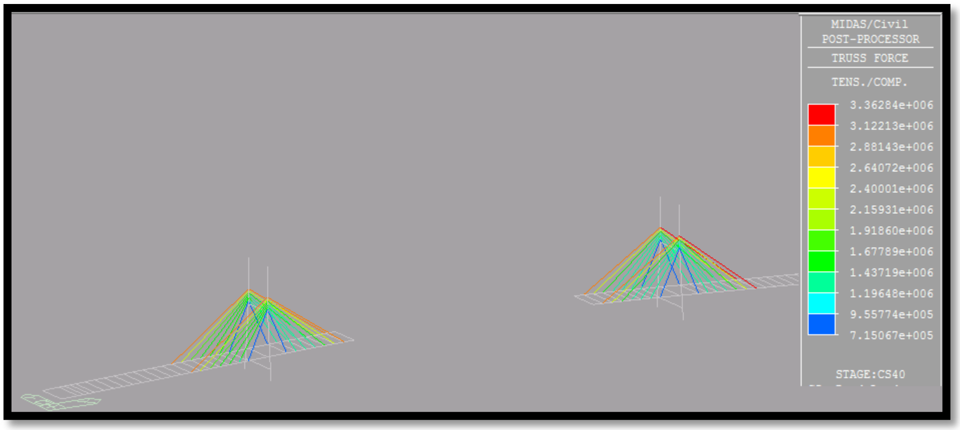


Figure (5-9): Cable Pre tension force during construction (BCS40).

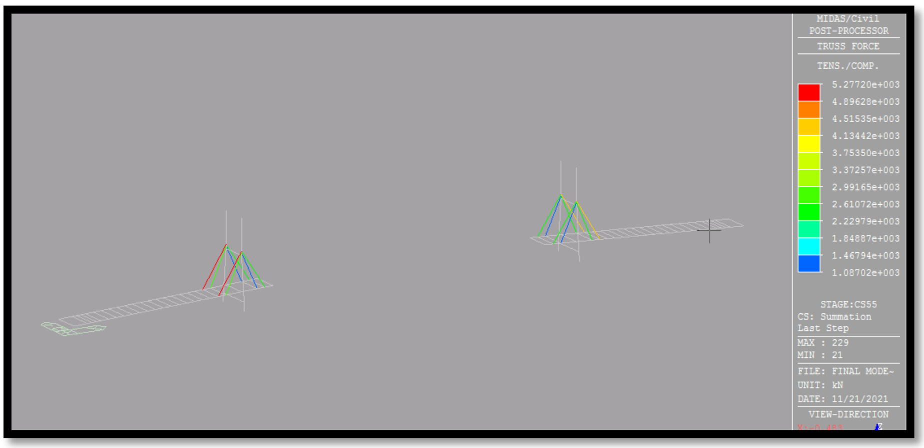


Figure (5-10): Cable Pre tension force during construction (BCS55).

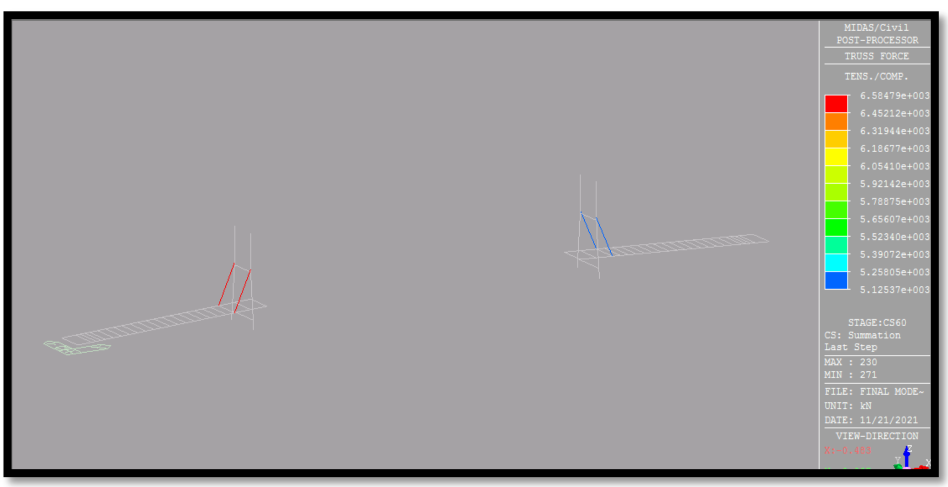


Figure (5-11): Cable Pre tension force during construction (BCS60).

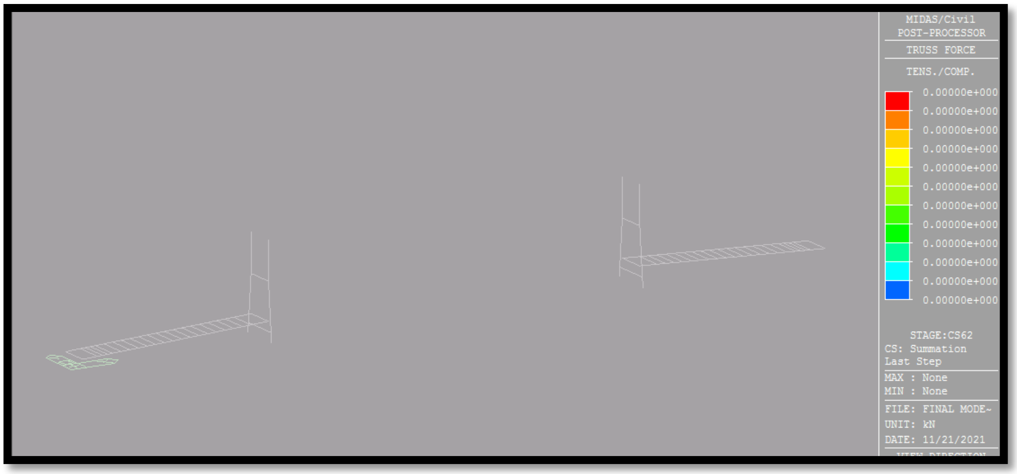


Figure (5-12): Cable Pre tension force during construction BCS62.

- Table (5-1) shows that the various values of pre –tension for cables 1 to 40 during construction stage BCS0, the maximum value of pre-tension reached 4610.40 kN at cable 1.

Table (5-1): Pre –tension force during BCS0

cable number	Forces at end I kN	Forces at end J kN
1	4523.257720	4610.404024
2	4078.093750	4163.155879
3	3607.737631	3690.715585
4	3192.394182	3273.287961
5	2765.010741	2843.611927
6	2566.084159	2642.392753
7	2544.273622	2618.289623
8	2623.510274	2695.233683
9	2624.541499	2693.972315
10	2442.121698	2509.259922
11	2237.626689	2302.472320
12	2108.773308	2171.326347
13	2031.325470	2091.585917
14	1898.111772	1956.079626
15	1774.192146	1829.867407
16	1609.422092	1662.804761
17	1510.759117	1561.849193
18	1579.747831	1628.545315
19	1497.734126	1544.239018
20	735.899212	780.111511
21	622.921435	665.233893
22	1353.929866	1397.619037
23	1487.820490	1532.909069
24	1394.028321	1440.539002
25	1489.076686	1537.032164
26	1665.928455	1715.351423
27	1787.221634	1838.134787
28	1903.503443	1955.929475
29	2034.845957	2088.807564
30	2148.490276	2204.010150
31	2272.525486	2329.626323
32	2399.541084	2458.245578
33	2525.585112	2585.915957
34	2666.526795	2728.506686
35	2817.858714	2881.510345
36	2960.653736	3025.999801
37	3167.840690	3234.903883
38	3454.548181	3523.351198
39	3766.979022	3837.544556
40	3911.834144	3984.184890

- Table (5-2) shows the variation of pre –tension for cable 20 and cable 21 during each construction stage, these cables are the first cables to be installed. The maximum tension value of cable 20 equals 6930.77 kN at stage (BCS 59). For cable 21 the maximum value is 2197.69kN at stage (BCS57).

Table (5-2): Cable Tension Force for cable (C20, C21) for Each Construction stage

Stage steps	**cable21(kN)**	**Stage steps**	**cable21(kN)**	**Stage steps**	**cable20(kN)**	**Stage steps**	**cable20(kN)**
BCS0	622.92140	BCS32	742.97620	BCS0	735.89920	BCS32	913.28230
BCS1	622.92140	BCS33	701.95240	BCS1	735.89920	BCS33	919.81470
BCS2	558.80710	BCS34	714.56890	BCS2	815.49380	BCS34	854.53390
BCS3	563.10970	BCS35	771.07050	BCS3	818.74460	BCS35	905.90980
BCS4	575.72510	BCS36	700.17130	BCS4	788.40980	BCS36	918.67530
BCS5	576.80010	BCS37	717.29340	BCS5	819.29120	BCS37	831.92300
BCS6	582.72300	BCS38	798.82950	BCS6	822.31650	BCS38	904.07330
BCS7	594.60960	BCS39	692.79180	BCS7	792.81960	BCS39	926.02860
BCS8	594.32610	BCS40	715.06730	BCS8	825.14170	BCS40	820.44190
BCS9	601.91670	BCS41	821.32710	BCS9	827.82730	BCS41	920.95870
BCS10	612.47550	BCS42	681.14770	BCS10	800.65230	BCS42	954.98140
BCS11	610.99670	BCS43	742.97620	BCS11	834.07430	BCS43	838.39940
BCS12	620.17280	BCS44	829.96970	BCS12	836.30010	BCS44	968.37970
BCS13	629.25430	BCS45	672.42600	BCS13	811.73810	BCS45	1015.37900
BCS14	627.00390	BCS46	705.20620	BCS14	845.90560	BCS46	902.54580
BCS15	637.40260	BCS47	814.12900	BCS15	847.57690	BCS47	1051.21700
BCS16	644.56670	BCS48	689.70950	BCS16	827.77230	BCS48	1105.14300
BCS17	642.36330	BCS49	726.38030	BCS17	862.19810	BCS49	1046.61700
BCS18	653.10450	BCS50	762.63050	BCS18	863.26260	BCS50	1186.12600
BCS19	659.06230	BCS51	784.37250	BCS19	846.03290	BCS51	1230.33300
BCS20	658.18910	BCS52	811.46520	BCS20	880.37260	BCS52	1419.48600
BCS21	667.43450	BCS53	680.52120	BCS21	880.95870	BCS53	1500.51100
BCS22	672.87430	BCS54	1155.67400	BCS22	863.67710	BCS54	1498.45800
BCS23	675.38200	BCS55	1087.01600	BCS23	897.66400	BCS55	2667.39000
BCS24	680.16980	BCS56	651.19100	BCS24	898.13370	BCS56	2635.46100
BCS25	685.83730	BCS57	2197.69100	BCS25	877.09190	BCS57	2482.45600
BCS26	694.82050	BCS58	1388.99900	BCS26	910.72550	BCS58	6790.29300
BCS27	690.75650	BCS59	406.12990	BCS27	911.78120	BCS59	6930.77000
BCS28	697.58430	BCS60	0.00000	BCS28	881.68840	BCS60	6528.21500
BCS29	717.26370	BCS61	0.00000	BCS29	916.20440	BCS61	0.00000
BCS30	698.36950	BCS62	0.00000	BCS30	919.05990	BCS62	0.00000
BCS31	707.53350			BCS31	873.85850		

5.2.2 Cable Stress

For each construction stage, cables stresses, main girder and tower stresses were evaluated. For construction stage BCS0, cables stresses, main girder and tower stresses are shown in Figures (5-13) and (5-14), respectively.. It was noticed that the cable stress varies during construction stages as can be seen from Table (5-3). The maximum value of cable stresses is 260 N/mm^{2}at cables (1).The maximum value of cable stress occurs at the cable with largest value of pre - tension force for each construction stage.

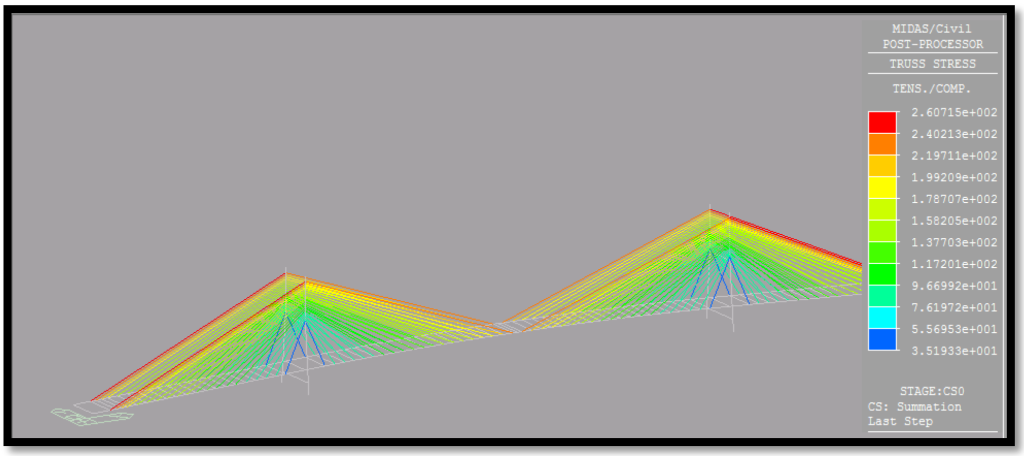


Figure (5- 13): Cable stress during construction (BCS0).

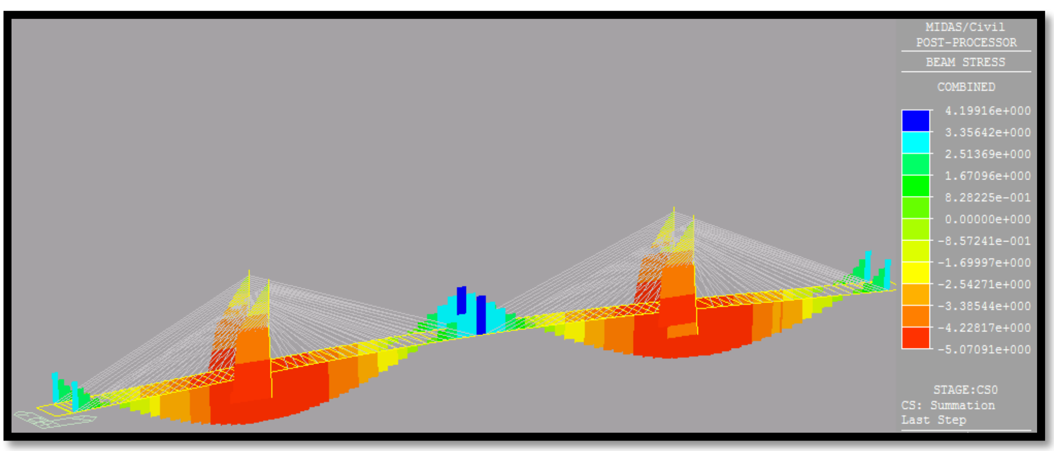


Figure (5-14) :Towers and main girder stress during construction (BCS0).

- From Figure (5-15) to Figure (5-25), a sample of the stress result in cables during different construction stages is presented. It is noticeable that the stress changes during these different stages, also, the number of cables reduced according to the stage of construction. The maximum value of the cable stress occurs in the first construction stage BCS60 equal to 372 N/mm^2 at the first cable in construction.

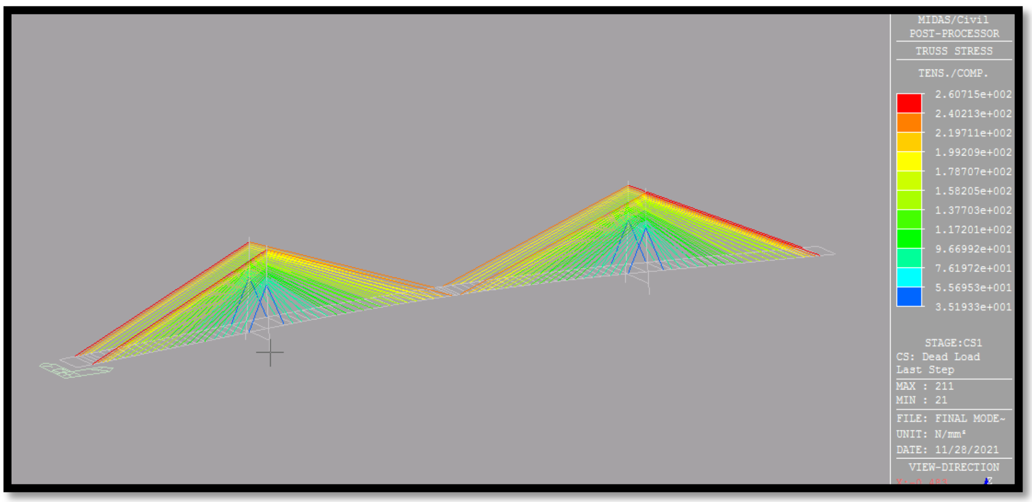


Figure (5-15): Cable stress under (BCS1).

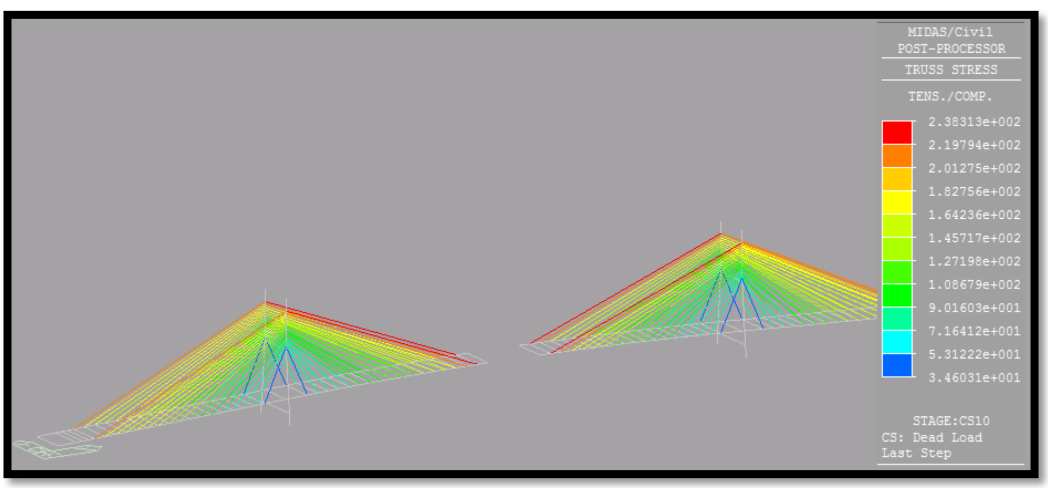


Figure (5- 16): Cable stress during construction (BCS10).

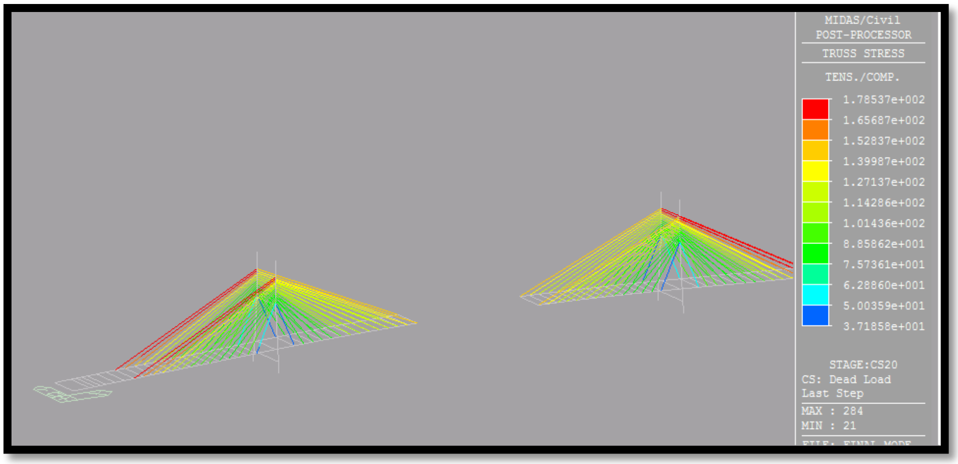


Figure (5- 17): Cable stress during construction (BCS20).

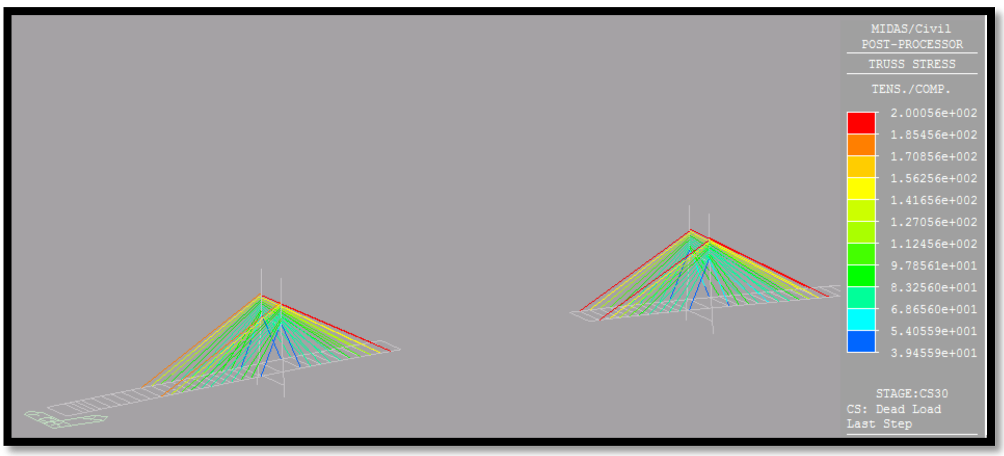


Figure (5-18): Cable stress under (BCS30).

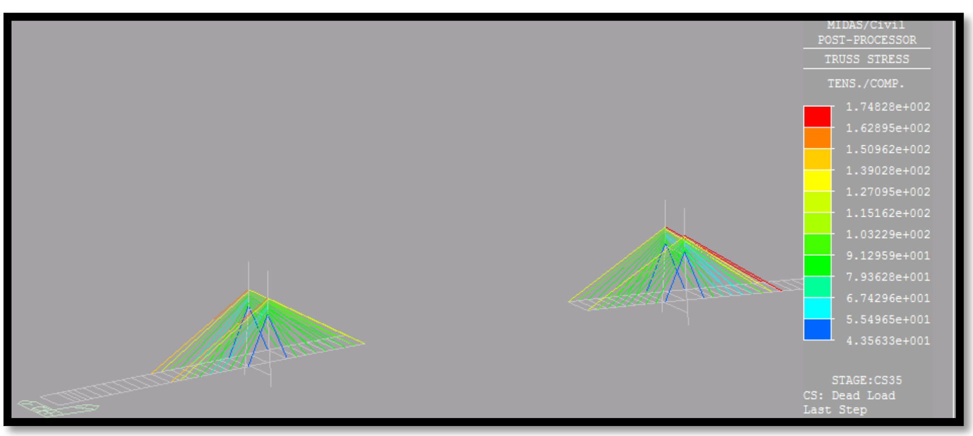


Figure (5-19): Cable stress during construction (BCS35).

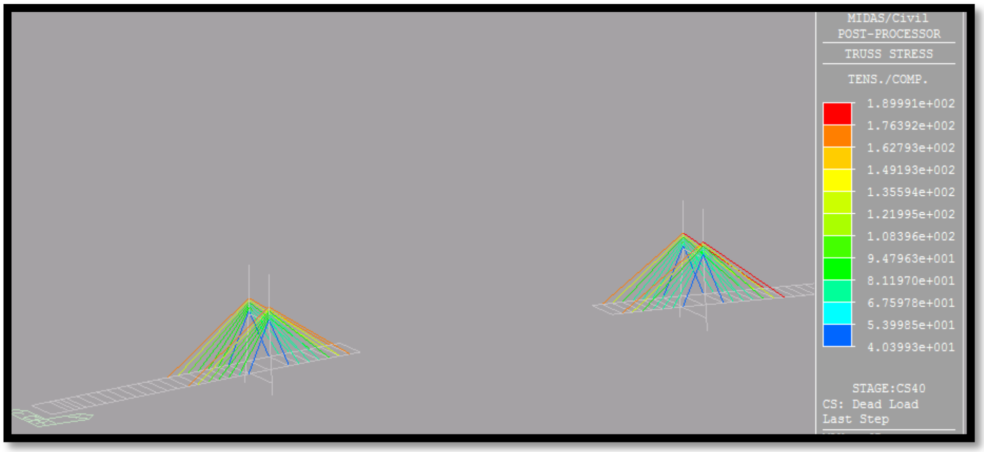


Figure (5- 20): Cable stress during construction (BCS40).

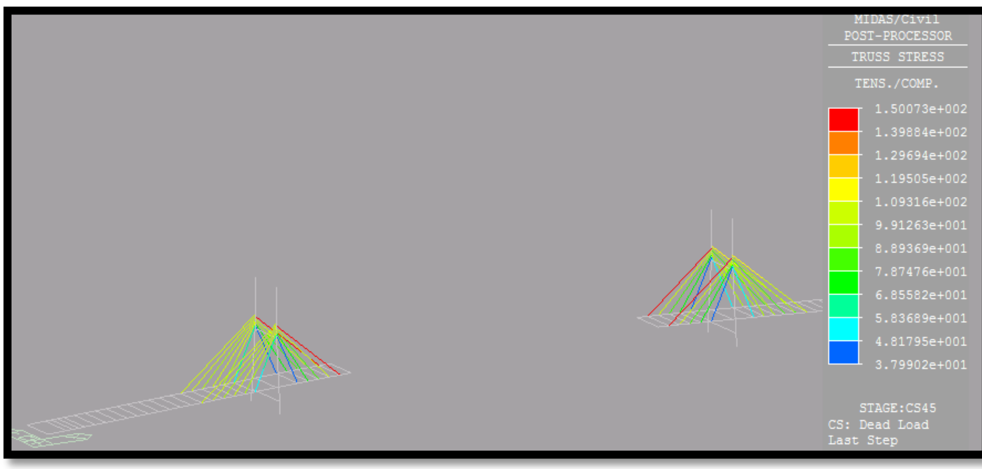


Figure (5-21): Cable stress under (BCS45).

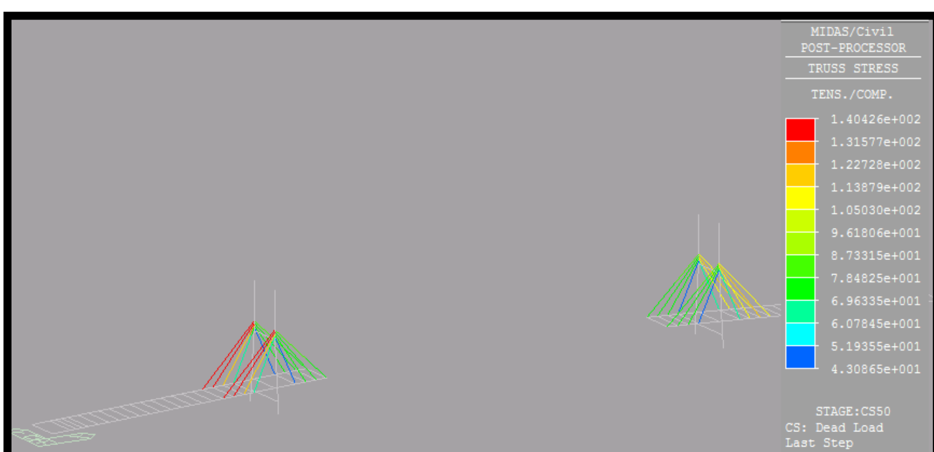


Figure (5-22): Cable stress under (BCS50).

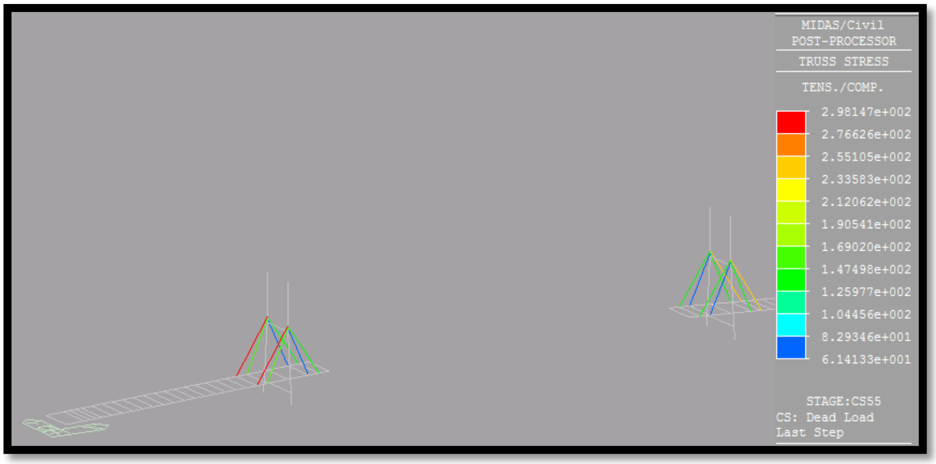


Figure (5- 23): Cable stress during construction (BCS55).

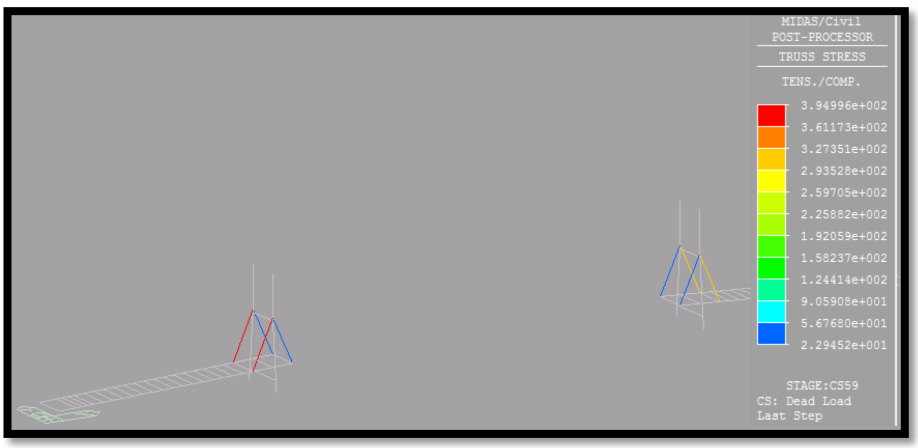


Figure (5-24): Cable stress under (BCS59).

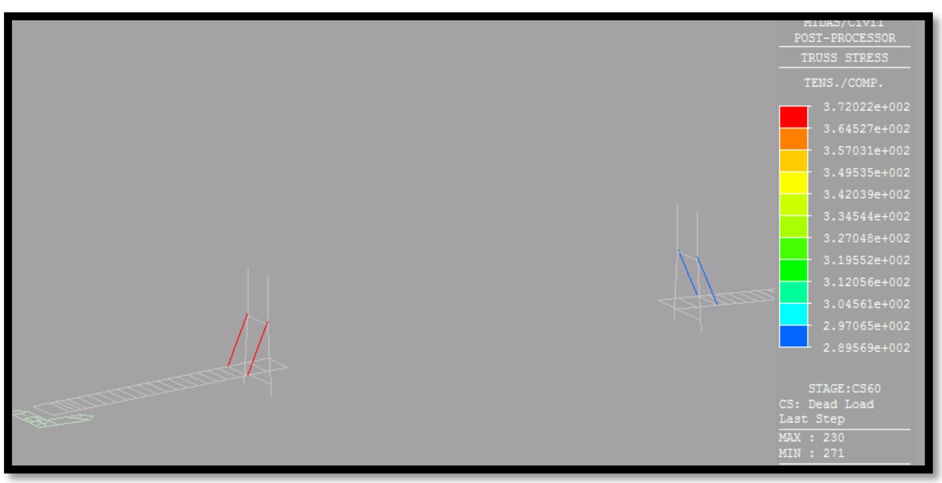


Figure (5- 25): Cable stress during construction (BCS60).

Table (5-3): Cable stress during construction stage BCS0

Cable number	**Stress at end I N/mm^2**	**Stress at end J N/mm^2**
1	2.556e+002	2.605e+002
2	2.304e+002	2.352e+002
3	2.038e+002	2.085e+002
4	1.804e+002	1.849e+002
5	1.562e+002	1.607e+002
6	1.450e+002	1.493e+002
7	1.437e+002	1.479e+002
8	1.482e+002	1.523e+002
9	1.483e+002	1.522e+002
10	1.380e+002	1.418e+002
11	1.264e+002	1.301e+002
12	1.191e+002	1.227e+002
13	1.148e+002	1.182e+002
14	1.072e+002	1.105e+002
15	1.002e+002	1.034e+002
16	9.093e+001	9.394e+001
17	8.535e+001	8.824e+001
18	8.925e+001	9.201e+001
19	8.462e+001	8.725e+001
20	4.158e+001	4.407e+001
21	3.519e+001	3.758e+001
22	7.649e+001	7.896e+001
23	8.406e+001	8.661e+001
24	7.876e+001	8.139e+001
25	8.413e+001	8.684e+001
26	9.412e+001	9.691e+001
27	1.010e+002	1.038e+002
28	1.075e+002	1.105e+002
29	1.150e+002	1.180e+002
30	1.214e+002	1.245e+002
31	1.284e+002	1.316e+002
32	1.356e+002	1.389e+002
33	1.427e+002	1.461e+002
34	1.507e+002	1.542e+002
35	1.592e+002	1.628e+002
36	1.673e+002	1.710e+002
37	1.790e+002	1.828e+002
38	1.952e+002	1.991e+002
39	2.128e+002	2.168e+002
40	2.210e+002	2.251e+002

5.2.3Deformed shape

The changes of deformed shapes for each construction stage are presented by construction stage BCS0 analysis as shown in Figure (5-26). The displacements shape of the cable for construction stage BCS0 as seen show in Table (5-4) and Figure (5-27).

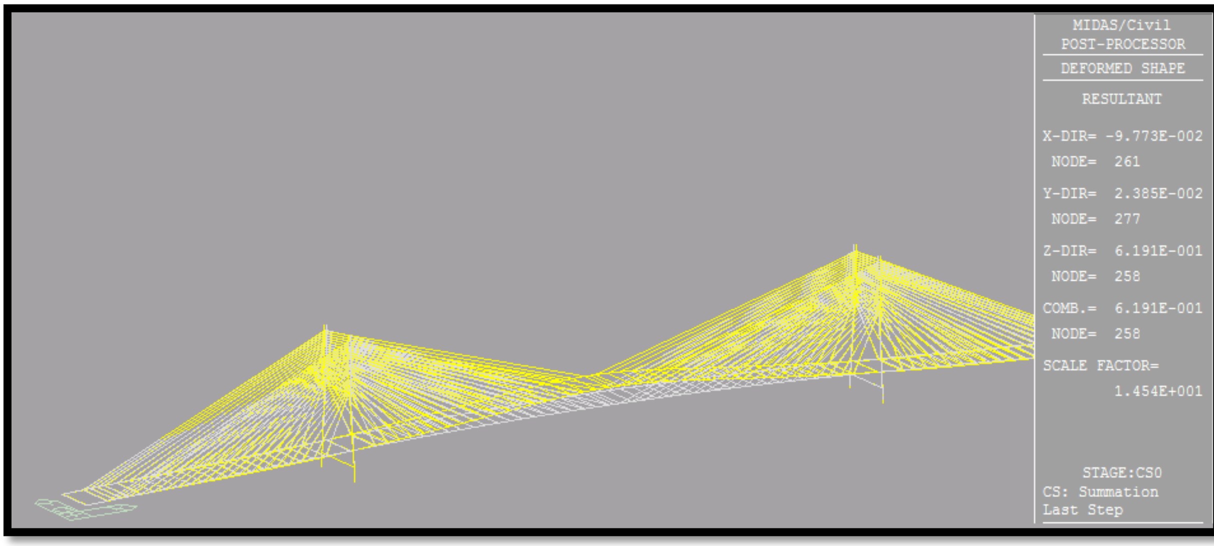


Figure (5- 26): Deformed shape during construction stage(BCS0).

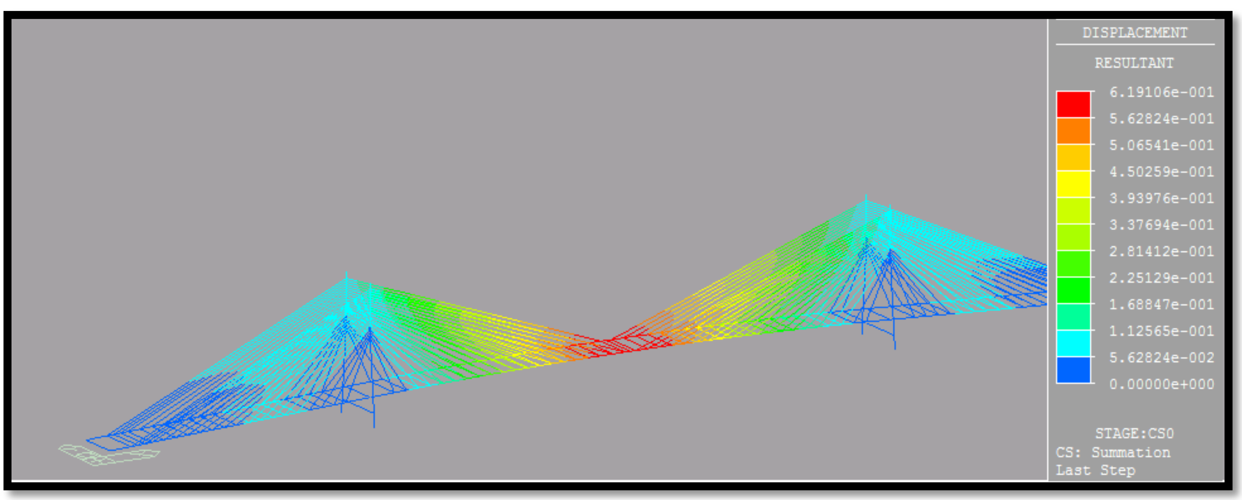


Figure (5- 27): Displacement of cable during construction (BCS0).

Table (5- 4): Displacement during construction stage (BCS0).

Node	Dx m	Dy m	Dz m	Rx rad	Ry rad	Rz rad
1	-0.097441	0.011924	-0.011629	-0.000262	-0.001928	0.000011
2	-0.091079	0.011060	-0.011622	-0.000262	-0.001928	0.000011
3	-0.088574	0.010720	-0.011605	-0.000262	-0.001926	0.000011
4	-0.086072	0.010380	-0.011576	-0.000262	-0.001922	0.000011
5	-0.083578	0.010040	-0.011535	-0.000262	-0.001914	0.000011
6	-0.081096	0.009699	-0.011483	-0.000262	-0.001903	0.000011
7	-0.078631	0.009359	-0.011421	-0.000262	-0.001889	0.000011
8	-0.076184	0.009019	-0.011349	-0.000262	-0.001875	0.000011
9	-0.073757	0.008679	-0.011267	-0.000262	-0.001860	0.000011
10	-0.071347	0.008338	-0.011176	-0.000262	-0.001847	0.000011
11	-0.068955	0.007998	-0.011074	-0.000262	-0.001834	0.000011
12	-0.066579	0.007658	-0.010963	-0.000262	-0.001821	0.000011
13	-0.064221	0.007317	-0.010842	-0.000262	-0.001808	0.000011
14	-0.061879	0.006977	-0.010712	-0.000262	-0.001794	0.000011
15	-0.059556	0.006637	-0.010573	-0.000262	-0.001780	0.000011
16	-0.057251	0.006296	-0.010424	-0.000262	-0.001766	0.000011
17	-0.054964	0.005955	-0.010266	-0.000262	-0.001751	0.000011
18	-0.052698	0.005615	-0.010098	-0.000262	-0.001735	0.000011
19	-0.050453	0.005274	-0.009922	-0.000262	-0.001719	0.000011
20	-0.048230	0.004933	-0.009736	-0.000262	-0.001701	0.000011
21	-0.046031	0.004592	-0.009540	-0.000262	-0.001682	0.000011
22	0.097627	0.008503	-0.010764	-0.000291	0.001928	-0.000014
23	0.091264	0.007541	-0.010758	-0.000291	0.001928	-0.000014
24	0.088757	0.007162	-0.010741	-0.000291	0.001927	-0.000014
25	0.086254	0.006783	-0.010711	-0.000291	0.001923	-0.000014
26	0.083759	0.006404	-0.010670	-0.000291	0.001915	-0.000014
27	0.081276	0.006025	-0.010618	-0.000291	0.001904	-0.000014
28	0.078810	0.005647	-0.010556	-0.000291	0.001890	-0.000014
29	0.076362	0.005268	-0.010484	-0.000291	0.001876	-0.000014
30	0.073933	0.004889	-0.010403	-0.000291	0.001861	-0.000014
31	0.071522	0.004510	-0.010311	-0.000291	0.001848	-0.000014
32	0.069128	0.004131	-0.010209	-0.000291	0.001835	-0.000014
33	0.066750	0.003752	-0.010098	-0.000292	0.001823	-0.000014
34	0.064389	0.003373	-0.009977	-0.000292	0.001810	-0.000014
35	0.062046	0.002994	-0.009847	-0.000292	0.001796	-0.000014
36	0.059719	0.002615	-0.009708	-0.000292	0.001783	-0.000014
37	0.057411	0.002236	-0.009559	-0.000292	0.001769	-0.000014
38	0.055121	0.001857	-0.009400	-0.000292	0.001754	-0.000014
39	0.052851	0.001477	-0.009233	-0.000292	0.001738	-0.000014
40	0.050601	0.001098	-0.009057	-0.000292	0.001722	-0.000014

5.3 Backward construction stage result discussion

5.3.1 Cable pre –tension

The various values of cable pre-tension force during construction stage BCS0 are as shown in Figure (5-28). The maximum value of cable pre- tension force during construction stage BCS0 occurs at the first cable and is equal to 4610.40 kN .

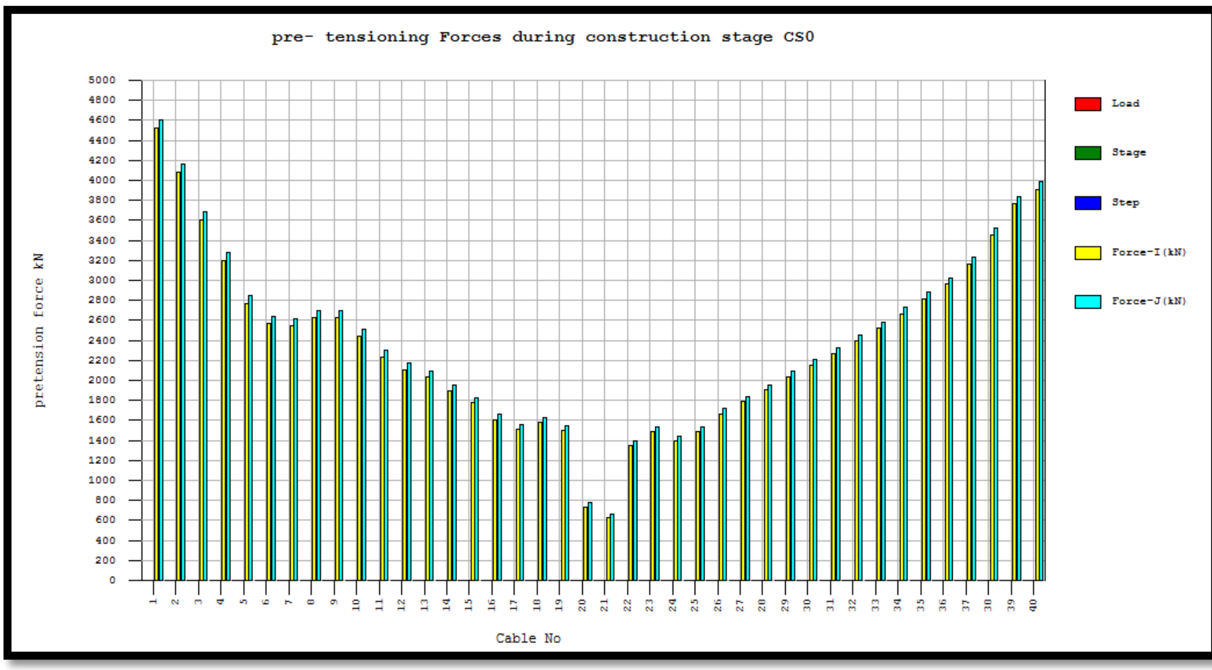


Figure (5-28): Variation of cable pre-tension forcee for each (BCS0).

- Figure (5-29), presents the different values of pre-tension force for cable 20 and cable 21 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 20 is 6930.77 kN at BCS59. Also, the maximum value for cable 21equals 2197.69 kN at BCS57.

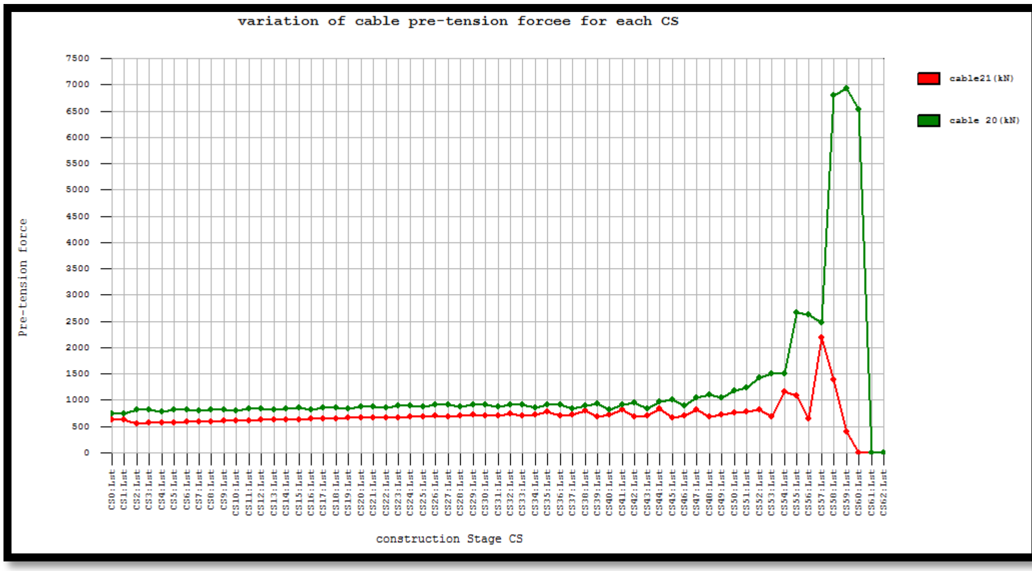


Figure (5-29): variation of cable pre-tension from cable 20and cable 21 for each BCS.

- The variation value of pre-tension force for cable 19 and cable 22 during different construction stage BCS is as shown in Figure (5-30).It can see that the maximum value for cable 19 equals 5257 kN occuring at BCS 55. Also, the maximum value for cable 22 equals 2519 kN occuring at BCS54.

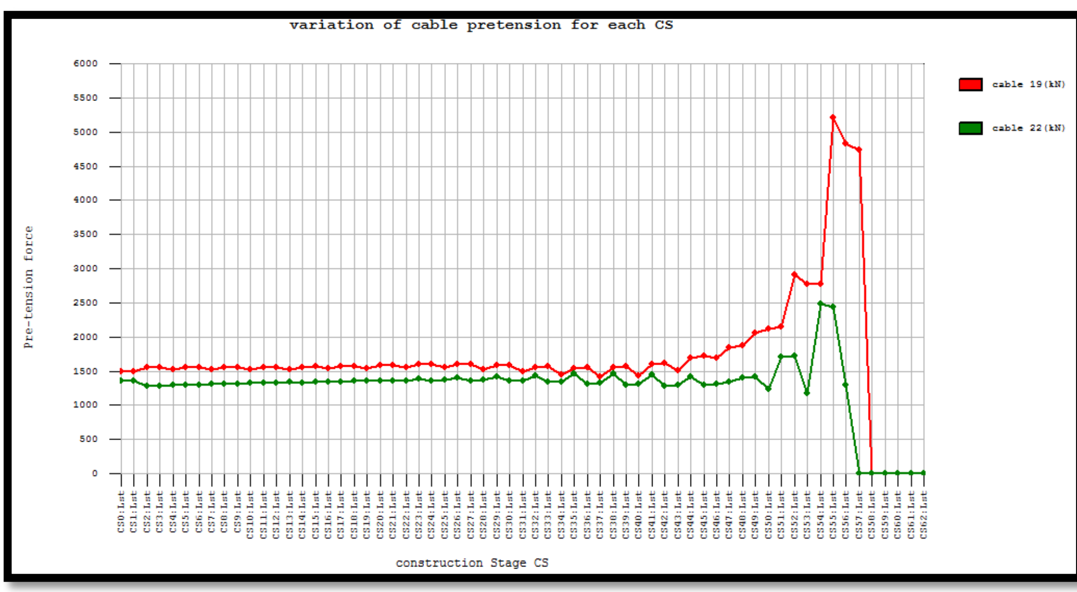


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Figure (5-30): variation of cable pretension from cable 19 and cable 22 for each BCS

- Figure (5-31), presents the different values of pre-tension force for cable 18 and cable 23 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 18 is 4014.84 kN at BCS52. Also, the maximum value for cable 23equals 2485.62 kN at BCS52.

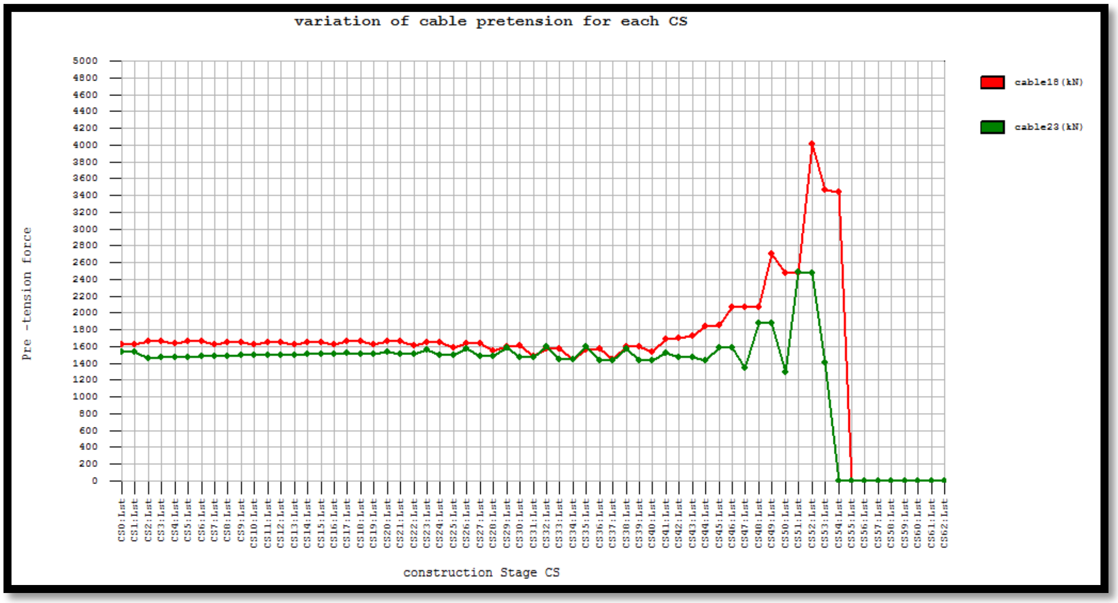

Figure (5-31): variation of cable pretension from cable 18 and cable 23 for each BCS

- It can be noted that from figure (5-29) , (5-30)and (5-31) that the values of the pre-tension forces obtained for the different construction stages are far apart, and this is clearly shown in the cables 20and 21.But in the remaining figures noted that the values of the pre-tension forces in the cables converge.
- Figure (5-32), presents the different values of pre-tension force for cable 16 and cable 25 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 16 is 2809.99 kN at BCS47. Also, the maximum value for cable 25equals 2608.34 kN at BCS46.

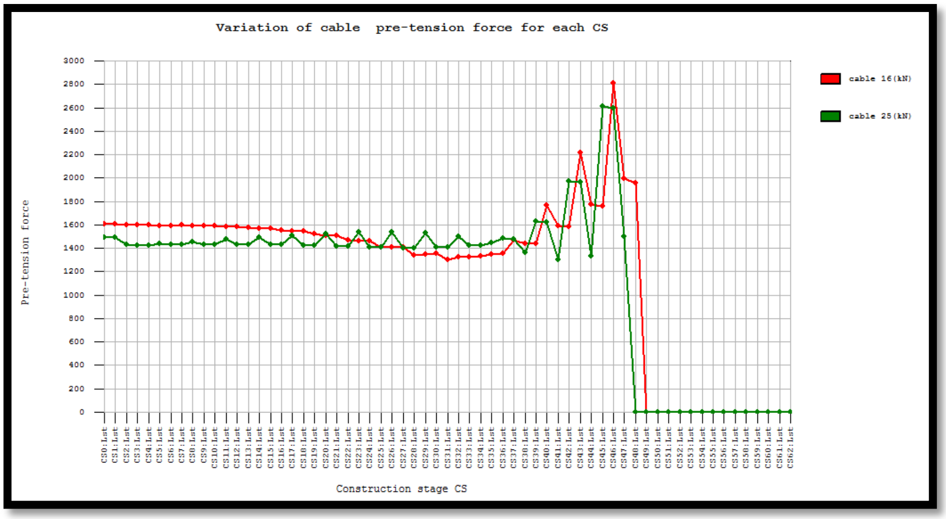

Figure (5-32): variation of cable pretension from cable 16 and cable 25 for each BCS

- Figure (5-33), presents the different values of pre-tension force for cable 15 and cable 26 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 15 is 2828.57kN at BCS43. Also, the maximum value for cable 26equals 2799.55 kN at BCS41.

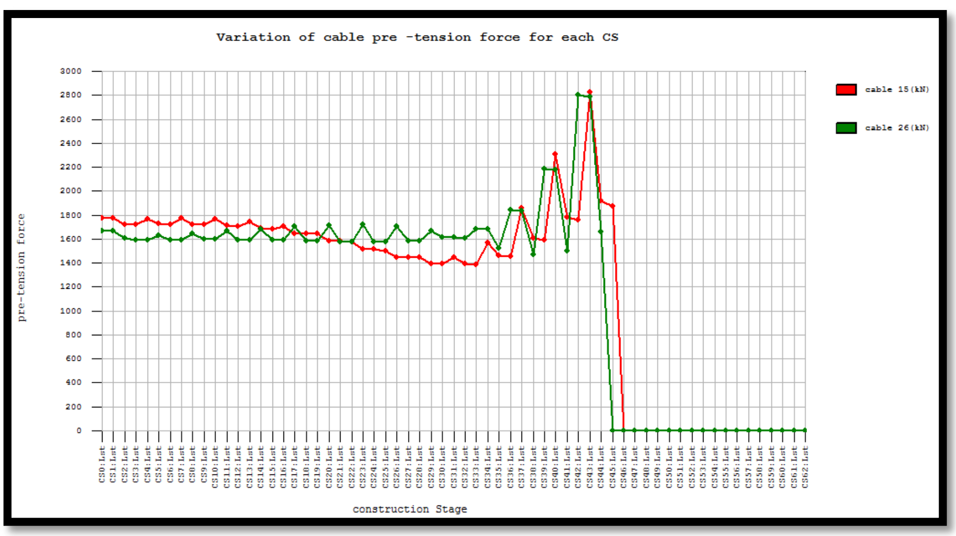

Figure (5-33): variation of cable pretension from cable 15 and cable 26 for each BCS

- Figure (5-34), presents the different values of pre-tension force for cable 14 and cable 27 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 14 is 3034.53 kN at BCS40. Also, the maximum value for cable 27equals 2947.35kN at BCS39.
- Figure (5-35), presents the different values of pre-tension force for cable 13 and cable 28 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 13 is 3365.99 kN at BCS37. Also, the maximum value for cable 28equals 3092.28 kNat BCS36.

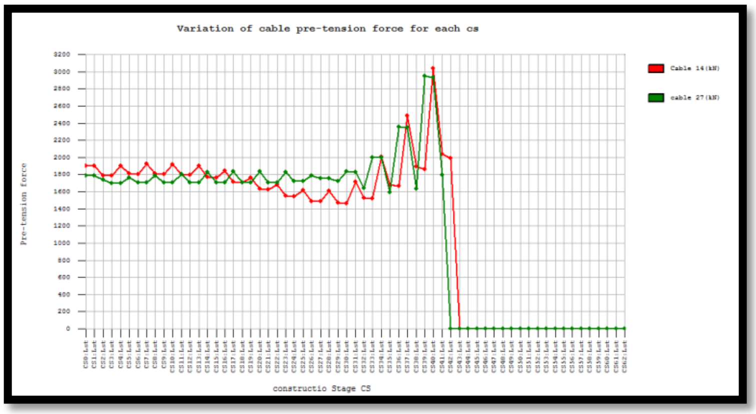

Figure (5-34): variation of cable pretension from cable 14 and cable 27 for each BCS.

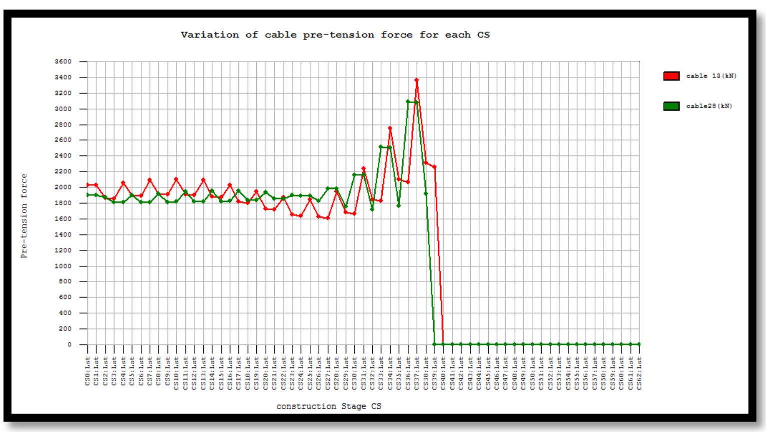

Figure (5-35): variation of cable pretension from cable 13 and cable 28 for each BCS.

- Figure (5-36), presents the different values of pre-tension force for cable 12 and cable 29 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 12 is 3365.99 kN at BCS34. Also, the maximum value for cable 29equals 3092.28 kNat BCS33

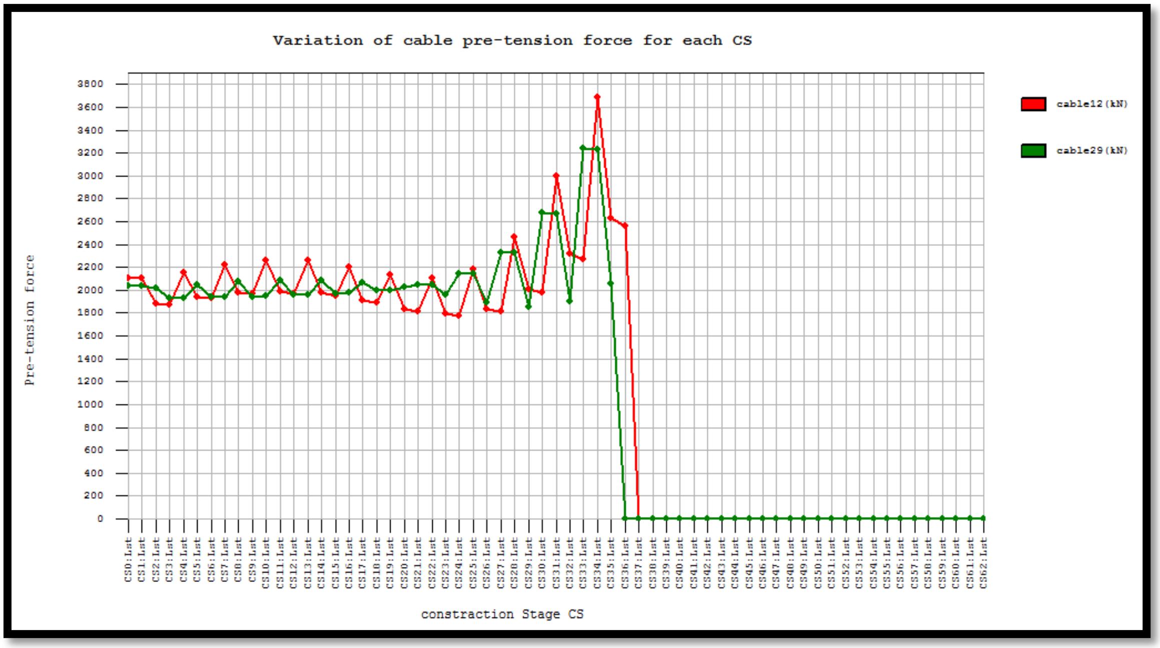

Figure (5-36): variation of cable pretension from cable 12 and cable 29 for each BCS

- Figure (5-37), presents the different values of pre-tension force for cable 11 and cable 30 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 11 is 3946.68 kN at BCS31. Also, the maximum value for cable 30 equals 3378.33kN at BCS 30
- Figure (5-38), presents the different values of pre-tension force for cable 10 and cable 31 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 10 is 3365.99 kN at BCS 28. Also, the maximum value for cable 31equals 3092.28 kNat BCS 27

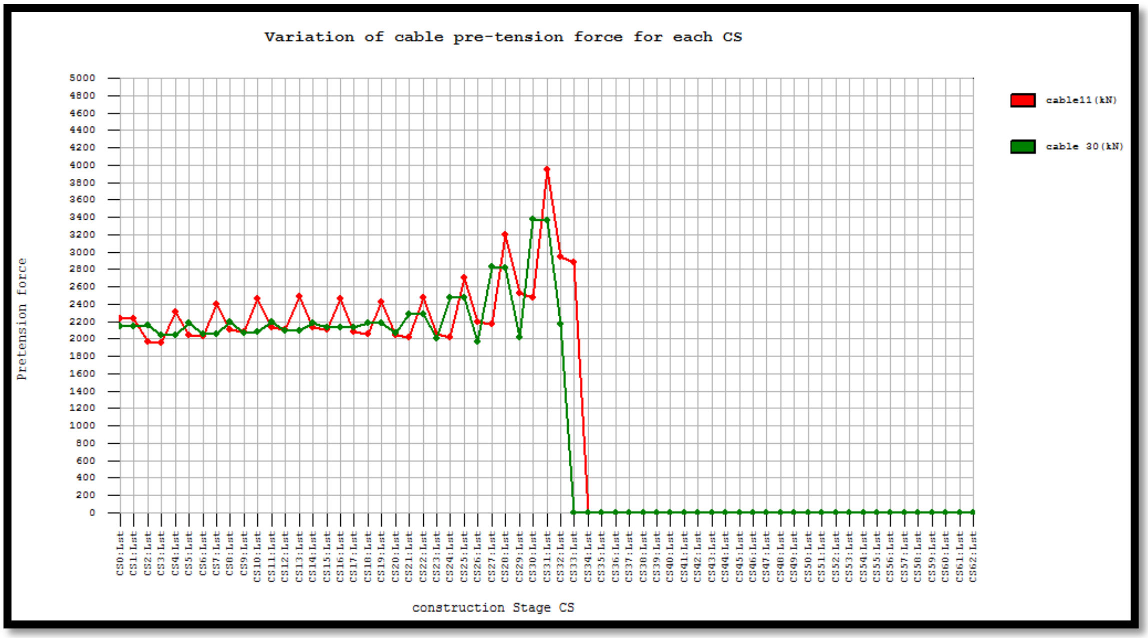

Figure (5-37): variation of cable pretension from cable 11 and cable 30 for each BCS.

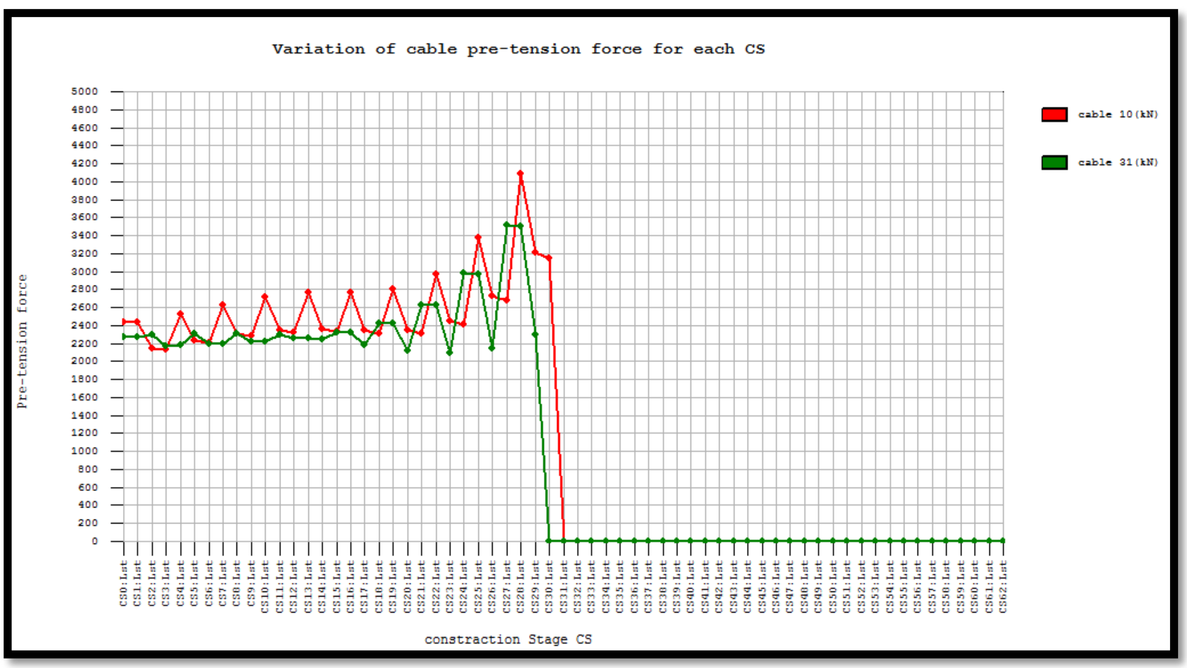

Figure (5-38): variation of cable pretension from cable 10 and cable 31 for each CS

- Figure (5-39), presents the different values of pre-tension force for cable 9and cable 32 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 9 is 4062.08 kN at BCS 25. Also, the maximum value for cable 32equals 3643.18Kn at BCS 24

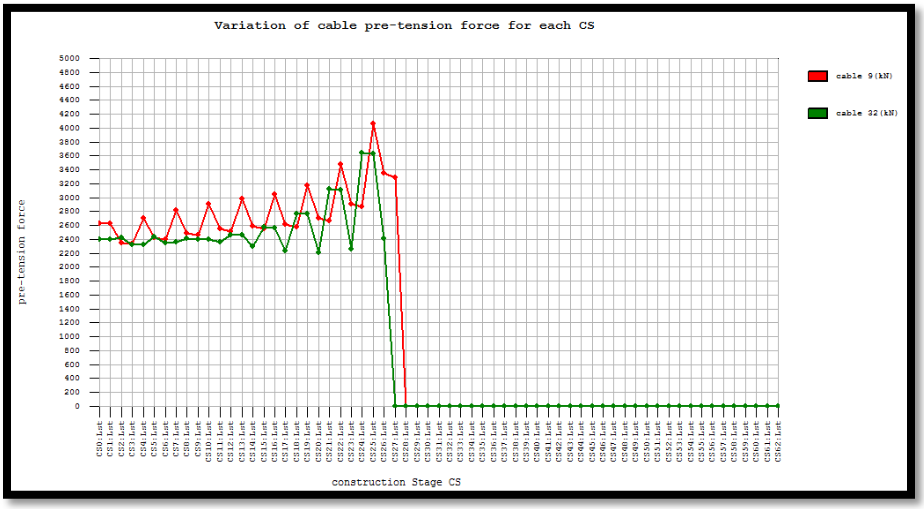

Figure (5-39): variation of cable pretension from cable 9 and cable 32 for each BCS

- Figure (5-40), presents the different values of pre-tension force for cable 8and cable 33 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 8 is 3807.77 kN at BCS 22. Also, the maximum value for cable 33equals 3761.47KN at BCS 21.
- Figure (5-41), presents the different values of pre-tension force for cable 7and cable 34 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 7 is 3487.94 kN at BCS 19. Also, the maximum value for cable 34equals 3877.36.18KN at BCS 18.

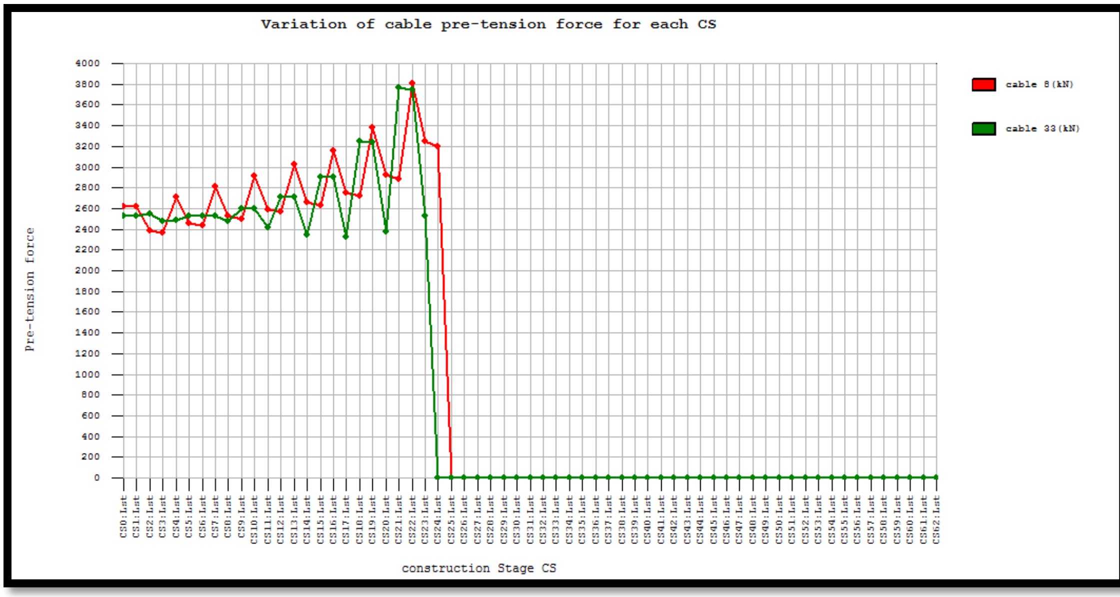

Figure (5-40): variation of cable pretension from cable 8 and cable 33 for each BCS

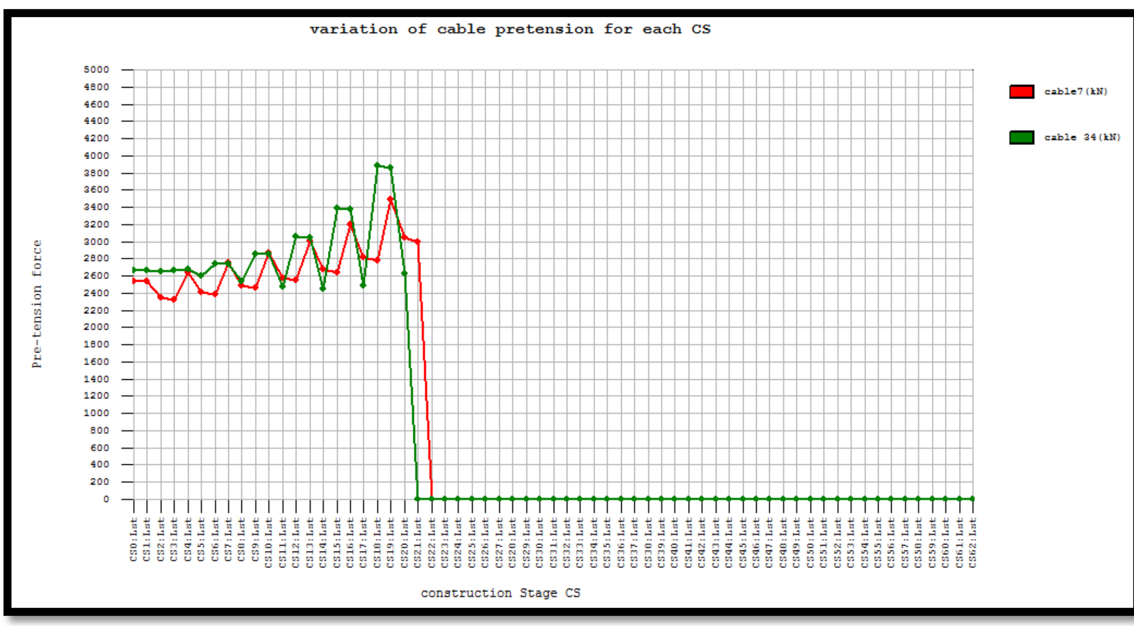

Figure (5-41): variation of cable pretension from cable 7 and cable 34 for each BCS

Figure (5-42), presents the different values of pre-tension force for cable 6 and cable 35 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 6 is 3392.80 kN at BCS 16. Also, the maximum value for cable 35equals 3984.54.18KN at BCS 15

Figure (5-42): variation of cable pretension from cable 6 and cable 35 for each BCS

- Figure (5-43), presents the different values of pre-tension force for cable 5and cable 36 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 5 is 3385.22 kN at BCS 13. Also, the maximum value for cable 36equals 4072.22kN at BCS 12
- Figure (5-44), presents the different values of pre-tension force for cable 4and cable 37 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 4 is 3714.66 kN at BCS 10. Also, the maximum value for cable 37equals 4173.58 kN at BCS 9.

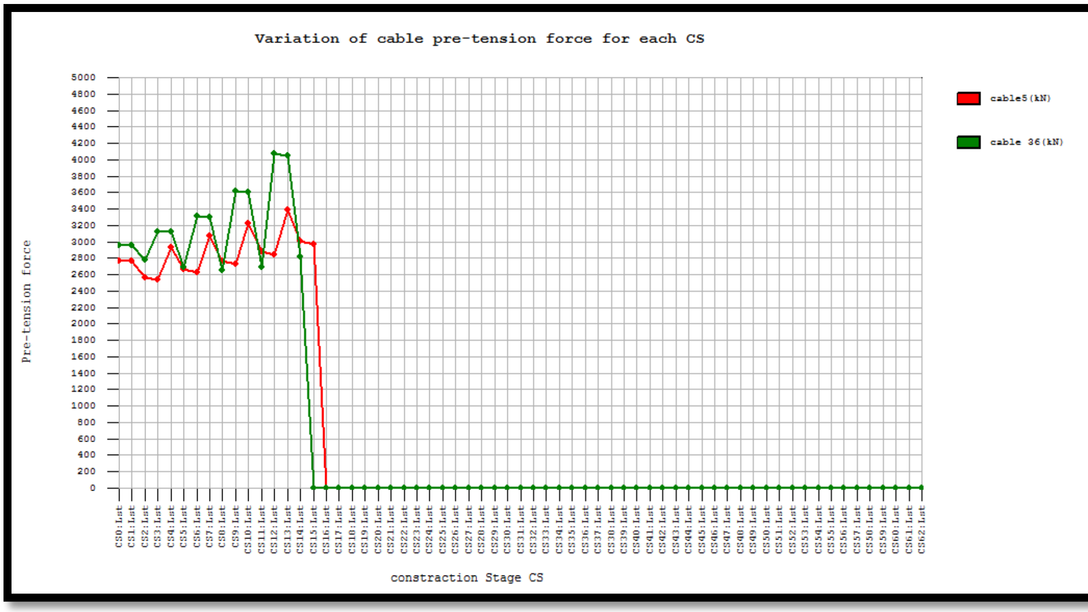

Figure (5-43): variation of cable pretension from cable 5 and cable 36 for each BCS

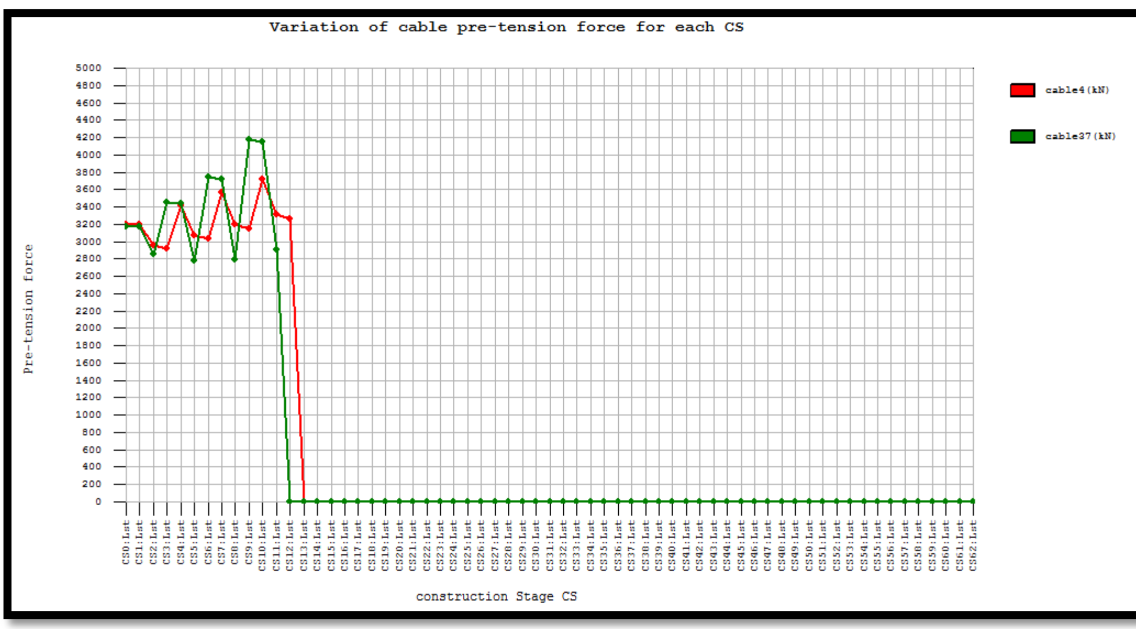

Figure (5-44): variation of cable pretension from cable 4 and cable 37 for each BCS

Figure (5-45), presents the different values of pre-tension force for cable 3and cable 38 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 3 is 4025.83 kN at BCS 7. Also, the maximum value for cable 38equals 4277.01 kN at BCS 6.

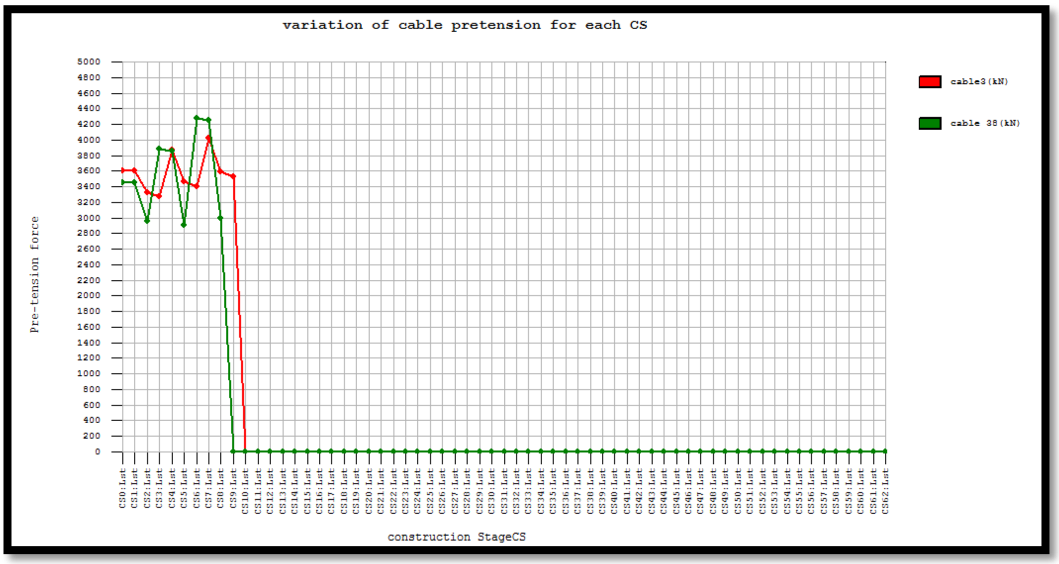

Figure (5-45): variation of cable pretension from cable 3 and cable 38 for each BCS

- Figure (5-46), presents the different values of pre-tension force for cable 2and cable 39 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 2 is 4365.88 kN at BCS 4. Also, the maximum value for cable 39equals 4337.77 kN at BCS 3.
- Figure (5-47), presents the different values of pre-tension force for cable 1and cable 40 during each construction stage BCS. It can be observed that the maximum value of pre –tension for cable 1 is 4610.40 kN at BCS 1. Also, the maximum value for cable 40 equals 3984.18 kN at BCS 1.

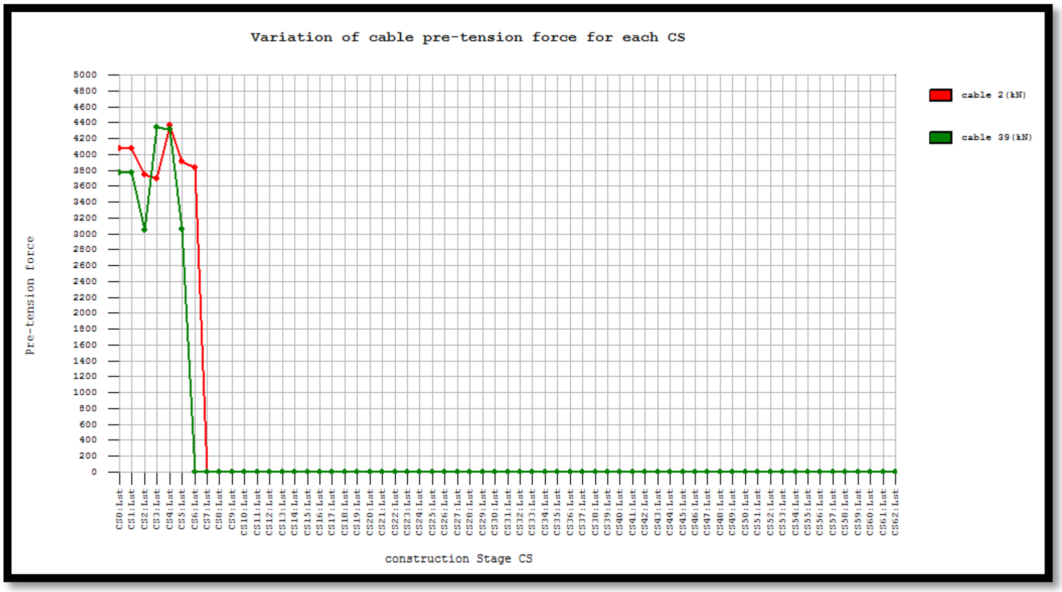

Figure (5-46): variation of cable pretension from cable 2 and cable 39 for each BCS

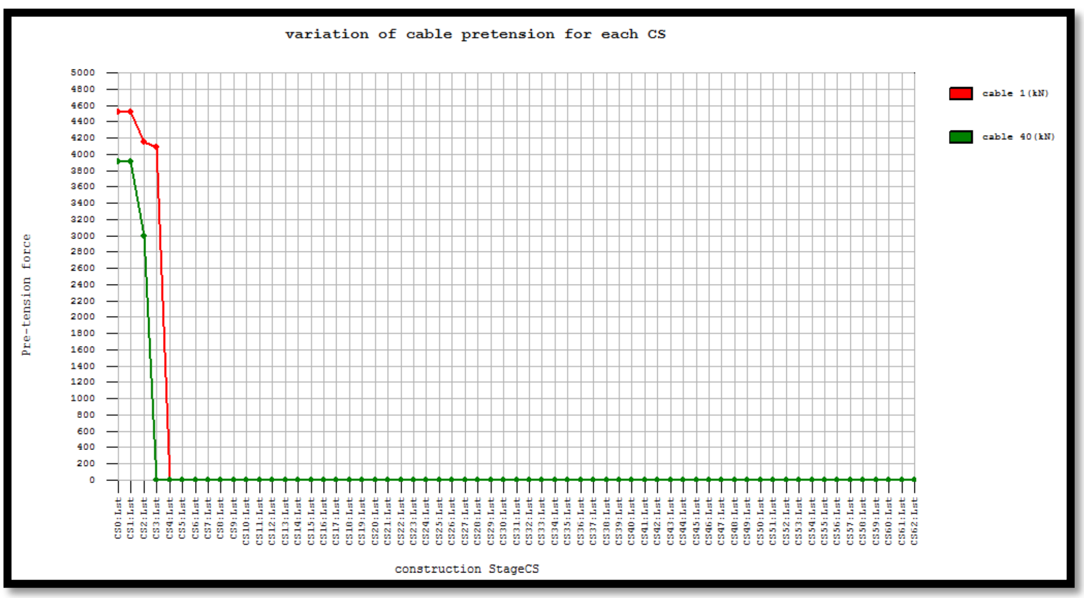

Figure (5-47): variation of cable pretension from cable 1 and cable 40 for each BCS

5.3.2 Displacement

Figure (5-48), shows the graphic variation of horizontal displacements for the towers and vertical displacements for the main girders at the ¼ point location of a side span for each construction stage BCS.

The maximum vertical displacement in the girder equals 0.039 m at BCS61and the minimum vertical displacement in the girder equals -0.014m at BCS31. Also, the maximum horizontal displacement in the tower equals 0.043m at BCS62and the minimum horizontal displacement in the tower equals -0.053m at BCS 57.

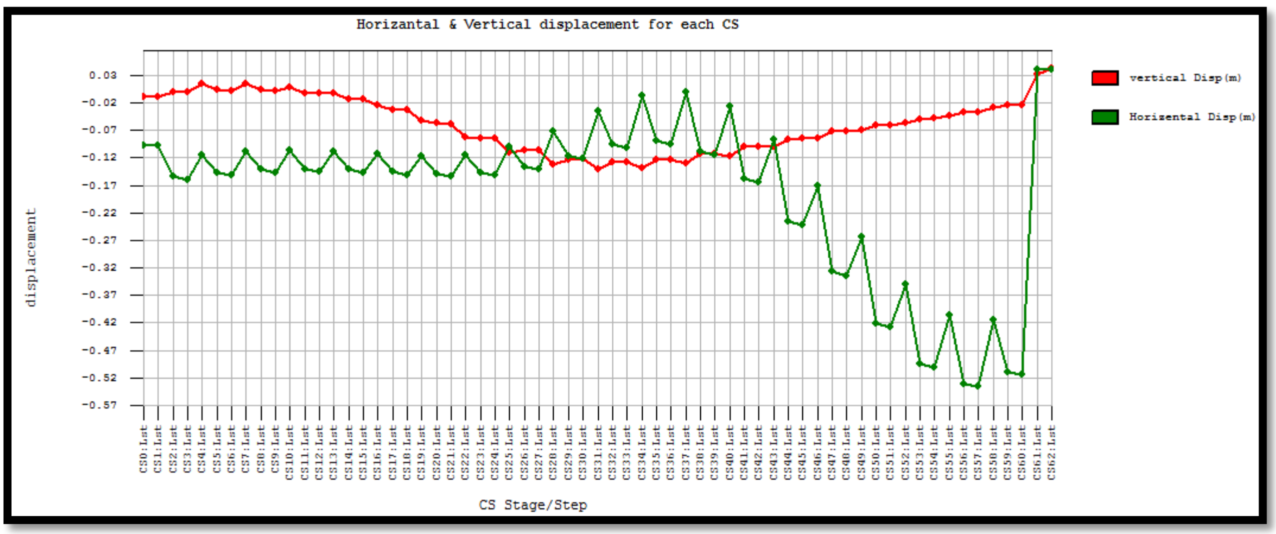

Figure (5-48): Variation of horizontal in the tower and vertical displacement in the girder for each BCS

5.3.3 Cable stresses

For each construction stage, the variation of cable stresses is presented by using the Step History Graph function from the final stage BCS0, as shown in Figure (5-49). The maximum value of cable stress is 260 N/mm^2 at first cable during BCS0. Also, Figure (5-50) and Figure (5-51) show samples of the maximum value of cable stress during the construction stages BCS10, BCS30 to be 240N/mm^2, 195N/mm^2

respectively. The similar samples of the cable stresses are presented in the Appendix (C) from Figure (C-1) to (C-9).

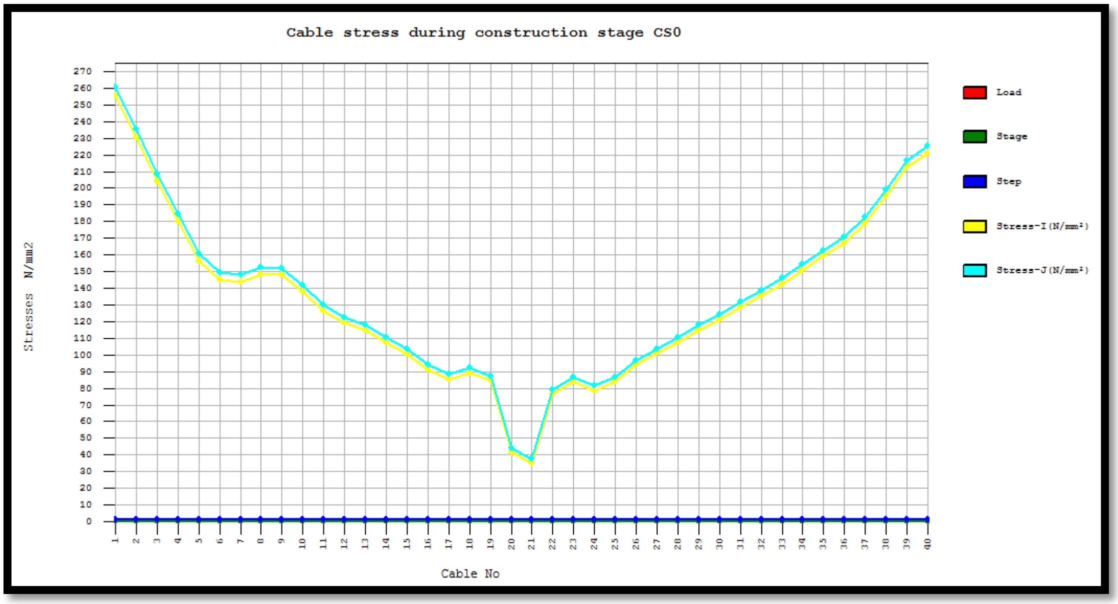

Figure (5-49): Cables stress variation during BCS0

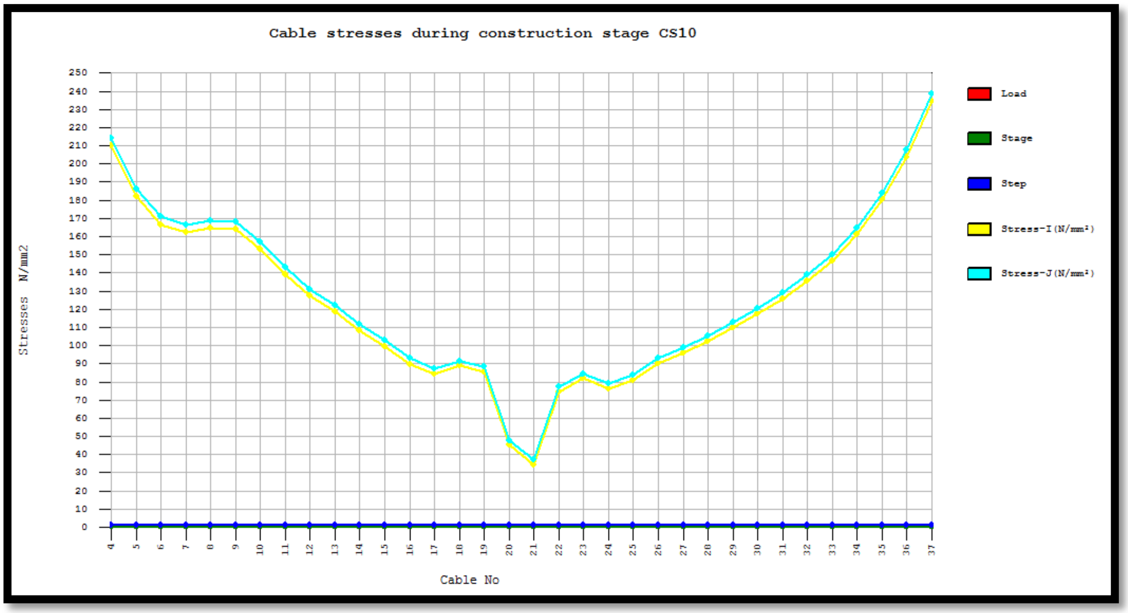

Figure (5-50): Cables stress variation during BCS10

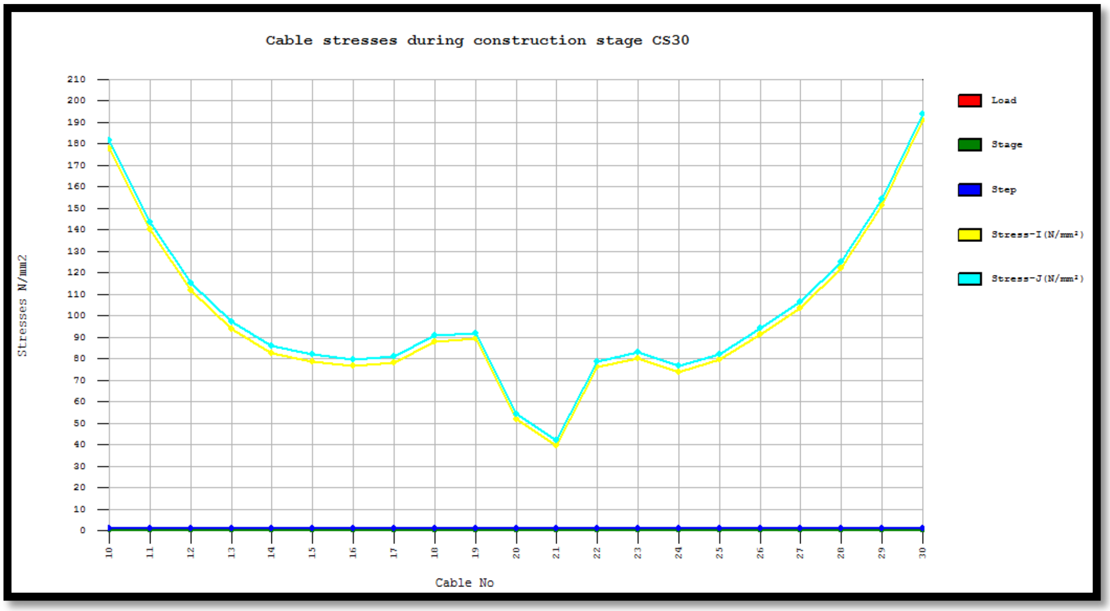

Figure (5-51): Cables stress variation during BCS 30

- Figure (5-52), shows the maximum values of cable stress during the construction stages BCS45 is 150.10 N/mm^2 , Figure (5-53) shows the maximum values of cable stress during the construction stages BCS47 is 116.6 N/mm^2 .
- Looking at these plans, it can be noted that the change in the number of cables during the different construction stages, results in a change in the shape of the distribution of stresses, and this change appears clearly in Figure (5-52)and Figure (5-53).

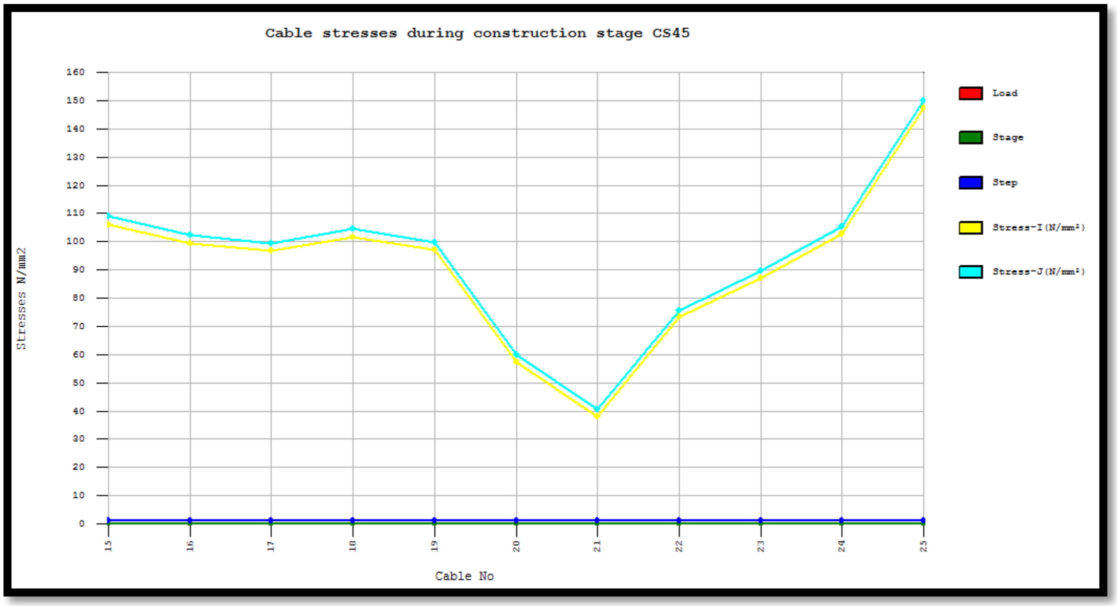

Figure (5-52): Cables stress variation during BCS 45

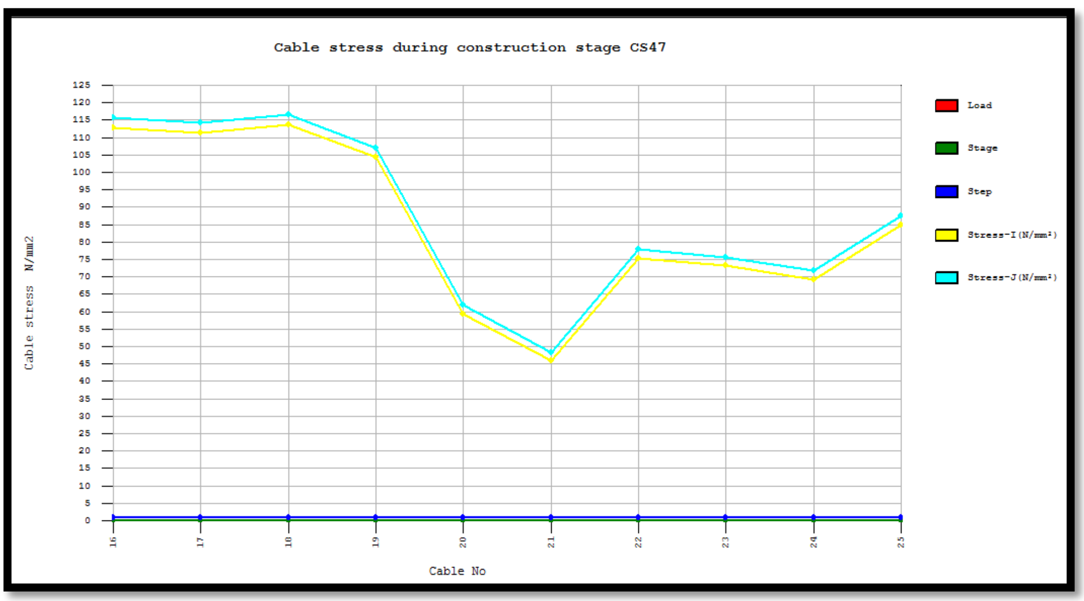

Figure (5-53): Cables stress variation during BCS 47

- Figure (5-54), shows the maximum values of cable stress during the construction stages BCS49 is 179.70 N/mm^2 Figure (5-55), shows the maximum values of cable stress during the construction stages BCS50 is 140 N/mm^2

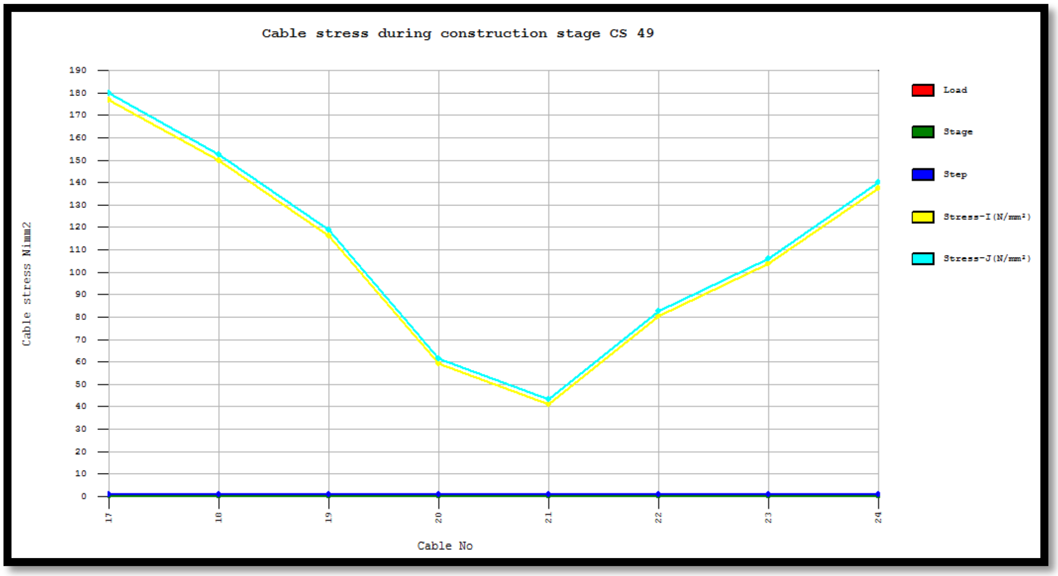

Figure (5-54): Cables stress variation during BCS49

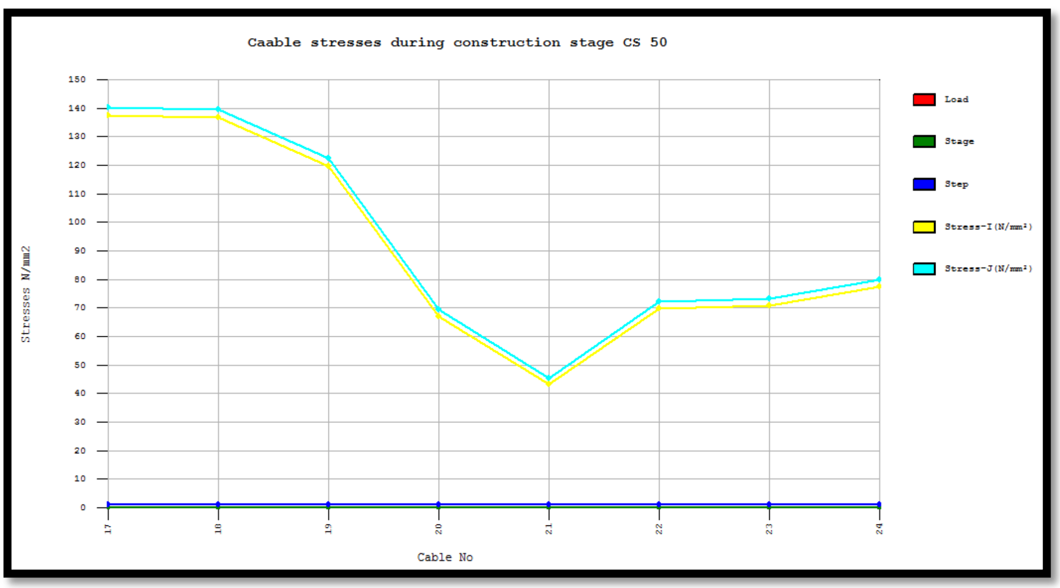

Figure (5-55): Cables stress variation during BCS 50

- Figure (5-56), shows the maximum values of cable stress during the construction stages BCS 51 is 140.4N/mm^2 Figure (5-57), shows the maximum values of cable stress during the construction stages BCS 52 is 226.8 N/mm^2

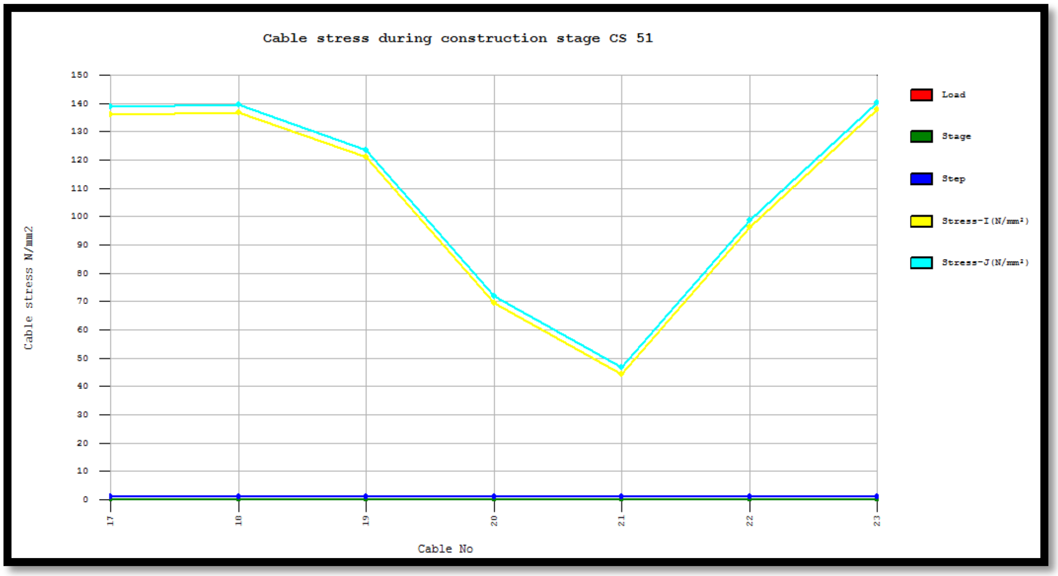

Figure (5-56): Cables stress variation during BCS 51

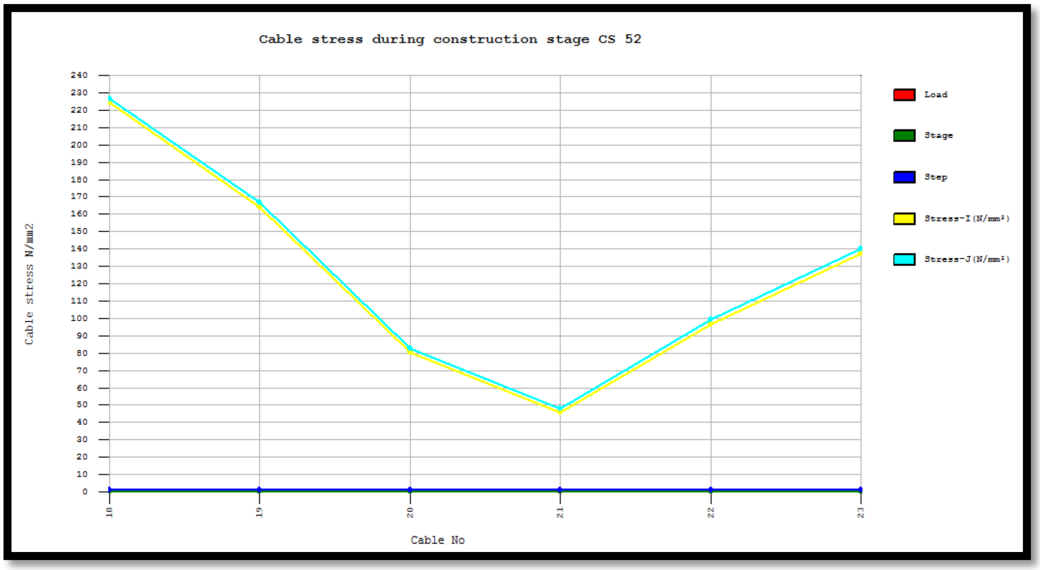

Figure (5-57): Cables stress variation during BCS 52

- Figure (5-58), shows the maximum values of cable stress during the construction stages BCS 53 is 195.4 N/mm^2 Figure (5-59), shows the maximum values of cable stress during the construction stages BCS 54 is 194.5 N/mm^2

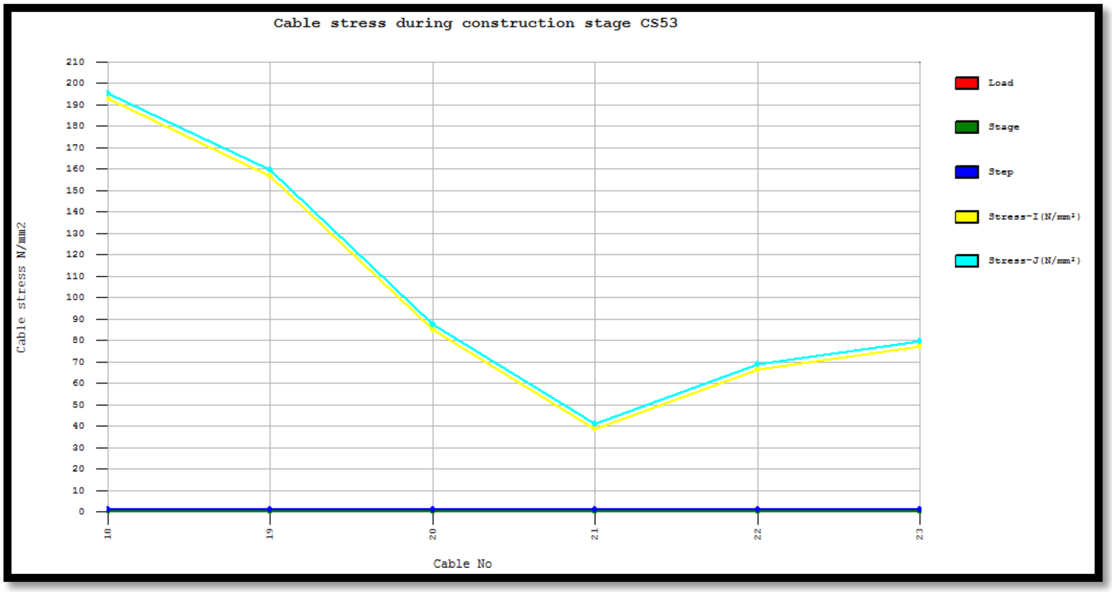

Figure (5-58): Cables stress variation during BCS 53

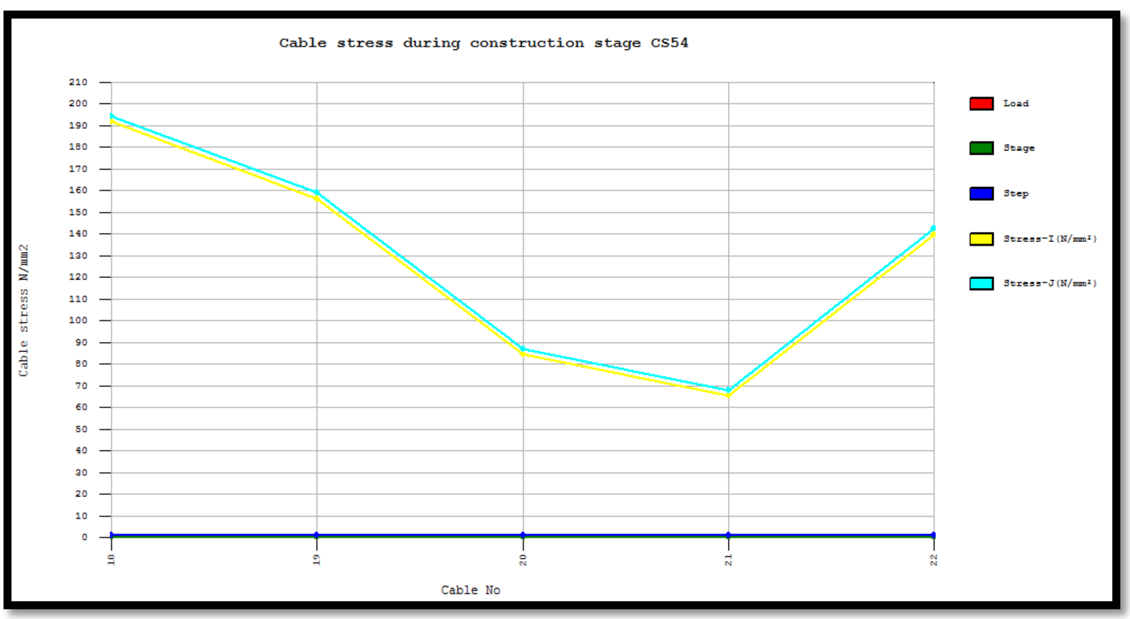

Figure (5-59): Cables stress variation during BCS 54

- Figure (5-60), shows the maximum values of cable stress during the construction stages BCS 55 is 297 N/mm^2 Figure (5-61), shows the maximum values of cable stress during the construction stages BCS 56 is 275.10 N/mm^2

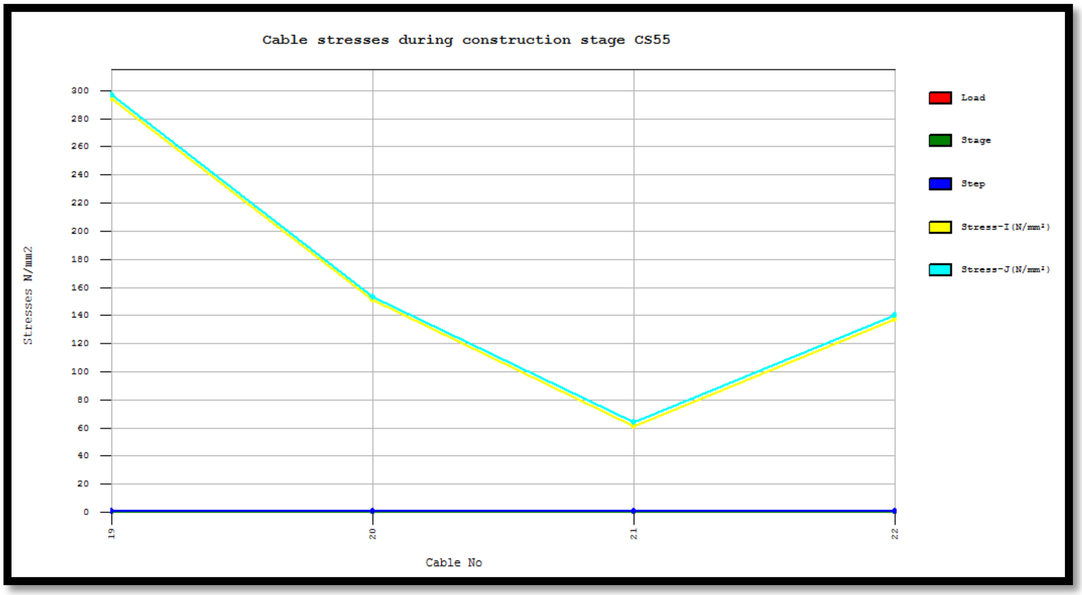

Figure (5-60): Cables stress variation during BCS55

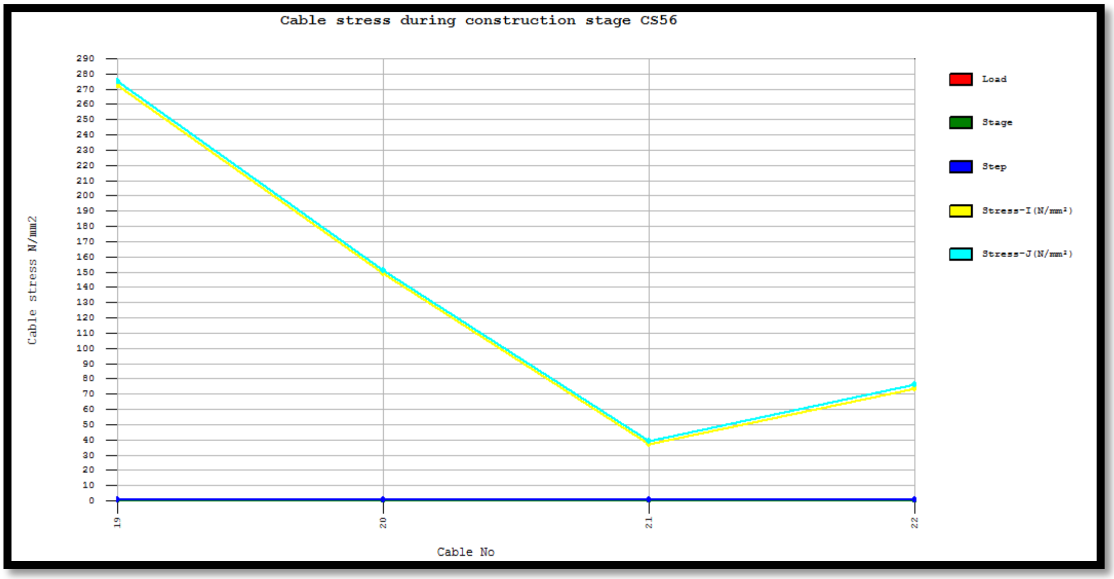

Figure (5-61): Cables stress variation during BCS56

- Figure (5-62), shows the maximum values of cable stress during the construction stages BCS 57 is 270 N/mm^2 Figure (5-63), shows the maximum values of cable stress during the construction stages BCS 58 is 386.10 N/mm^2

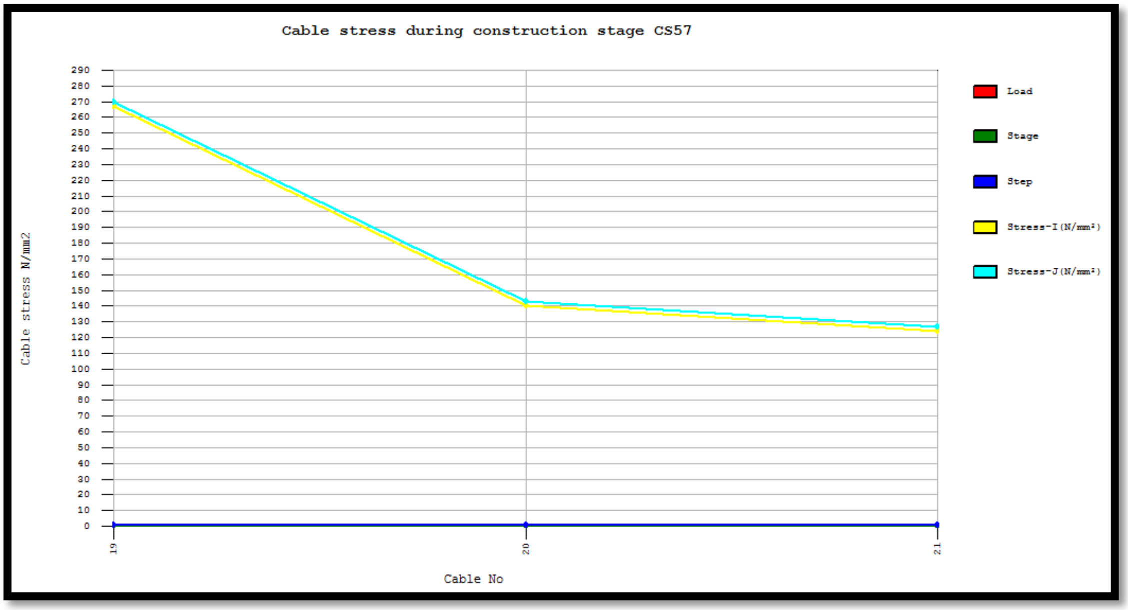

Figure (5-62): Cables stress variation during BCS57

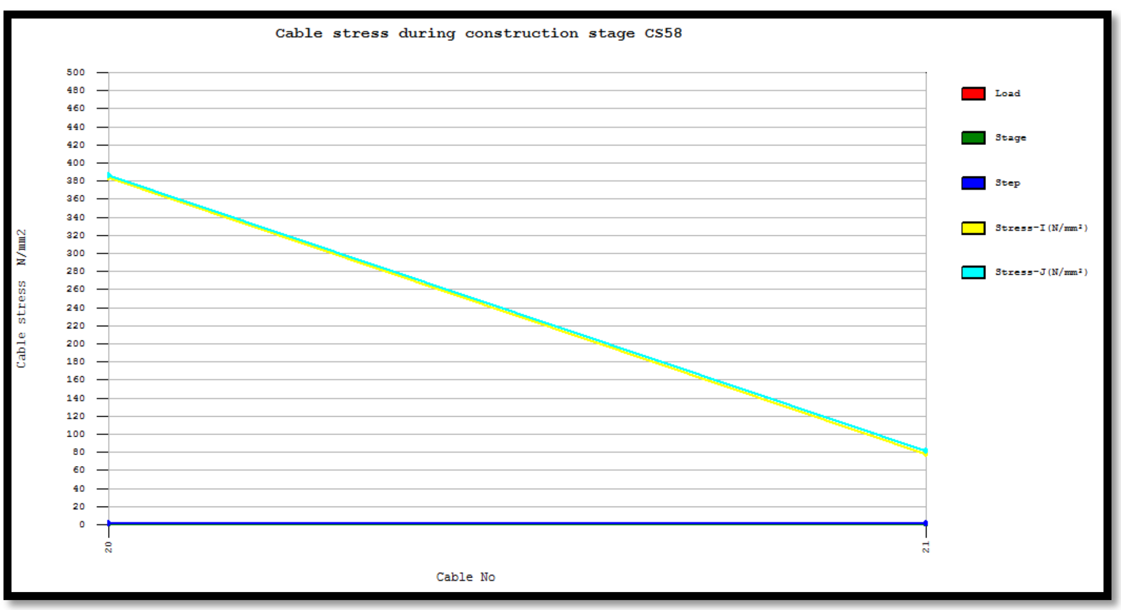

Figure (5-63): Cables stress variation during BCS58

- Figure (5-64), shows the maximum values of cable stress during the construction stages BCS 59 is 394.10 N/mm^2

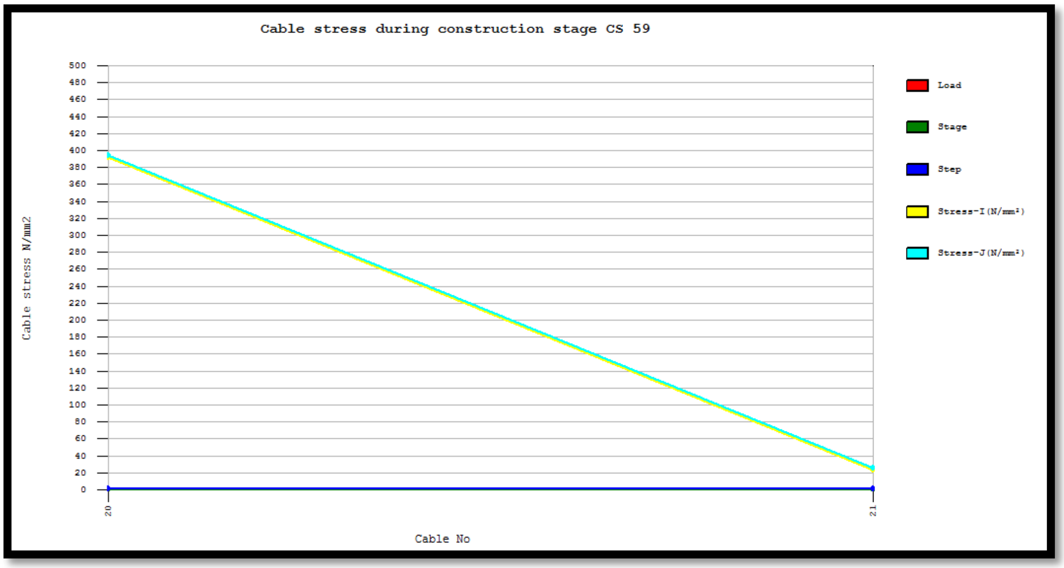

Figure (5-64): Cables stress variation during BCS59

5.4 Summary of results:

5.4.1 Result of displacement

The horizontal displacement that occurs at the top of the tower and vertical displacement in the center of main girder during construction stages are shown in Table (5-5).

Table (5-5): Result of Maximum and Minimum displacement during different construction stage

Description	Main girder z (m)	Top tower x(m)	Maximum Allowable rang AASHTO 2010 Main girder	Maximum Allowable rang AASHTO 2010 Tower
max horizontal displacement at BCS62		0.043m		
min horizontal displacement at BCS57		0.053m	L/800 = 0.375 m	H/300= 0.22m
max vertical displacement at BCS61	0.039m			
min vertical displacement at BCS31	-0.014m			

5.4.2 Result of stresses

The sample of the maximum value of cable stress during construction stages BCS0, BCS10, BCS30, BCS45, BCS55, BCS50, and BCS59 is shown in Table (5-6). From this table, it can be noted that the maximum value of stress occurs at BCS59 and the minimum values occur at BCS47.All these values are less than the allowable range according to AASHTO 2010, and are very small compare to the allowable because the load applied in the different stages of construction is the dead load only.

Table (5-6): Result of stress during different construction stage

Description	**Cable stress**	**Maximum Allowable range AASHTO 2010**
Last stage construction BCS0	260 N/mm^2	837N/mm^2
BCS10	240 N/mm^2	-
BCS30	195 N/mm^2	-
BCS45	192 N/mm^2	-
BCS47	116.6 N/mm^2	-
BCS49	179.70 N/mm^2	-
BCS50	140 N/mm^2	-
BCS 51	140.4N/mm^2	-
BCS 52	226.8 N/mm^2	-
BCS 53	195.4 N/mm^2	-
BCS 54	194.5 N/mm^2	-
BCS 55	297 N/mm^2	-
BCS 56	275.10 N/mm^2	-
BCS 57	270 N/mm^2	-
BCS 58	386.10 N/mm^2	-
BCS59	394 N/mm^2	-

CHAPTER SIX

Summary, Conclusions and Recommendations

6.1 Summary:

Cable stayed bridges are complex structures consisting of various structural components with different stiffness and damping characteristics. They are more flexible than other types of bridges.

In this study, the unknown load factor optimization method was the method used to determine the cable forces for three different cable types (strand cable, new parallel wire, and Carbone fiber cable). The unknown load factor optimization method was used to determine the cable pre–tension forces to achieve a perfectly safe and stable bridge. The procedure was based on using finite element analysis programs. The cable tension of a cable stayed bridge is evaluated under the effect of Dead load (Self weight, additional loads), Initial pre- tension force in the cable, live load (moving load) and wind load. Then analysis of cable-stayed bridges at different erection stages during construction, the backward construction process analysis was carried out. The objective of the construction stage analysis simulation to identify stresses and deformations of the concrete girder and towers, as well as the cable tension stress, to meet the design requirements was achieved. The results obtained during the analysis were compared with the AASHTO 2010 requirements and were found to be within the allowable limits

6.2 Conclusions:

The current study presents abstract of arrangement types of cable –stayed bridge, tower types, and deck systems and cables type. Also, studied the various loads condition, method of non-linear analysis and construction methods.

The analysis of the cable - stayed bridge was divided into two parts. In the first part, bending moment, axial forces and deflections due to dead and live loads were determined. In the second part, the pre – tensioning forces in the cables required to reduce to specified value the stresses determined in the first part were calculated.

A finite element methodology was adopted for the analysis of the TUTI BAHARI cable-stayed bridge. The analysis had been used to determine the pre-tension force in the cable under different loads conditions and different cable types, for each construction stage and to identify the consequent deformation and stress of the structure. The Finite Element (FEM) analysis program MIDAS Civil has been applied in the analysis process. The results obtained of static analysis; wind load and backward construction stage analysis were compared with the AASHTO 2010, requirements.

6.2.1 Analysis and design of cables stages:

1. The bridge was first analyzed for the dead loads by static analysis to get the deformed configuration. The target of static analysis was to get the initial deformed shape of the cable stayed bridge; Deformation under the self-weight of the structure was found to be small as required.
2. The ideal state of the structure system had been developed by an appropriate cable pre-tensioning, where in unknown load factors were applied in the

analysis. With the restriction of the moment and vertical displacement, a continuous beam condition for the main girder had been achieved.

3. The ideal cable pre-tension forces had been determined to achieve an optimal structural performance due to its permanent loads.
4. The geometric non-linear theory of cable – stayed bridge has been studied; but despite being presented in the methodology and objective, it was not formulated and the required program was not developed ,implemented and applied for analysis of cable stayed bridges.
5. The maximum cable force under LCB2 is equal 6670 kN from case 1, is 6604.180 kN from case 2 and is equal 6276.7451 kN from case3, these occurs in the first cable respectively. Also, the maximum cable force under LCB3 is 7229 KN
6. The maximum values of the horizontal displacements at the tower top under LCB2 is equal 0.033308m from case1, is equal 0.033166m from case 2, and is equal 0.031438m from case 3 and under LCB3 is equal 0.060295m.The maximum values of vertical displacement at the center of main girder under LCB2 from case1 is -0.001m, from case 2 is -0.009310m, and from case3 is -0.008173m, and under LCB3 0.026m.
7. The maximum cable stress under LCB2 is equal 377 N/mm^2 from case1, is equal 407 N/mm^2 from case2, and is equal 355 N/mm^2 from case3, and under LCB3 is equal 408.5 N/mm^2.This maximum stress occurs in the two long stay cables with maximum pre-tensions.
8. It can be observed from the above result number 5 and 6 and 7, that the result obtained from the use of Carbone fiber cable (case3), for the cable forces stress and displacement is lower than these obtained by the use of the other types of cable (case 1 and case2).

9. The maximum value of horizontal displacement under applied wind pressure on the structure at the tower top is 0.0372m and the maximum value of vertical displacement is -0.0016m in the Center of main girder.
10. The maximum horizontal displacement in top of the tower and vertical displacement in the main girder under dead load, live and wind load stages are controlled and within the allowable range.
11. Carbon fiber cables have high strength and protection against the corrosion compare with the steel cable. Also, the value of displacement obtained from this cable is lower than other types of cable.
12. The maximum stress in the cable had occurred in the cable with the greatest pre tension force. These stresses are within the allowable range.
13. The simple equation (preliminary manual calculation) were used to calculate the initial pre –tension forces in cables for a cable stayed bridges as a first estimate, the results obtained in Table (4-19) showed that there is a difference between them in the obtained values, which were obtained from Midas Civil program, the max difference is 26% between these values.
14. The dynamics analysis of cables results show that the effect of damping is small and can be neglected. The cable –stayed bridge system is stable under wind load analysis.
15. The effect of construction stages on the design of cables only was studied. The study of such effect on other elements of the bridge should be studied.
16. It can be noted from the diagrams of the cable pre-tension force and cables stress, that there is non-symmetry in the values of pre tension force and stresses. Further studies are required to determine the causes of non-symmetry.

6.1.2: Construction stage analysis

1. The numbers of different construction stages required for the cable –stayed bridge were calculated assuming Equation (3.17).
2. The maximum value of pre –tension for cables 20 reached the 6930.77 kN at BCS 59. Also, for cable 21 the maximum value of pre-tension is equal 2197.69kN at BCS57.these cable are the first cables in construction.
3. The maximum value of cable stresses is equal 394 N/mm^{2}at the first cable during construction stage at BCS59, the maximum value of cable stress under different construction stage BCS0, BCS10, BCS30, BCS45, and BCS50 is equals to 260N/mm^2, 240N/mm^2, 195N/mm^2, and 192N/mm^{2}and 140N/mm^2 respectively. It can be observed that the cables stress is various during construction stage. Also, the maximum value of cable stress occurs at the cable with large value of pre - tension force during construction stage.
4. The maximum vertical displacement in the girder is equal 0.039 m at BCS62, while the minimum vertical displacement in the girder is equal -0.014m at BCS31, also, the maximum horizontal displacement in the tower is equal 0.043m at BCS62, the minimum horizontal displacement in the tower is equal - 0.053m at BCS 57.
5. During the construction stage the pre- tensions in the cables were changing, because every cable was stressed initially in the installation stage.
6. Maximum value of the cable stresses occur at the outer cable, this value within the allowable range
7. Observed that the cable –stayed bridge under this study during the static analysis, wind load and each construction stages are stable and this result is reasonable.

6.3 Recommendations:

- From the study results it is recommended to:

1. Use carbon fiber cables for fan arrangement type cable stayed bridges.
2. Use the formula (3.17) established for construction stage analysis of fan type of cable –stayed bridge to determine the required number of stages.
3. Adopt the simplified equations, proposed by Podolny and Scalzi, (1986), for calculation of pre-tension which were verified in Table (4-19), for preparing initial arrangements or as an initial estimate for the trial and error method for determining initial pre-tension forces for cable –stayed bridges.
4. Adopt free vibration analysis for dynamic analysis of cables in cable- stayed bridges.

- For future studies it is recommended to :

1. Analyze the cable –stayed bridge by considering the other type of tower and arrangement of cable and compare with this study.
2. Use the equations modified and presented in Appendix (B) to develop a model of the cable – stayed bridge using EXCEL or MATLAB to obtain the pre-tension force for cable stayed bridge
3. Use the forward construction stages analysis for the modeling of cable – stayed bridge, and compare the result with the backward construction stage analysis.
4. Formulate develop, implement and apply a geometrically non-linear finite element model for the analysis of cable –stayed bridge.
5. Carry out and verify the design of the bridge elements, other than cables, taking into account the effect of construction stages.
6. Study the causes of the non-symmetric of cable pretension forces and cable stresses during the evaluation of the pretension forces and construction stage analysis.

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APPENDIX (A): Figures and Tables from chapter two

This appendix contains paragraphs and tables from chapter two.

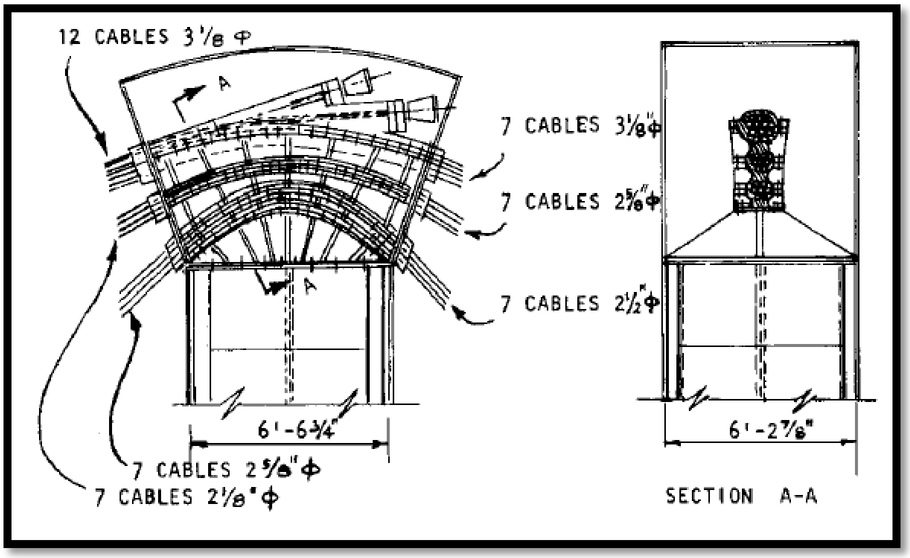


Figure A-1: Three – level cable connectioms ,Troitsky ,(1988)

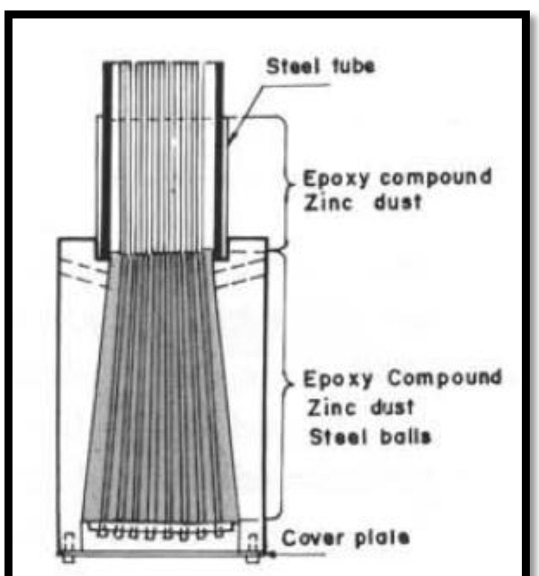


Figure A-2: HiAm socket, Troitsky, (1988)

Table (A-1): Load Combinations and Load Factors, AASHTO (2010)

										Use One of These at a Time			
Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	*EQ*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Extreme Event I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—
Fatigue I—*LL, IM & CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—
Fatigue I II—*LL, IM & CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—

Table A-2: Permanent &Transient loads, AASHTO (2010)

Permanent Loads		Transient Loads	
CR	Force effects due to creep.	BR	Vehicular braking force
DD	Down drag force.	CE	Vehicular centrifugal force
DC	Dead load of structural components and nonstructural attachments	CT	Vehicular collision force
DW	Dead load of wearing surfaces	CV	Vessel collision force
EH	Horizontal earth pressure load	EQ	Earthquake load
EL	Miscellaneous locked-in force effects resulting from the construction process.	FR	Friction load
ES	Earth surcharge load.	IC	Ice load
EV	Vertical pressures from dead load of earth fill.	IM	Vehicular dynamic load allowance
PS	Secondary forces from post-tensioning.	LL	Vehicular live load
SH	Force effects due to shrinkage.	LS	Live load surcharge
		PL	Pedestrian live load
		SE	Force effect due to settlement
		TG	Force effect due to temperature gradient
		TU	Force effect due to uniform temperature
		WA	Water load and stream pressure
		WL	Wind on live load

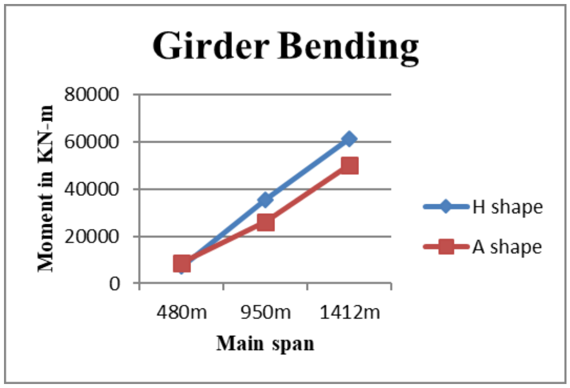


Figure A-3: Girder Bending

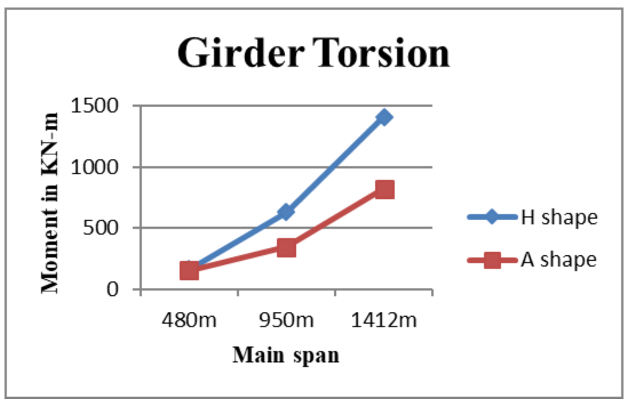


Figure A-4: Girder Torsion

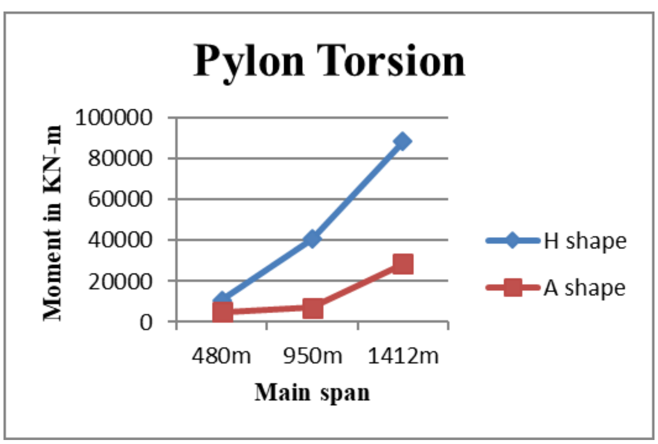


Figure A-5: Pylon Torsion

Table1A-3: Calculations for wind load on each cable

(As per cl 209, pg 23, IRC:6-2010)

Cable No.	Diameter	Length	Area	Total Force on 1st Plane (F_1)	Total Force on 2st Plane ($C_s \times F_1$)	Force on Each node on 1st Plane of cables	Force on Each node on 2nd Plane of cables
	mm	m	m^2	t	t	t	t
C1	160	177.01	28.32	6.36	6.30	3.18	3.15
C2	160	169.38	27.10	6.09	6.03	3.04	3.02
C3	160	160.05	25.61	5.75	5.70	2.88	2.85
C4	160	150.82	24.13	5.42	5.37	2.71	2.69
C5	115	141.70	16.30	3.66	3.63	1.83	1.81
C6	120	132.71	15.93	3.58	3.54	1.79	1.77
C7	130	123.88	16.10	3.62	3.58	1.81	1.79
C8	130	115.24	14.98	3.37	3.33	1.68	1.67
C9	120	106.84	12.82	2.88	2.85	1.44	1.43
C10	115	98.72	11.35	2.55	2.53	1.28	1.26
C11	110	90.96	10.01	2.25	2.23	1.12	1.11
C12	110	83.62	9.20	2.07	2.05	1.03	1.02
C13	110	76.75	8.44	1.90	1.88	0.95	0.94
C14	110	70.36	7.74	1.74	1.72	0.87	0.86
C15	100	64.04	6.40	1.44	1.43	0.72	0.71
C16	100	54.67	5.47	1.23	1.22	0.61	0.61
C17	100	54.67	5.47	1.23	1.22	0.61	0.61
C18	100	64.04	6.40	1.44	1.43	0.72	0.71
C19	105	70.36	7.39	1.66	1.64	0.83	0.82
C20	105	76.75	8.06	1.81	1.79	0.91	0.90
C21	110	83.62	9.20	2.07	2.05	1.03	1.02
C22	110	90.96	10.01	2.25	2.23	1.12	1.11
C23	120	98.72	11.85	2.66	2.64	1.33	1.32
C24	120	106.84	12.82	2.88	2.85	1.44	1.43
C25	120	115.24	13.83	3.11	3.08	1.55	1.54
C26	125	123.88	15.49	3.48	3.45	1.74	1.72
C27	125	132.71	16.59	3.73	3.69	1.86	1.85
C28	125	141.70	17.71	3.98	3.94	1.99	1.97
C29	160	150.82	24.13	5.42	5.37	2.71	2.69

Table A-4: Axial force in the cables under normal condition 5. Vikaset al, (2013),

Cable No.	Axial Force (kN)	Cable No.	Axial Force (kN)
1	1329	11	386
2	1283	12	649
3	1162	13	825
4	1123	14	976
5	1079	15	1093
6	1012	16	1160
7	992	17	1223
8	955	18	1267
9	808	19	1271
10	627	20	1244

Table A-5 : Axial force in the cables after cable 19 reached ultimate tensile strength (Failed) due to corrosion, 5. Vikaset al, (2013)

Cable	Axial Force (kN)	Cable	Axial Force (kN)
1	1263	11	382
2	1221	12	641
3	1113	13	818
4	1091	14	986
5	1062	15	1138
6	1007	16	1274
7	993	17	1423
8	958	18	1537
9	812	19	0.00
10	628	20	1505

APPENDIX (B): Equation and Application in Midas civil

This Appendix included equations from chapter three and steps for analyzing the model in the Midas Civil program.

- **Calculation of post –tensioning forces**

It is proposed to use these equations below, which have been modified to develop a model of the cable – stayed bridge, these equation for analysis under dead load only .They were proposed by Troitsky,(1988), for a limited number of cables .They have been modified for any number of cables .can be easily programmed to reduce the bending moments and deflection of the stiffening girder, after erection, the cable –stayed bridge is under the action is dead load only. The bending moments and the deflections of the stiffening girder may be reduced by post- tensioning the cables. A procedure which permits the reduction of the maximum bending moment due to dead load may be programmed on a digital computer. Released structure will be chosen as shown in Figure B-1

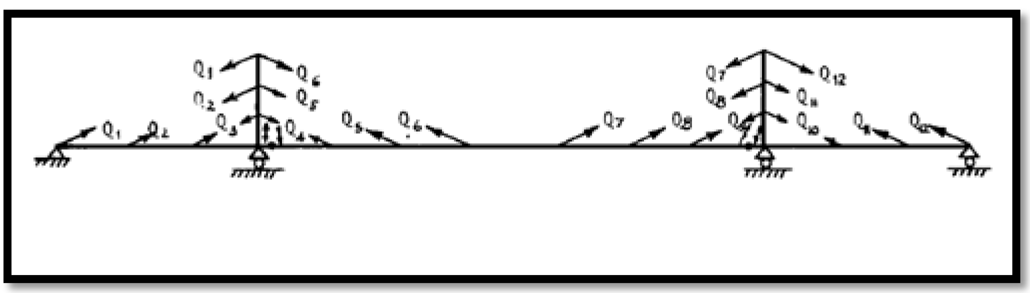


Figure B-1 Selection of cables as redundant Troitsky, (1988).

To determine unit displacements and bending moment due to unit loads applied along the cables, twelve superstructures are considered. Each superstructure consists of the original structure with one cable removed. Substructure No 1is represented in Figure B-2. The basic equations for this case are as follows

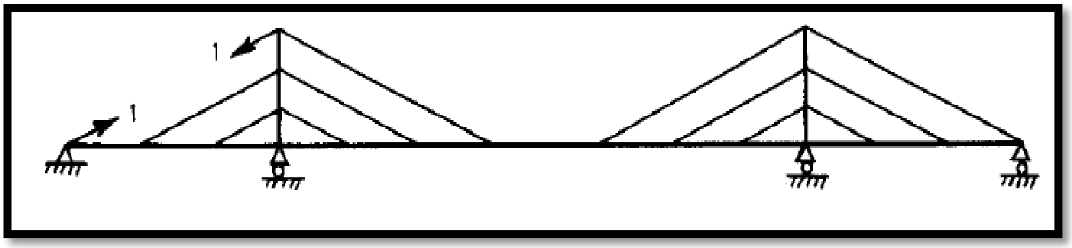


Figure B-2: substructure for calculation pre-tensioning forces ,Troitsky ,(1988).

$M_r^1 X_1 + M_r^n X_n = M_r(C_0 - 1)$…………………………...………………..……B.1

$\frac{N_1^f}{A_1} = \frac{N_2^f}{A_2} = \cdots = \frac{N_n^f}{A_n}$……………………………………………………...…..B.2

$a_{2,2}X_2 + a_{2,3}X_3 + \cdots + a_{2,i}X_i + \cdots + a_{2,n}\,X_n = A_{2,1} + A_{2,2}X_1$

$a_{3,2}X_2 + a_{3,3}X_3 + \cdots + a_{3,i}X_i + \cdots + a_{3,n}\,X_n = A_{3,1} + A_{3,2}X_1$ …………...B.3

$a_{n,2}X_2 + a_{n,3}X_3 + \cdots + a_{n,i}X_i + \cdots + a_{nn}\,X_n = A_{n,1} + A_{n,2}X_1$

The above Equation may be written in matrix form as

$[a] * \{X\} = \{A\}$…………………………………………………………......B.4

Where:

$[a] = \begin{bmatrix} a_{2,2} & a_{2,3} \cdots & a_{2,n} \\ a_{3,2} & a_{3,3} \cdots & a_{3,n} \\ & \vdots & \\ a_{n,2} & a_{n,3} \cdots & a_{n,n} \end{bmatrix}$……………..….,………..…………………B.5

$\{X\} = \{X_2, X_3, \cdots X_i, .., X_n\}$…………………………...……..……………..B.6

$\{A\} = \{A^1\} + \{A^2 X_1\}$ …………………...………………………….……...B.7

In B.7

$\{A^1\} = \{A_{2,1}, A_{3,1}, \cdots, A_{i \ldots\ldots}, A_{n,1}\}$……………………………………..B.8

$\{A^2\} = \{A_{2,2}, A_{3,2} \ldots\ldots, A_{i,2} \cdots A_{n,2}\}$……………………………………..B.9

From B.4

$\{X\} = [a]^{-1} * \{A\}$………………………………….................………..…B.10

$[a] * \{X\} = \{A\}$

Relation (B.1) may be rewritten as

$M_r(C_O - 1) = M_r^1 X_1 + (M_r) * \{X\}$……………..…..…B.11

Base on the method presented by Troitsky ,(1988),the basic equation for cables stayed with n cables are as follows:

Where

$\{M_r\} = (M_r^2, M_{r \ldots\ldots\ldots}^3, M_r^i \cdots M_r^n)$……………………………………………………………B.12

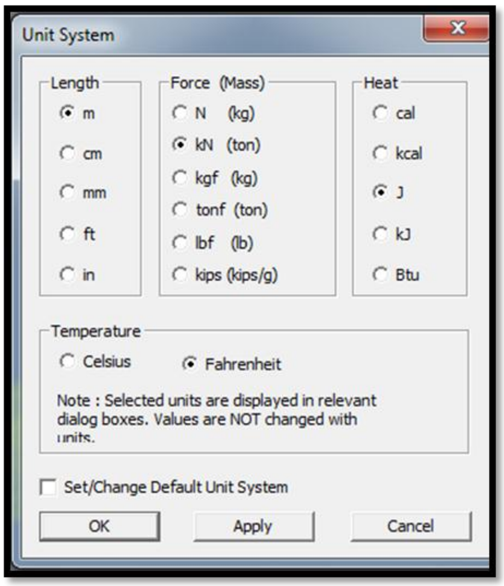
Substituting $\{X\}$ from (B.10) and taking account of (B.7),(B.11) becomes

$M_r(C_O - 1) = M_r^1 X_1 + (M_r * [a]^{-1}\{A^1\} + (M_r) * [a]^{-1} * \{A^2\} X_1)$………B.13

Steps of analysis the model of cable stayed bridge by using Midas civil

- **Step (1): Define Unit system**

- File / New Project
- File / Save (Cable Stayed model)
- Tools / Unit System
- Length > m; Force (Mass) > kN (ton) ↲



FigureB-3: Unit system

- **Step (2); Define Material and section properties**

- Click $\boxed{\text{Add}}$ button under Material tab in Properties dialog box
- Model / Properties / Material
- Material ID (1); Name (Cable); Type of Design > User Defined;
- User Defined > Standard > None; Type of Material > Isotropic;

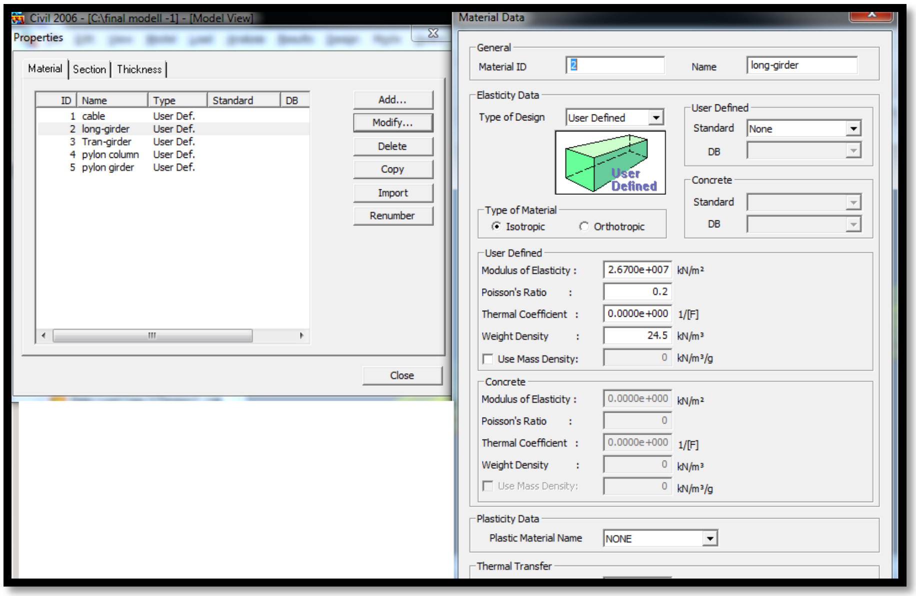


Figure B-4: Material properties

- Click 

button under Section tab in Properties dialog box.
- Model / Properties / Section

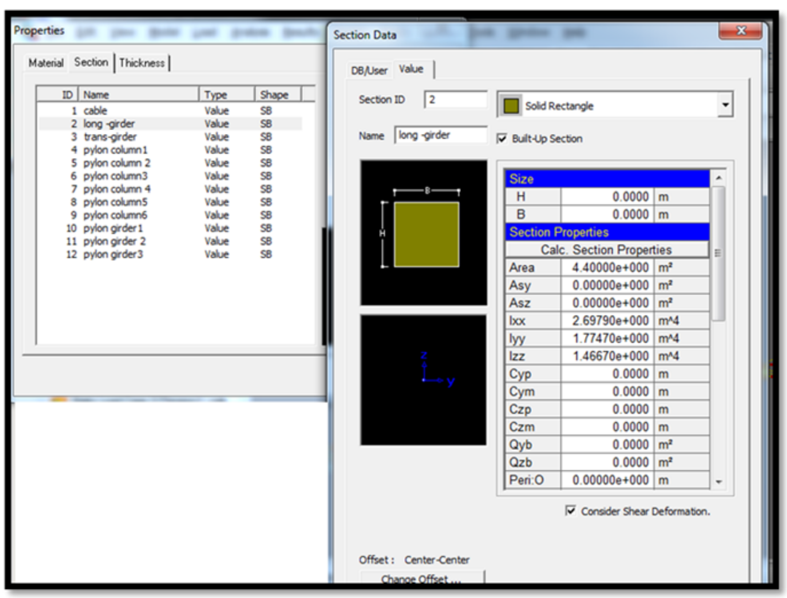


Figure B-5: Section Properties

➢ **Step (3): cable stayed bridge wizard**

- Model / Structure Wizard / Cable Stayed Bridge
- Type > Symmetric Bridge
- X (m) (0); Z (m) (7.18); B>X (m) (150); Z (m) (73.8)
- Height > H1 (m) (73.8)
- Material > Cable>1: Cable; Deck >2: long Girder; Tower >3: Pylon
- Section > Cable>1: Cable; Deck >2: long Girder; Tower >3: Pylon
- Selected Cable & Hanger Element Type>Truss

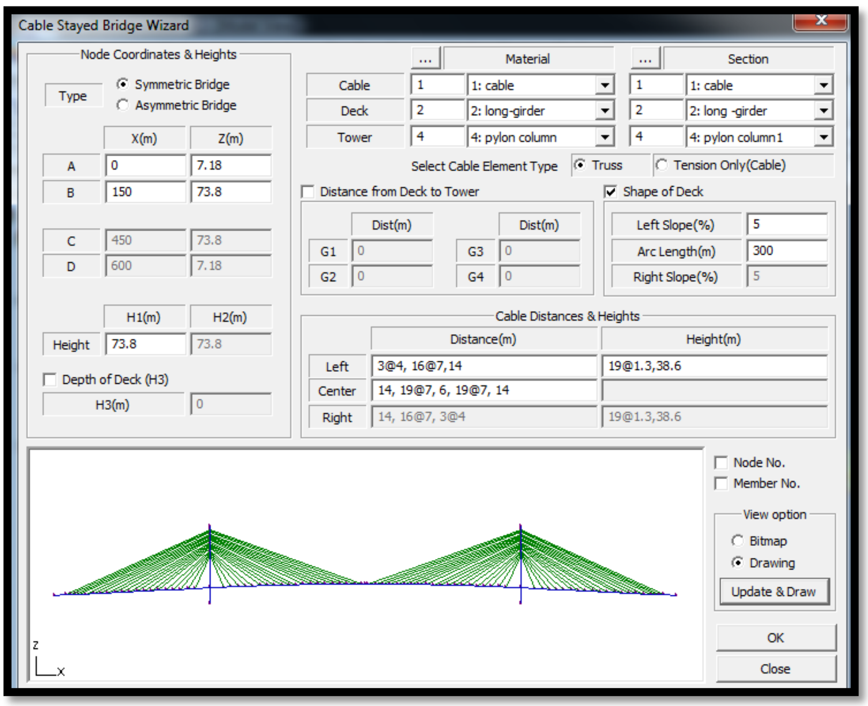
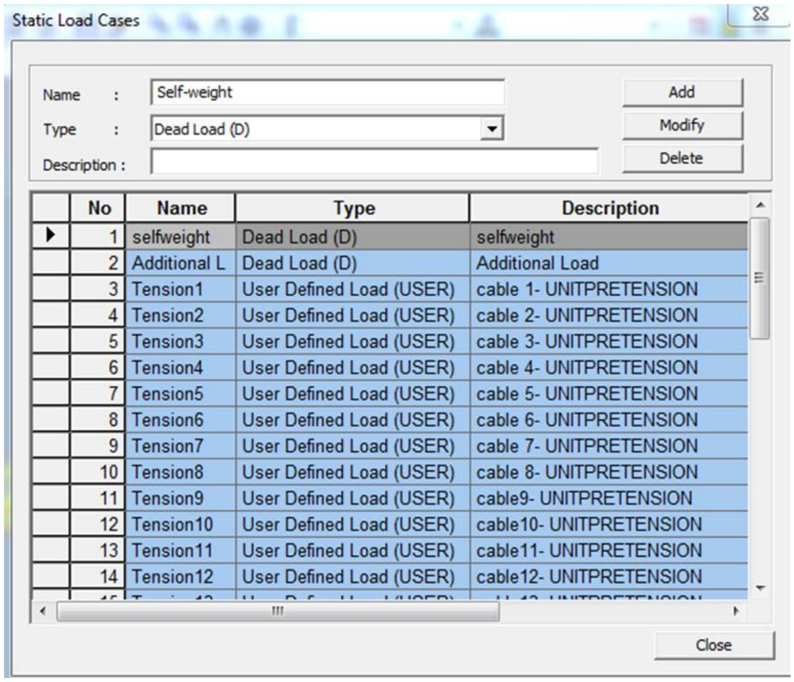


Figure B-6: cable stayed bridge wizard

➢ **Step (4): Loading Condition Input**

- Load / Static Load Cases
- Name (Self-Weight); Type>Dead Load
- Description (Self Weight) ↵
- Name (Additional Load); Type >Dead Load

- Description (Additional Load) ↵
- Name (Tension 1); Type >User Defined Load
- Description (Cable1- UNIT PRETENSION



	No	Name	Type	Description
▶	1	selfweight	Dead Load (D)	selfweight
	2	Additional L	Dead Load (D)	Additional Load
	3	Tension1	User Defined Load (USER)	cable 1- UNITPRETENSION
	4	Tension2	User Defined Load (USER)	cable 2- UNITPRETENSION
	5	Tension3	User Defined Load (USER)	cable 3- UNITPRETENSION
	6	Tension4	User Defined Load (USER)	cable 4- UNITPRETENSION
	7	Tension5	User Defined Load (USER)	cable 5- UNITPRETENSION
	8	Tension6	User Defined Load (USER)	cable 6- UNITPRETENSION
	9	Tension7	User Defined Load (USER)	cable 7- UNITPRETENSION
	10	Tension8	User Defined Load (USER)	cable 8- UNITPRETENSION
	11	Tension9	User Defined Load (USER)	cable9- UNITPRETENSION
	12	Tension10	User Defined Load (USER)	cable10- UNITPRETENSION
	13	Tension11	User Defined Load (USER)	cable11- UNITPRETENSION
	14	Tension12	User Defined Load (USER)	cable12- UNITPRETENSION

FigureB-7: Generated of Loading Conditions for Dead Loads and Unit Loads

- **Step (5): Loading Input**

- Load / Self Weight
- Load Case Name>Self-Weight
- Load Group Name>Default
- Self- Weight Factor>Z (-1) ↵
- Click Ok

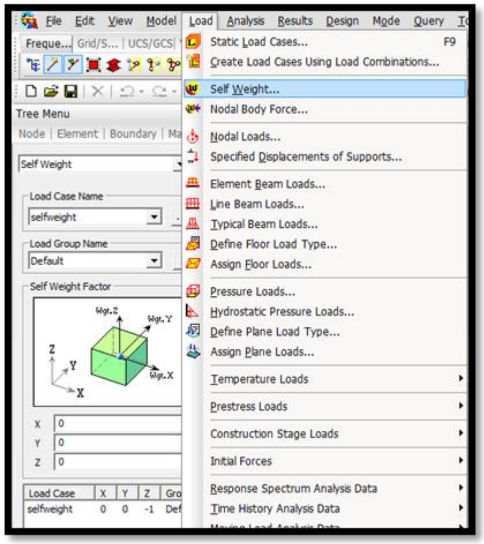


Figure B-8: Entering self-weight

- Load / Element Beam loads
- Select identity - Elements
- Select Type > Material > Girder ↵
- Load Case Name >Additional Load; Options >Add
- Load Type > Uniform Loads; Direction > Global Z
- Projection >Yes
- Value > Relative; x1 (0), x2 (1), w (-99.7) ↵

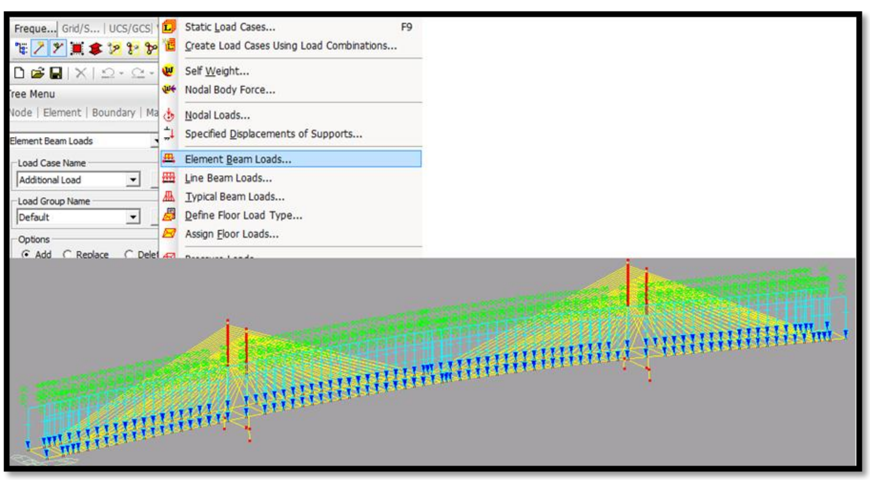


Figure B-9: Entering Superimposed Dead Loads to Main Girders

- Load / Pre-stress Loads / Pre-tension Loads
- Selected Intersect (Elements: A)
- Selected Intersect (Elements: B)
- Load Case Name>Tension 1; Load Group Name>Default
- Options>Add; Pretension Load (1) ↵

…

- Load Case Name>Tension 40; Load Group Name > Default
- Options > Add; Pretension Load (1) ↵

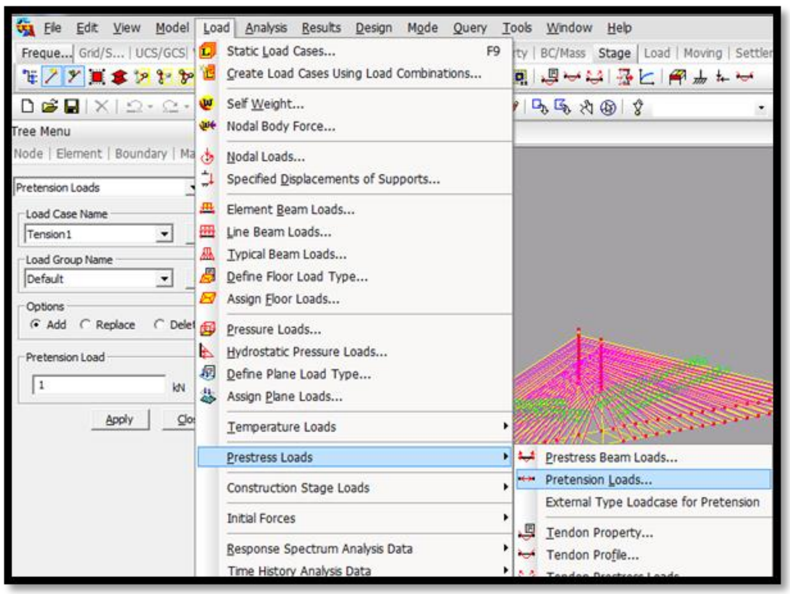


Figure B-10: Entering Unit Pretension Load to Cables

➢ **Step (6): Boundary conditions input**

- Model / Boundary / Supports
- Boundary Group Name>Default
- Options>Add; Support Type>D-ALL, R-ALL (on) ↵
- Model / Boundary / Rigid Link
- Boundary Group Name >Default; Options >Add/Replace

- Copy Rigid Link (on);
- Model / Boundaries / Elastic Link
- Options >Add; Link Type > General Type

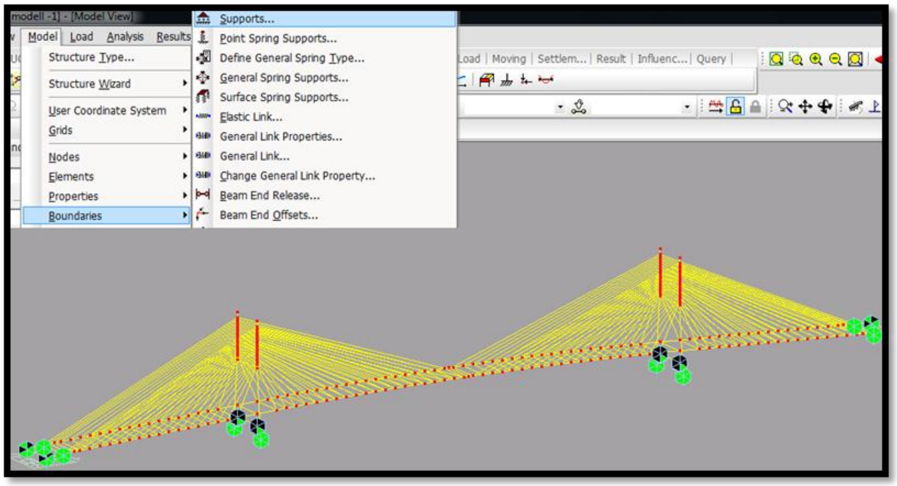


Figure B-11: boundary condition

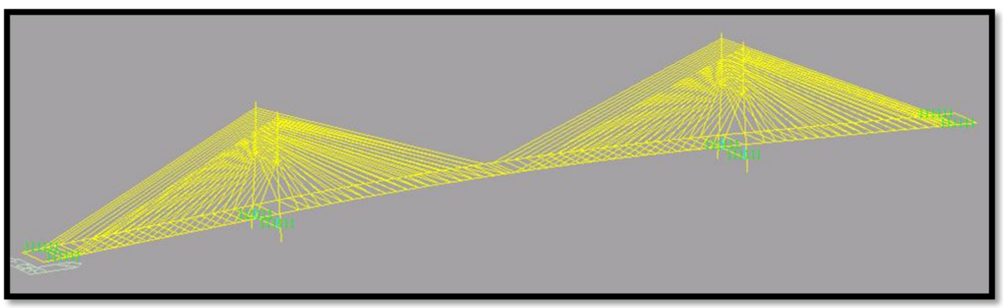


Figure B-12: rigid link

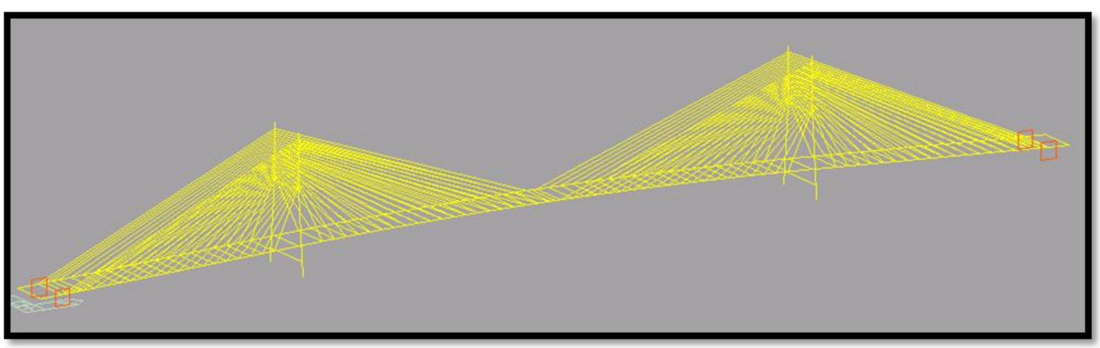


Figure B-13: elastic link

➢ **Step (7): Perform analysis**

- Analysis / Perform Analysis ↵

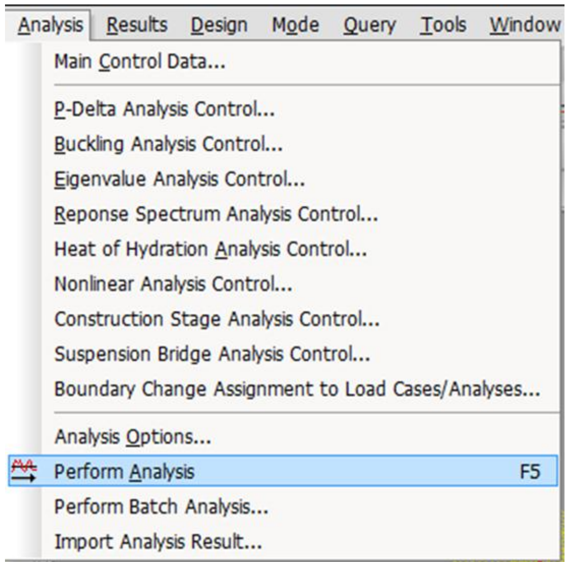


Figure B-14: Perform analysis

➢ **Step (8) Load Combinations for Dead Loads and Unit Loads**

- Results / Combinations

General Tab

- Load Combination List>Name>(LCB 1); Active>Active; Type>Add
- Load Case >Self-Weight (ST); Factor (1.0)
- Load Case >Additional Load (ST); Factor (1.0)
- Load Case >Tension 1(ST); Factor (1.0)

…

- Load Case>Tension 40(ST); Factor (1.0) ↵

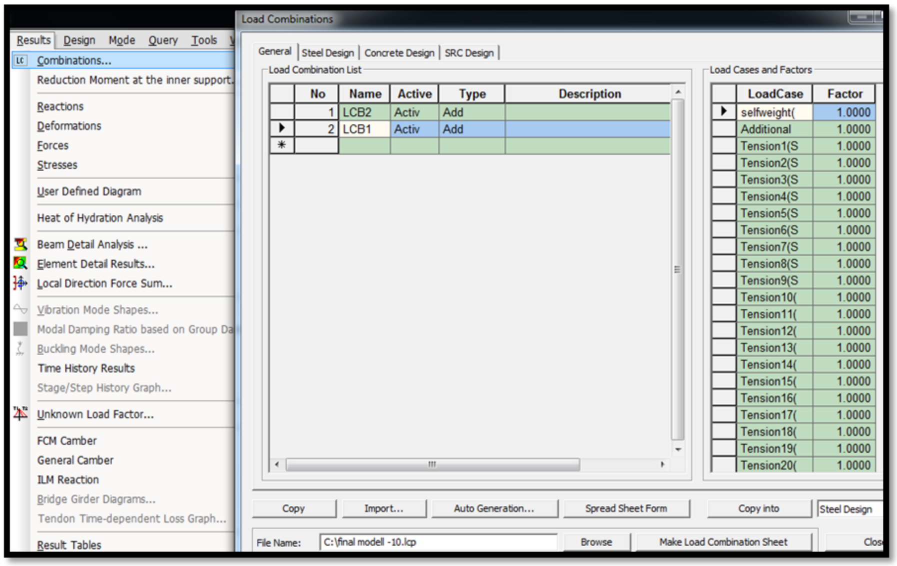


Figure B-15: load combination

- **Step (9) Unknown Load Factors Calculation**

- Results / Unknown Load Factor
- Unknown Load Factor Group>
- Item Name (Unknown); Load Comb >LCB 1
- Object function type>Square; Sign of unknowns>both
- L Case >Self-Weight (off)
- L Case >Additional Load (off)

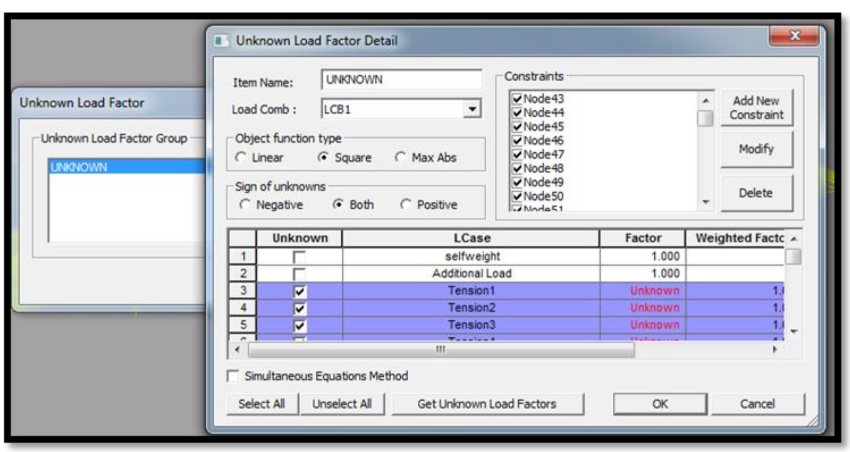


Figure B-16: Unknown load factor

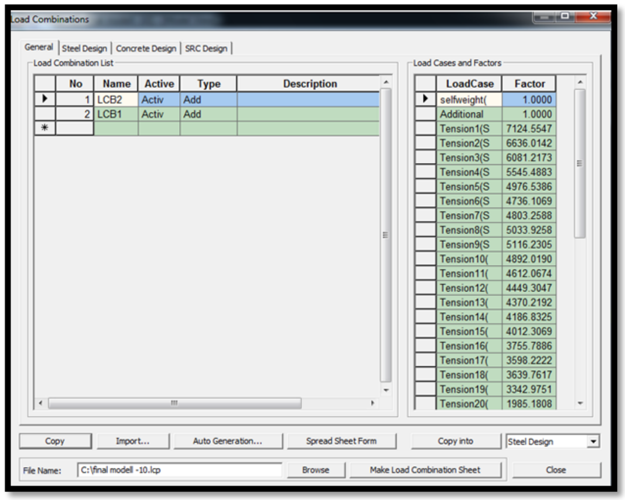


Figure B-17: load combination with unknown load factor

- ➢ **Moving load analysis:**

- ➢ **Step (1): Select the moving load analysis data**

- Selected the Moving Load Analysis > Moving Load Code in the Menu tab
- Selected the "AASHTO LRFD" in Selected Moving Load Code dialog box.
- Click Ok

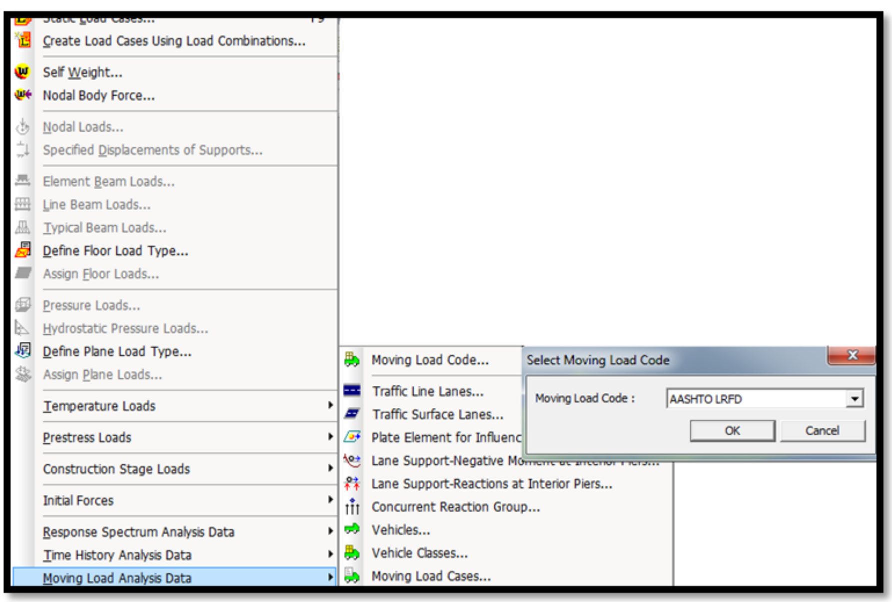


Figure B-18: moving load analysis data

- **Step(2): Define standard vehicle load**

- Selected the Moving Load Analysis>Vehicles in the Menu tab of the Tree Menu.
- Click **Add standard** in the Vehicles Load Type dialog box.
- Selected the "AASHTO LRFD Load" in the Standard Name field.
- Confirmed "HL-93 TRK" in Vehicle Load Name & Vehicle Load Type fields.
- Entered the "33" in the Dynamic Load Allowance field.
- Click **OK**
- Click **Add standard** in the Vehicles Load Type dialog box.
- Selected "AASHTO LRFD Load" in the Standard Name field.
- Confirmed "HL-93 TDM" in Vehicle Load Name & Vehicle Load Type fields.
- Entered "33" in the Dynamic Load Allowance field.
- Click **OK**
- Click **Close**

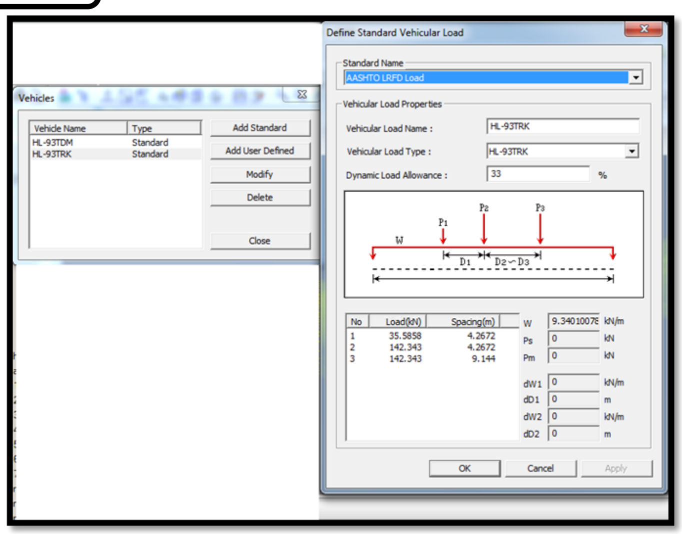


Figure B-19: define standard vehicle load

- **Step (3): Define moving load case**

- Selected the Moving Load Analysis >Moving Load Cases in the Tree Menu.
- Click Add in the Moving Load Cases dialog box.
- Entered “MVL” in the Load Case Name field of the Moving Load Case dialog box.

- Keep the default values of “Scale Factor” in Multiple Presence Factor.
- Click Add in the Sub-Load Cases field.
- Selected “VL:HL93-TDM” in the Vehicle Class field.
- Entered “1” in the Scale Factor field.
- Entered “1” in the Min. Number of Loaded Lanes field.
- Entered “2” in the Max. Number of Loaded Lanes field.
- Selected the “Lane 1” and “Lane 4” in List of Lanes of Assignment Lanes and Click ➡ to move to Selected Lanes
- Click Add in the Sub-Load Cases dialog box.
- Click Ok in the Sub-Load Cases field.
- Selected the “VL:HL93-TRK” in the Vehicle Class field.
- Entered “1” in the Scale Factor field.
- Entered “1” in the Min. Number of Loaded Lanes field.
- Entered “2” in the Max. Number of Loaded Lanes field.
- Selected the “Lane 1” and “Lane 4” in List of Lanes of Assignment Lanes and Click ➡ to move to Selected Lanes
- .Click Ok in the Sub-Load Cases dialog box.
- Click Ok in the Define Moving Load Case dialog box.
- Click close in the Moving Load Cases dialog box.

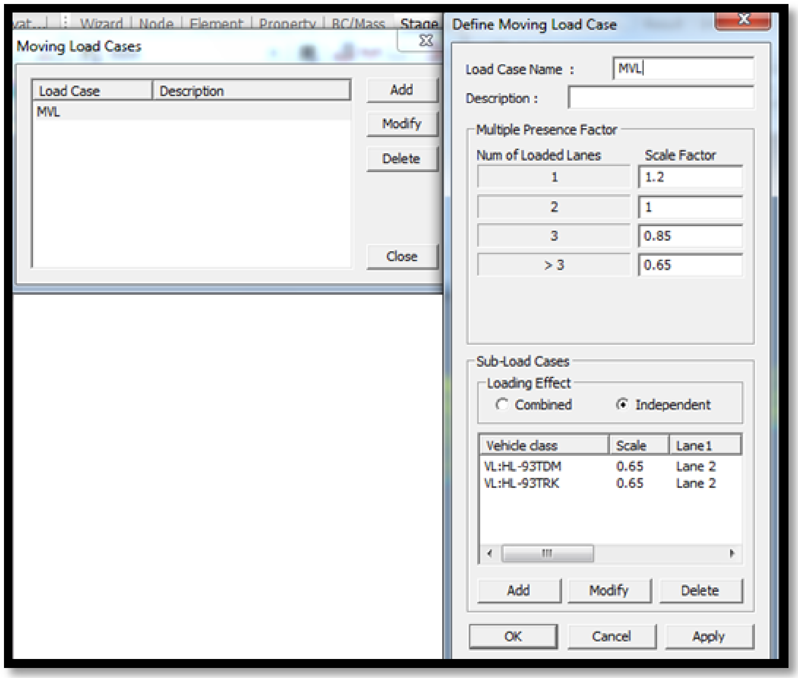


Figure B-20: Define moving load case

- **Step (4): moving load analysis control data**

- Selected Analysis >Moving Load Analysis Control from the Main Menu.
- Selected "Exact" in the Analysis Method field.
- Selected "All Points" in the Analysis Method field.
- Entered "5" in the Influence Generating Point No /Line Element field.
- Selected "Normal" in Frame in the Analysis Results field.
- Selected "All" in Reactions, Displacements and Forces/Moments under Calculation Filters.
- Click Ok .
- Click Node Number.

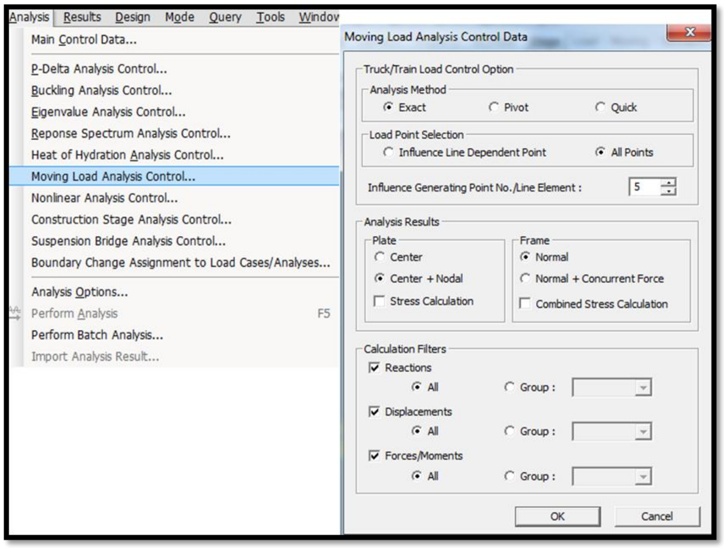


Figure B-21: moving load analysis

- **Step (5): Perform Structural Analysis is**

The structural analysis of the structure model with boundary Conditions and load cases is performed

- Analysis / Perform Analysis ↵

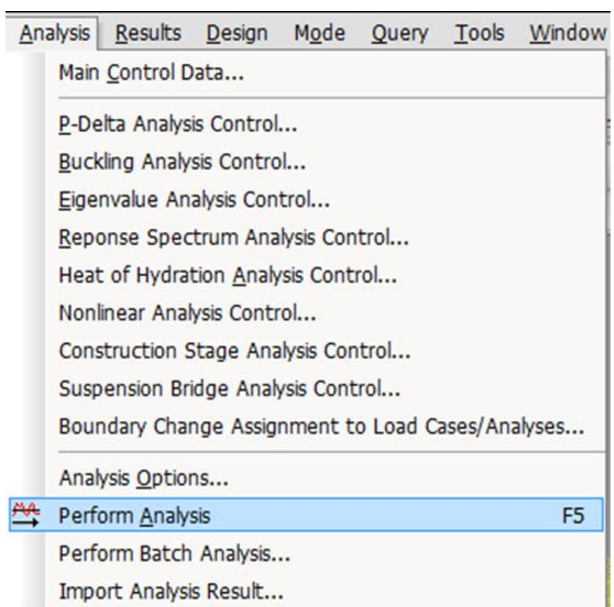


Figure B-22: Perform Structural Analysis

- Step (6): load combination

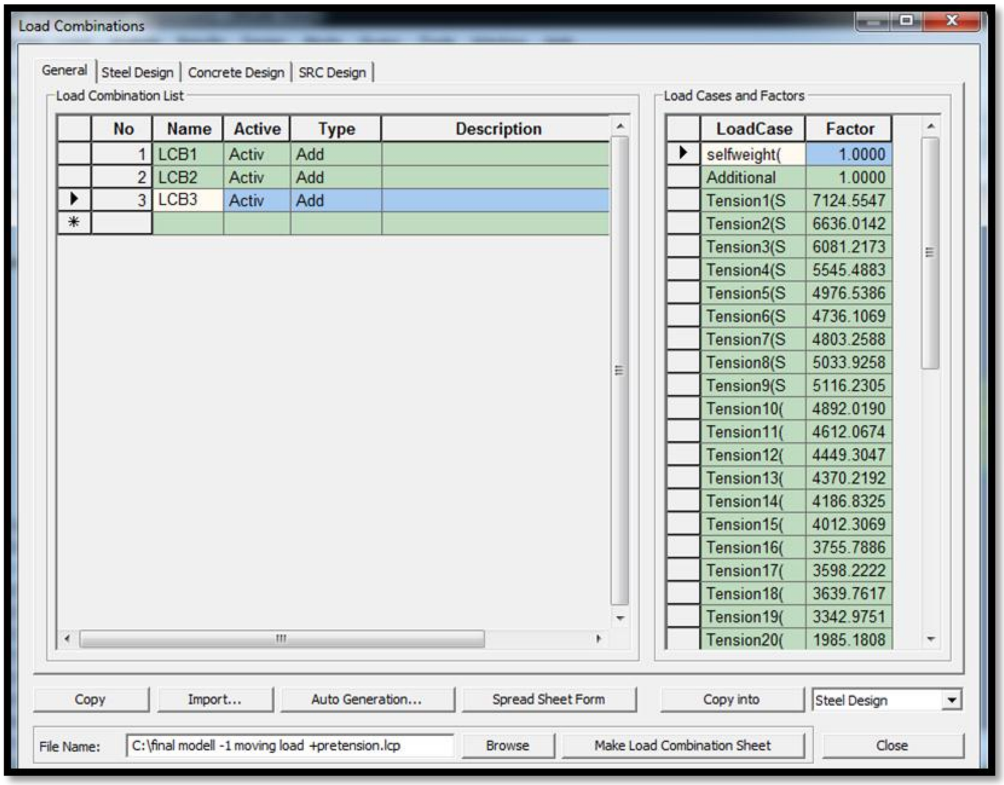


Figure B-23: combination load

- **Backward Construction Stage Analysis**
- ➢ **Step (1): generate a construction stage analytical model**

- File / Save As (Cable Stayed Backward Construction)

- ➢ **Step (2): Input Initial Cable Pre-tension**
- Results / Combinations
- Load Combination List >Name > LCB 1, LCB 2 Delete
- Load / Static Load Cases
- Name (Tension 1) ~ Name (Tension 40) Delete
- Name (Pretension); Type > User Defined Load ↵

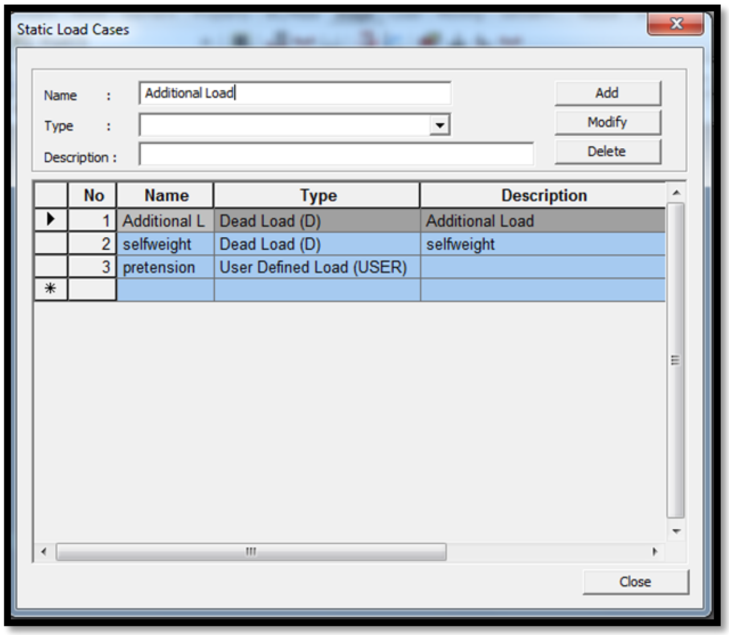


Figure B-24: Entering Initial Pretension Loading Condition

- **Step (3): Define Construction Stage**

- Load / Construction Stage Analysis Data / Construction Stage
- Define Construction Stage
- Stage >Name (CS); Suffix (0 to 62)
- Save Result > Stage (on) ↲

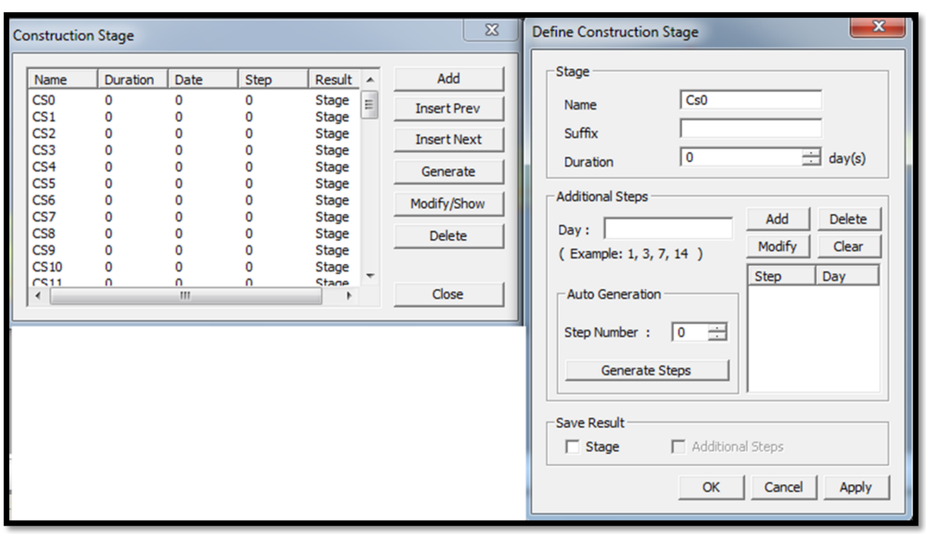


Figure B-25: define construction stage

➢ Step (4): Define Structure Group

- Group Tab
- Group>Structure Group >New… (right-click mouse)
- Name (SG); Suffix (0 to 62)

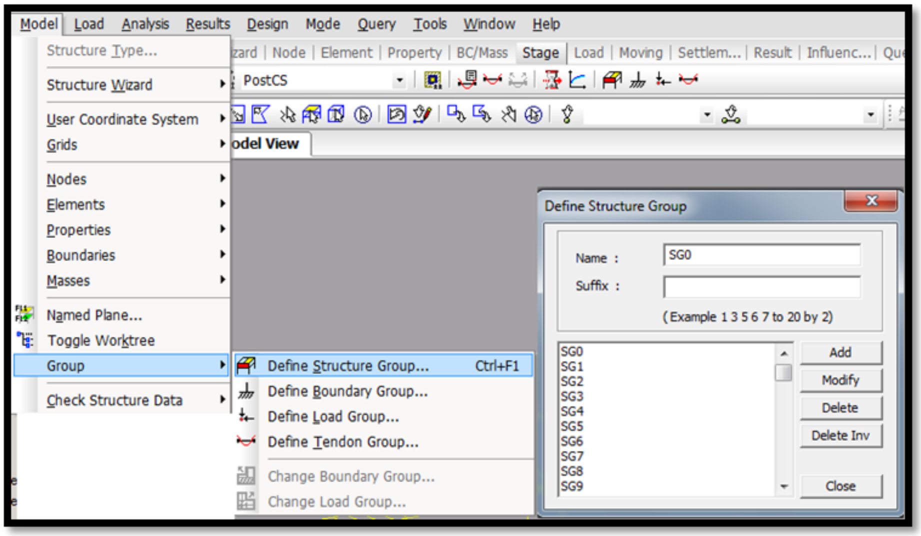


Figure B-26: define structure group

➢ Step (5): Define Boundary Group

- Group>Boundary Group>New… (right-click mouse)
- Name (Fixed Support) ↵
- Name (Elastic Link) ↵
- Name (Bent) ↵
- Name (Rigid Link) ↵

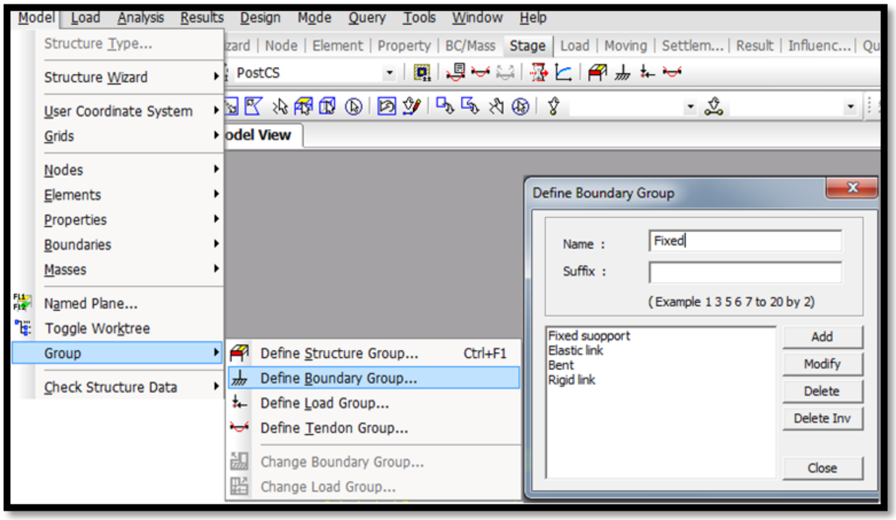


Figure B-27: define boundary group

➢ Step (6): Define Load Group

- Group >Load Group > New… (right-click mouse)
- Name (SelfWeight) ↵
- Name (Additional Load) ↵
- Name (Pretension Load) ↵

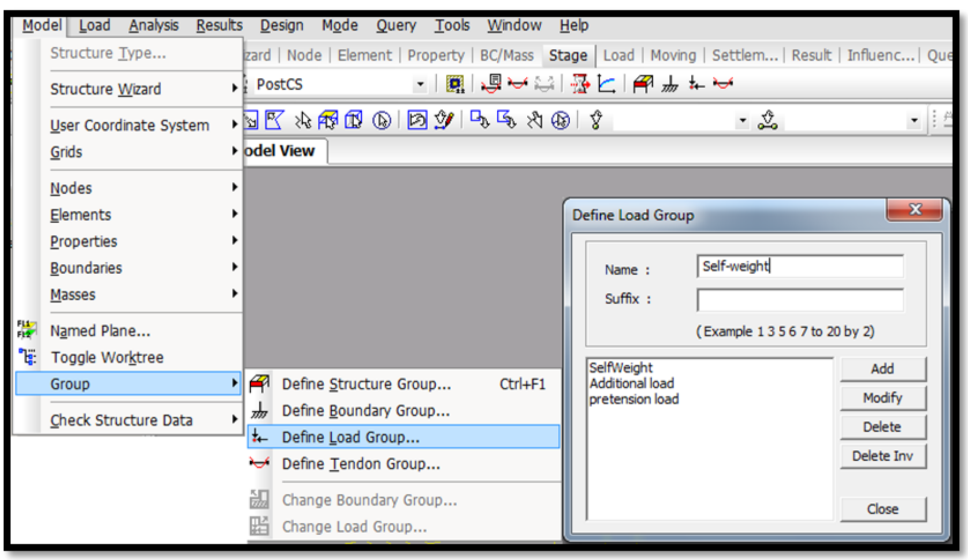


Figure B-28: define Load group

- **Step (7): Define Construction Stage**
- Load / Construction Stage Analysis Data / Define Construction Stage

  CS0 Modify/show
- Saved Result >Stage (on)
- Element tab >Group List > SG0; Activation> Add
- Boundary tab >Group List > Fixed Support, Elastic Link, Rigid Link
- Support / Spring Position>Original
- Activation > Add
- Load tab> Group List>Self-Weight, Additional Load, Pretension

  Activation> ↲ Add

Define Construction Stage for each construction stage from CS1 to CS62 using Table 3-6 in chapter three, Analytical sequence of backward construction stage as follows::

CS1 Modifv/show
- Save Result >Stage (on)
- Load tab> Group List > Additional Load
- Deactivation> ↲

CS2 Modify/show
- Save Result>Stage (on)
- Element tab>Group List > SG2; Deactivation > Add
- Element Force Redistribution > 100%
- Boundary tab>Group List > Bent; Support / Spring Position>Original
- Activation> Add

CS3 to CS62 Modify/show
- Saved Result>Stage (on)

- Element tab>Group List > SG3 to SG62; Deactivation> Add
- Element Force Redistribution> 100%

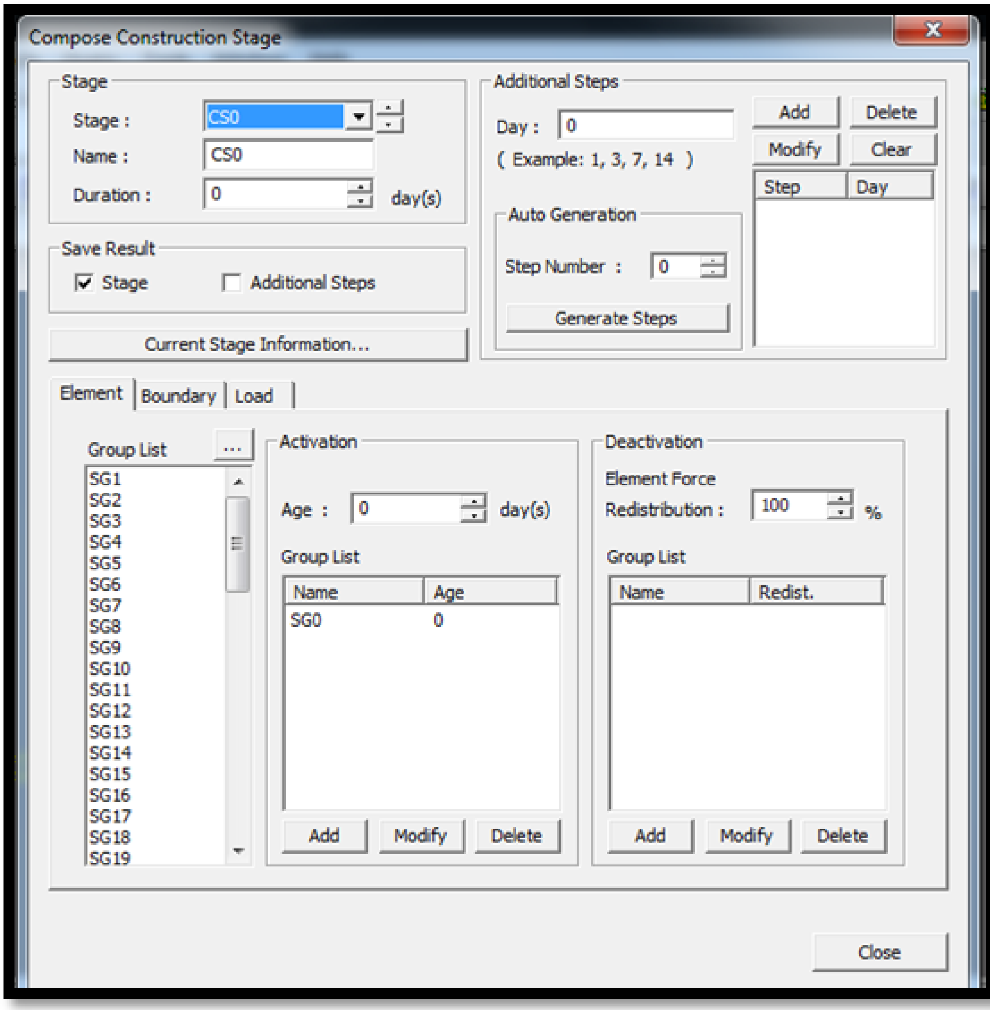


Figure B-29: compose construction stage

➢ **Step (8):Input Construction Stage Analysis Data**

- Analysis / Construction Stage Analysis Control
- Final Stage > Last Stage (on)
- Analysis Option>Include Time Dependent Effect (off) ↵

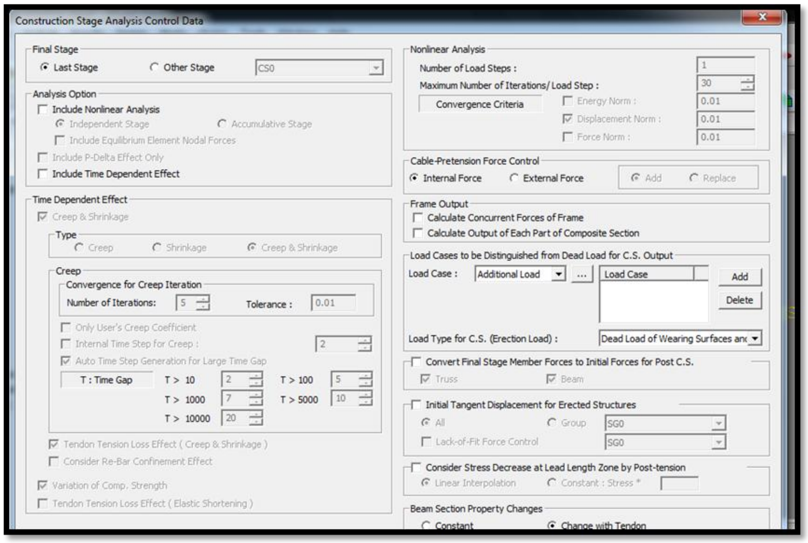


Figure B-30: construction stage analysis controls

- **Step(9):Perform Structural Analysis**
- Analysis / Perform Analysis ↵

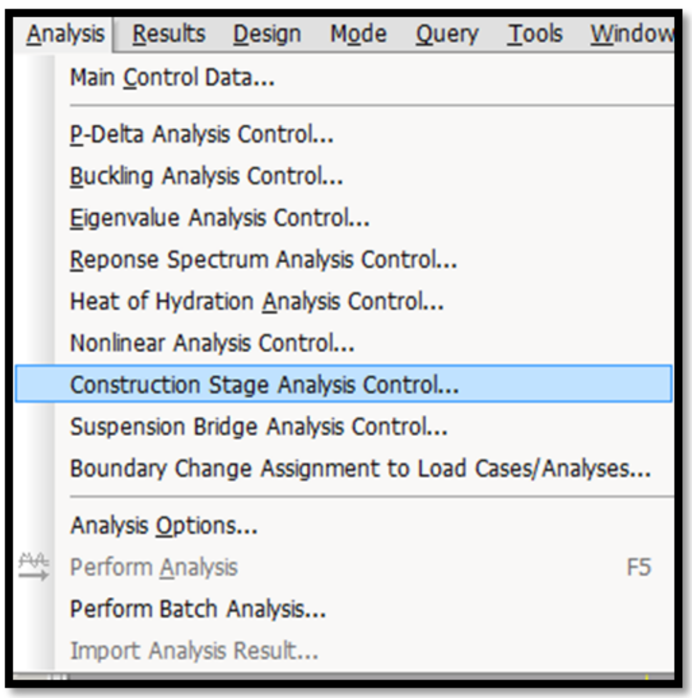


Figure B-31: construction stage analysis controls

- **Dynamic analysis:**
- **Step (1) structure type**
- Model /structure type /3D
- Lumped mass >convert to x, y,z
- Click Ok

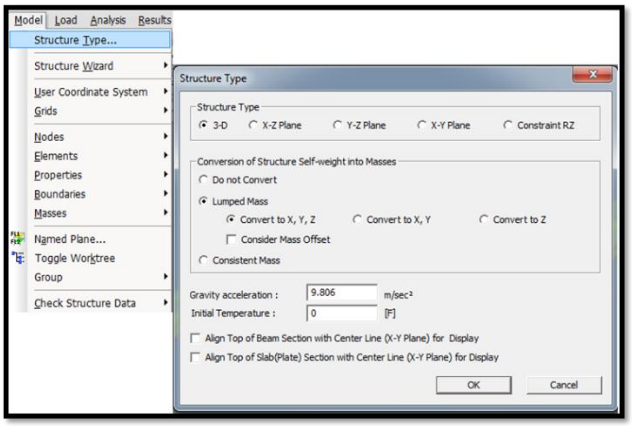
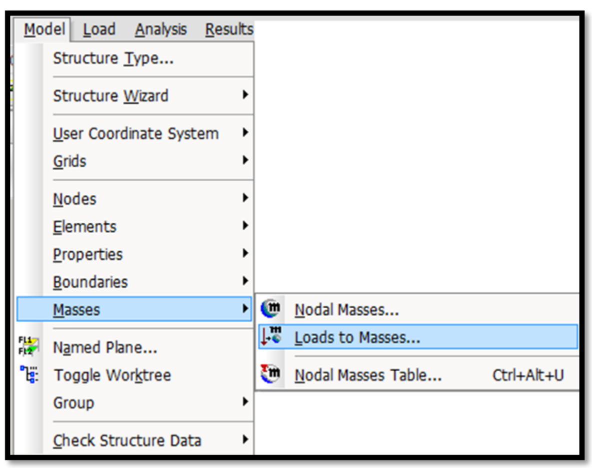


Figure B-32 structure type



FigureB-33: convert load to mass

- **Step 2 convert loads to mass**
- Model / mass / loads to masses
- Mass direction >x, y,z
- Load type for converting > Nodel load, Beam load, Floor load,

- Gravity >9.806 m/sec^2
- Click Ok

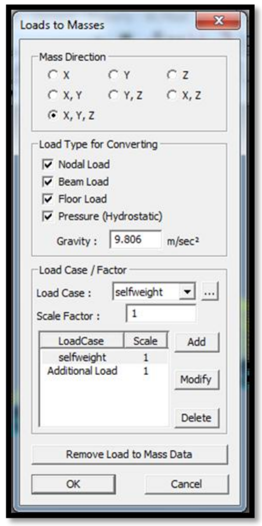


Figure B-34: convert load to mass

➢ Step (3) Eigenvalue analyses

- Analysis /Eigen value analysis control
- Type of analysis > Ritz vector
- Starting load vectors>load case >self-weigth Add
- Click OK

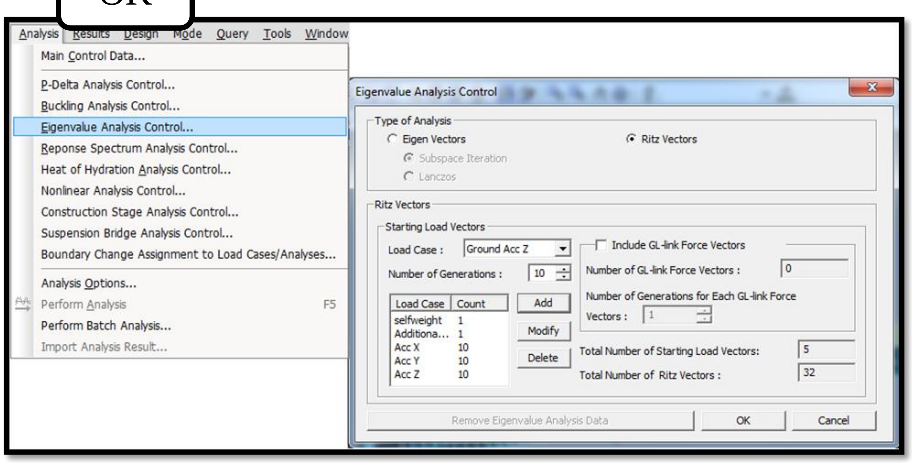


Figure B-35: Eigenvalue analysis control

APPENDIX(C) Analysis Result:

This Appendix contains the obtained result for chapter five.

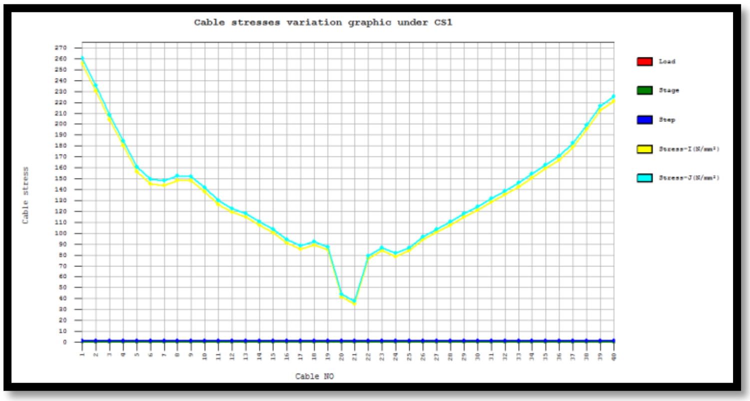


Figure C-1: Cable stress variation graphic under BCS1

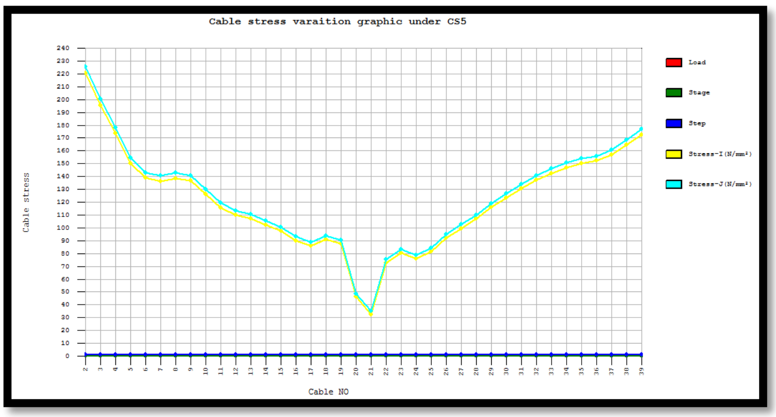


Figure C-2: Cable stress variation graphic under BCS5

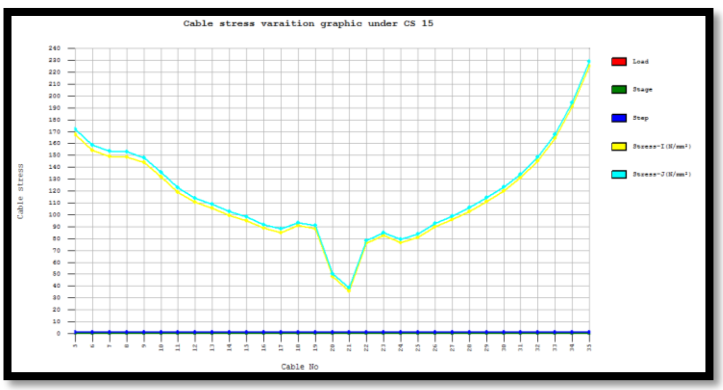


Figure C-3: Cable stress variation graphic under BCS15

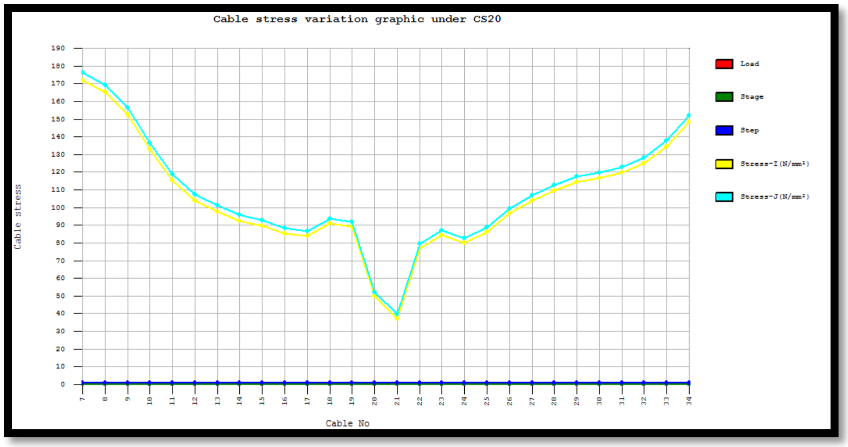


Figure C- 4: Cable stress variation graphic under BCS20

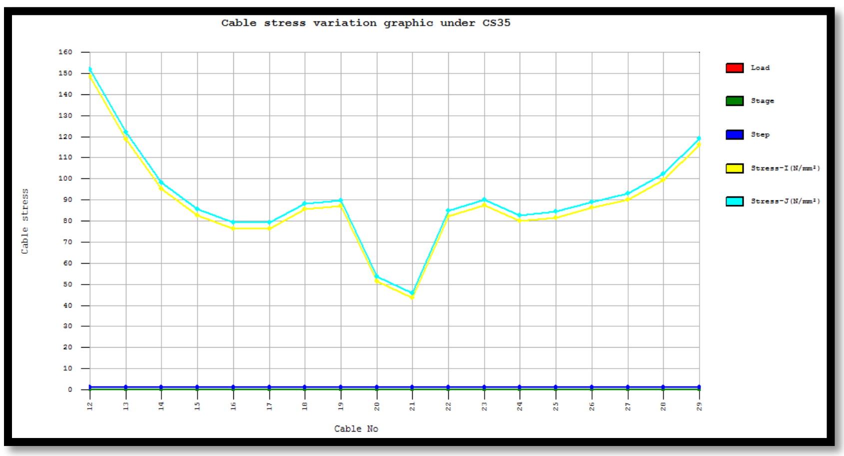


Figure C-5: Cable stress variation graphic under BCS35

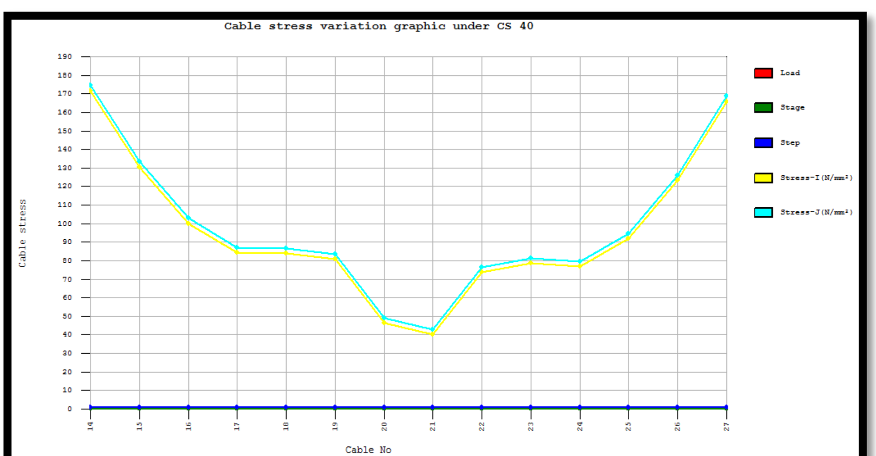


Figure C-6: Cable stress variation graphic under BCS40

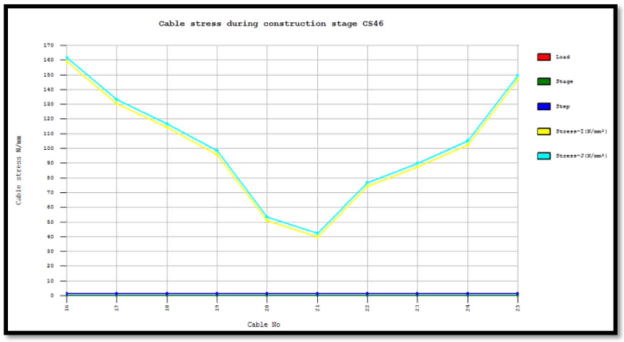


Figure C-7: Cable stress variation graphic under BCS46

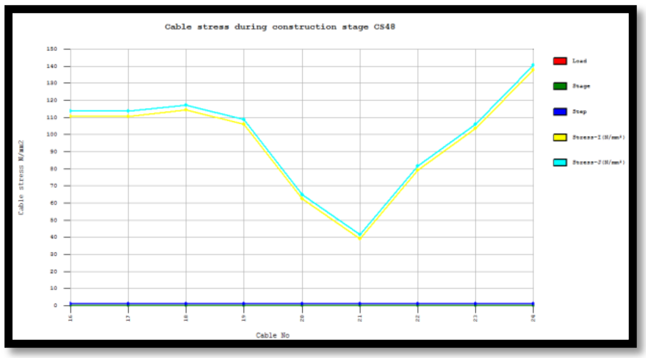


Figure C-8: Cable stress variation graphic under BCS48

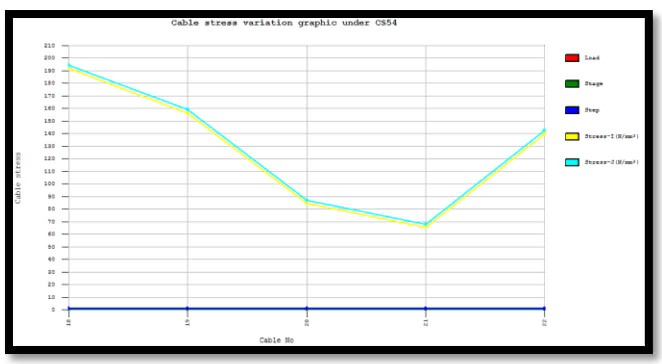


Figure C-9: Cable stress variation graphic under BCS54

Table C-1: Cable stress under BCS60

	Elem	Load	Stage	Step	Stress-I (N/mm^2)	Stress-J (N/mm^2)
▶	20	Summati	CS60	001(last)	3.688e+002	3.713e+002