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Evaluation of the Optimum Pre-Tensioning Forces for Cable Stayed Bridges

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***ABSTRACT* -** The Cable-stayed Bridge is one of the most modern bridges. The structural system of this type of bridge is effectively composed of cables, main girders and towers. Because of their complex structural system, Cable-stayed bridges are highly indeterminate structures that require a high degree of technology for analysis and design. Hence, they demand sophisticated structural techniques for analysis and design when compared with other types of conventional bridges. In such bridges the cables, being flexible supports, require pre-tensioning. These pre-tension forces are important factors in the design and construction process. Thus, the response of the bridge is highly non-linear and an optimization procedure is required to evaluate the pre-tensioning forces. In this study, the unknown load factor optimization method is the method used to determine the cable forces. The procedure is 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, and live load (moving load) according to AASHTO LRFD 2010 and using MIDAS Civil computer program. TUTI BAHARI cable stayed bridge of semi-fan type arrangement is analyzed for static load as a case study. The unknown load factor optimization method is used to determine the cable pre–tension forces to achieve a perfectly safe and stable bridge. The maximum cable forces (6670 kN), as well as the stresses (372 N/mm^2) and displacements at the top of tower (0.033308m), are found to be within the allowable limits. The results obtained illustrate that the unknown load factor optimization method leads to optimal structural performance for the cable stayed bridge. Hence it might be a useful tool for the analysis and design of such bridges.

Keyword: Moving Load**,** Dead Load, Additional Load, Flexible Supports, Pre-Tension, Optimization, Unknown Load Factor

المستخلص – يتألف النظام الإنشائي للجسور المدعمة بالكوابل من الكوابل والعارضة الرئيسية والابراج. هذا النوع من الجسوريتطلب درجة عالية من التقنية في التصميم والتحليل مقارنةً مع الانواع التقليدية الأخري من الجسور. وتعتبر قوة الكوابل عاملاً مهماً واساسياًفي العملية التصميمية للجسور المدعمة بالكوابل. ولأن الكوابل سواند مرنة فإنها تتطلب الشد المسبق. وبما أن للجسر درجة عالية من اللاخطية تستخدم عملية التحسين للحصول على قوى الشد المسبق, وطريقة التحسين لمعامل الحمل المجهول هي إحدى طرق الحصول على القوى فى الكوابل. تتناول هذه الدراسه تقييم قوه الشد المثلي للجسور المدعمة بالكوابل وذلك باستخدام برنامج الحاسب الاليMidas وهو أحد البرامج الهندسية المستخدمة في تحليل وتصميم الجسور.حيث تم التطبيق لجسر توتي – بحري الرابط بين جزيرةتوتي ومدينة بحري كحالة للدراسة . مع الأخذ في الاعتبار الحالات الحدودية والتغيير في التحميل حيث يعتمد البرنامج عليطريقةالتحسينلمعامل الحمل المجهول للوصول للحالة المثالية, حيث تم إيجاد قوى الشدالمثلى لكوابل الجسر التي تحقق الحدود المثالية للاستقرار وإجراء تحليل للاحمال الحيه للجسر وفقا للمدونه الامريكيه AASHTO LRFD2010. ووجد من خلال تحليل نموذج الجسر قيد الدراسةأن اقصي قوة للكوابل في الحدود المسموح بها والنتائج المتحصل عليها للازاحات والاجهادات مقبولة ويمكن أن تعتبر مرجعاً مناسباً لتصميم الجسور المماثلة.

INTRODUCTION

As stated by Xonthanko,[1], during the past decade cable–stayed bridges have been widely applied, especially in Western Europe, Canada, South America; Japan, Sweden and the United states. According to Vikas et al,[2] 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 by a great extent, aesthetical appearance and architectural considerations. Chen and Duan.[3] had posited that a cable-stayed bridge is a more economical solution for spans up to about 1000 m. Also, Vikas et al noted that this bridge form has a fine-looking appearance and fits in with most surrounding environments.

Toritsky,[4] and Vikas et al stated that the main structural elements of a cable stayed bridges 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 yielding 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 and both the pylon and the girder under compression. The members under axially loads are more efficient than flexural members.

Toritsky, Xonthanko and Scalzi,[5] had shown that 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. In the fan system the stay cable arrangement represents a modification of the harp systems, all ropes have fixed connections in the tower, whereas parallel stay cables are used in the harp system. The radiating type, or a converging system, is an arrangement where-in the cables intersect or meet at a common point at the top of the tower.

In the star arrangement, the star pattern used is an aesthetically attractive cable arrangement. However, it contradicts the principle that the points of attachment of the cables should be distributed as much as possible along the main girder. 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, economy and the cost.

Neils and Georgakis,[6] claimed that cable layout is a fundamental issue that concerns cable stayed bridges. As also, stated by Chen and Duan it not only affects the structural performance of the bridge, but also the method of erection and the economics. While, Scalzi argued that 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.

As highlighted by Barker and Puckett,[7], 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.

The permanent loads should be taken as the actual loads. These loads include the self-weight of the girders and deck, wearing surface, curbs, parapets and railings, utilities, luminaries, and pressures from earth retainment’s. 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. 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, and finally, the superstructure supports may move, inducing forces in the statically indeterminate bridge. Each type of load is presented individually with the appropriate reference to the AASHTO specification. One of the important sides in the design of a cable- stayed bridge is the determination of the optimum tensioning forces in the cables, which is directly related to forces in the tower and girder. Control of the cable tension force is critical. The pre-tension of the cables must be known because it changes the stresses in the girder and tower.

In recent years, the construction of cable-stayed bridges has been developing and rapidly increasing all over the world. Elmek Nimir Bridge and the proposed TUTI BAHARI cable stayed bridge are example bridges in Sudan.

LITERATURE REVIEW

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

Patel et al, (2017),[8] studied the steel box girder of the new bridge system under load combination of dead load secondary dead load and moving load by using the Indian Standards, Also, the check of the effect of reducing horizontal pressure on various structural elements was studied in depth.

Babu and Prasad, (2017),[9] in their paper reviewed the various wind effects and the different vibrations which are induced due to the wind on cable-stayed bridges.

Hararwalal and Maaru, (2016),[10] 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.

Chengfeng et al, (2015),[11] 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, element birth and death function, and multi-frame restart function.

Rageh and Maslennikov, (2013),[12] presented in their paper a study of cable-stayed bridges having three spans with double plane of cables. Three types of bridge arrangement were considered - 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.

Jani and Amin, (2017) [13] 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 with a proper technique in SAP2000. 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.

Garg and Chaturvedi, (2019),[14] 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 a 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.

Vikas et al, (2013),[2] analyzed a cable stayed bridge of fan type arrangement for static and dynamic load by using finite element method software MIDAS Civil. The bridge was analyzed under moving load case by using the IRC 6-2000 and earthquake load (Time History analysis of El Centro) and for different load combinations. They, then, studied the effect of the axial forces in cable, deck deflection, natural frequency, mode shape of the structure and earthquake response of the Cable Stayed bridge.

Chen, (2000),[15] 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. This method only works on the equilibrium force, when defining the initial cable forces nonlinearities were not considered. Because of the three modeling stages of the analysis this approach is more time-consuming than the other methods [16].

In this paper computer program MIDAS Civil is used to model and analyze TUTI BAHARI cable stayed Bridge. The displacement and stress in the towers and main girder are minimized by the chosen cable forces. Optimization methods are applied to minimize the internal forces in the calculation of the most ideal cable forces. The calculation considers user define restrictions for forces or members, displacements.

Modeling of Cable - Stayed Bridge
Description of the Bridge

The TUTI BAHARI cable - stayed Bridge proposed over the Nile River on one side of TUTI Island is a three-span unsymmetrical bridge. The total length of bridge is 600m (150m + 300m + 150m). The concreted deck (24m wide) is made of reinforced concrete slab (350mm depth) with pre-stressed cross beams (1.6x0.6m), and longitudinal concrete girders (2.2x2m). The deck of each cable-stayed cantilever section is supported by a total of 40 cables, with the 20 cables arranged in a semi-fan arrangement on each side of the tower, in two planes, on either side of the bridge deck. Each reinforced concrete pylon comprises two towers (H= 73.8m) and two cross beams (3x6) m. Explicated in Figure 3, the lower one supporting the deck. Figure 1 shows an elevation view of the bridge. A photograph of the completed TUTI BAHARI cable-stayed bridge is shown in Figure 2.

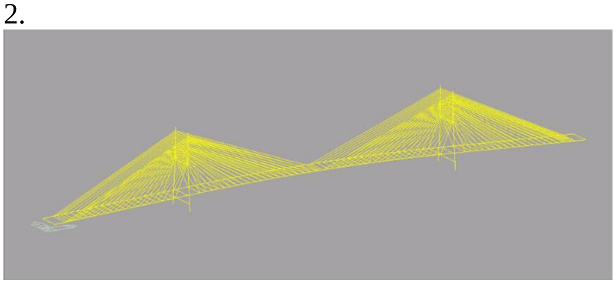

Figure 1: TUTI BAHARI bridge elevation view

Figure 2: TUTI BAHARI cable stayed bridge

Finite Element Method & Modeling
The most important part in the analysis is modeling the bridge. A three-dimensional finite element model of TUTI BAHARI bridges is developed and analyzed using the FEM software MIDAS Civil. The cable- stayed bridge components like deck, pylon, cables must be modeled as per the actual forces they are subjected to. The cables are modeled as truss elements (160 elements), The pylon and deck are modeled as elastic beam elements (356 elements). The bridge is first analyzed for the dead loads by static analysis to get the deformed configuration [17].

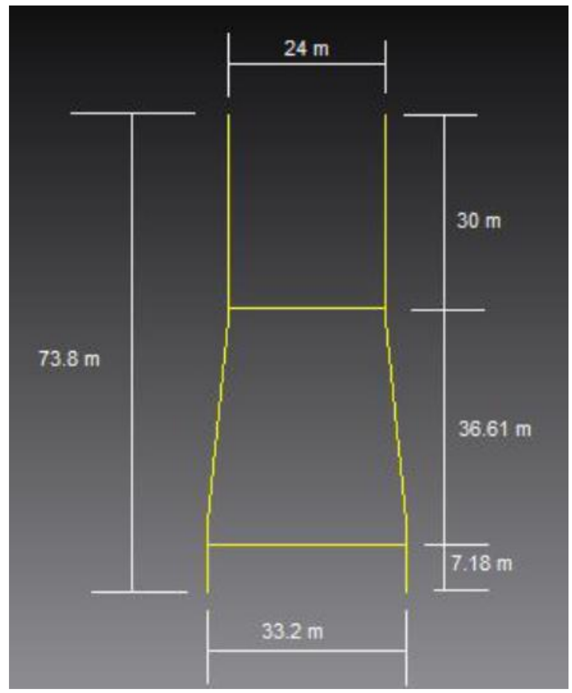

Figure 3 Tower dimensions

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 required modeling data to be used for the calculations of TUTI BAHARI model is presented in tables. A 2D&3D Cable stayed model is chosen to clarify main considerations in modeling and to determine the cable forces. The structure is modeled in MIDAS software using the data considerations shown in Table 1 to Table 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.
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 4 shows the steps that are carried out to generate the unknown load factors.

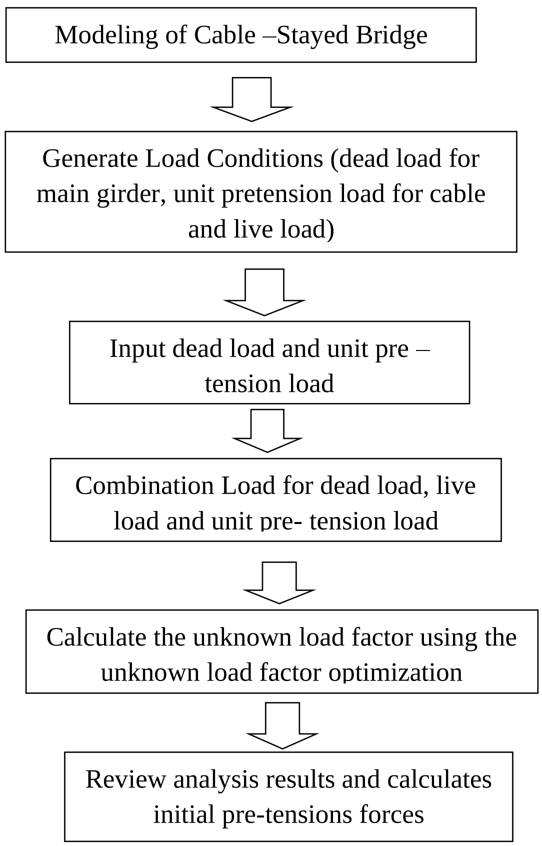

Figure 4 Flowchart for Initial Cable Pre-tension Calculation

Moving Load Condition
Moving load analysis in this paper is performed by using AASHTO LRFD 2010section 3.6.1.2. The vehicles are generated and applied in the existing lanes following the guidelines from AASHTO LRFD 2010. Moving load generation in MIDAS civil is based on: (a) Traffic line lanes (b) Vehicle load (c) Moving load application. The vehicles are applied to the lanes using the vehicle classes.

Vehicular Live Loads
According to AASHTO the Vehicular live loading on the roadways of bridges, is designated HL-93, and shall consist of a combination of the:
• Design truck or design tandem, and
• Design lane

Load Combinations
In this study, the cable stayed bridge is evaluated under the effect of Dead load (Self weight, additional loads), Initial pre-tensioning cable force, and live load (moving load). During the study, moving loads on cable-stayed bridges are taken as proposed in AASHTOO LRFD 2010 SPECIFICATION. The deck is divided in four lanes according to AASHTOO LRFD 2010 requirements. The following load combinations are used when evaluating cable pre- tensioning.
LCB1= Dead loads (self-weight + additional load) +Unit 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).

RESULTS AND DISCUSSION
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. Since, the cable stayed bridges are very long and highly indeterminate structures, with geometrically nonlinear characteristics that are reflected in the nonlinear load deflection behavior under any loading conditions.

Cables Pre-tensioning Force
The unknown load factors optimizations result for Cables 1, 2 and 3 are 7124.55, 6636 and 6081.27 respectively. Figure5 illustrates the result table given by MIDAS for unknown load factor optimization. Also, the resulting of pre-tensioning forces distribution including the factors for the tension forces in the cable stays 1 to 40 on the existing partial model, are as shown in Figures 6 and 7 and Table 5. The maximum cable force under LCB2 is 6670 kN at the beginning cable of the main girder, which is within the allowable range of the tension strength limit of the tendon.

The result of the maximum cable force under LCB3 is 7229 KN at the cable tension (1) on the existing partial structure. For the TUTI BAHARI cable-stayed bridge, the results are as shown in Figures 8 and 9 and Table 6.

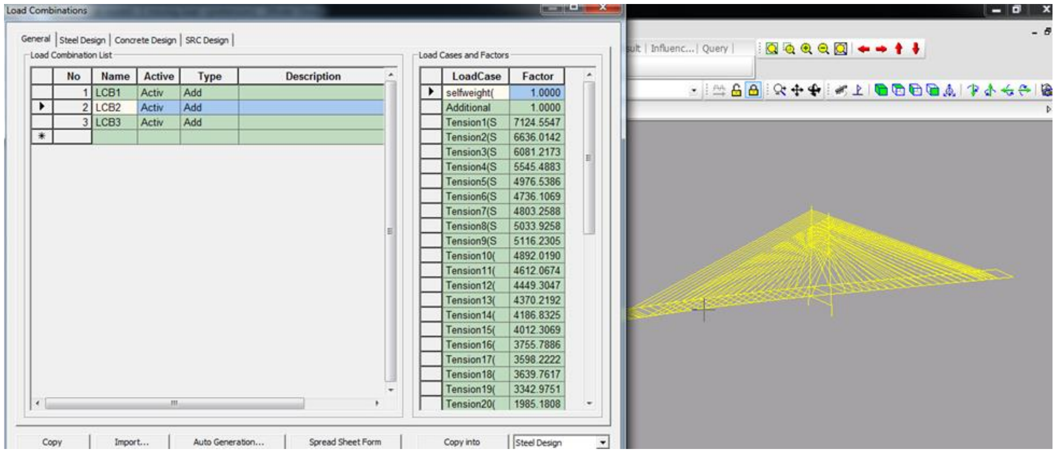

Figure 5: Result of unknown load factors

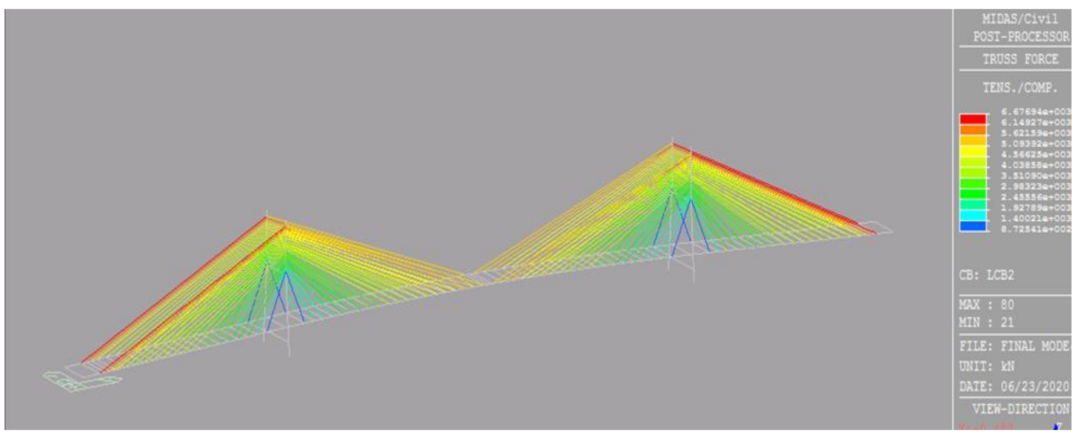

Figure 6: Pre-tension force in the cable stays for LCB2

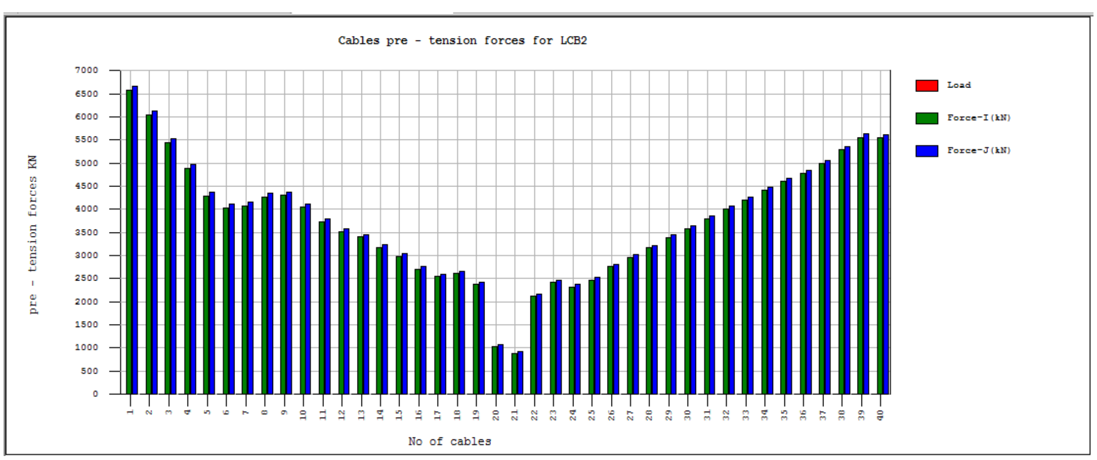

Figure 7 Cable pre-tension Force Variation Graph for LCB2

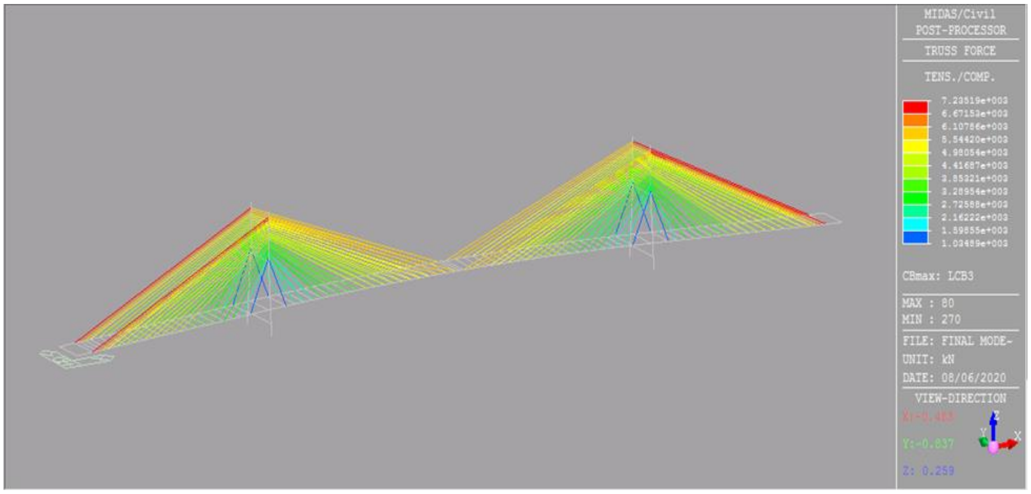

Figure 8: Pre-tension forces in the cable stays for LCB3

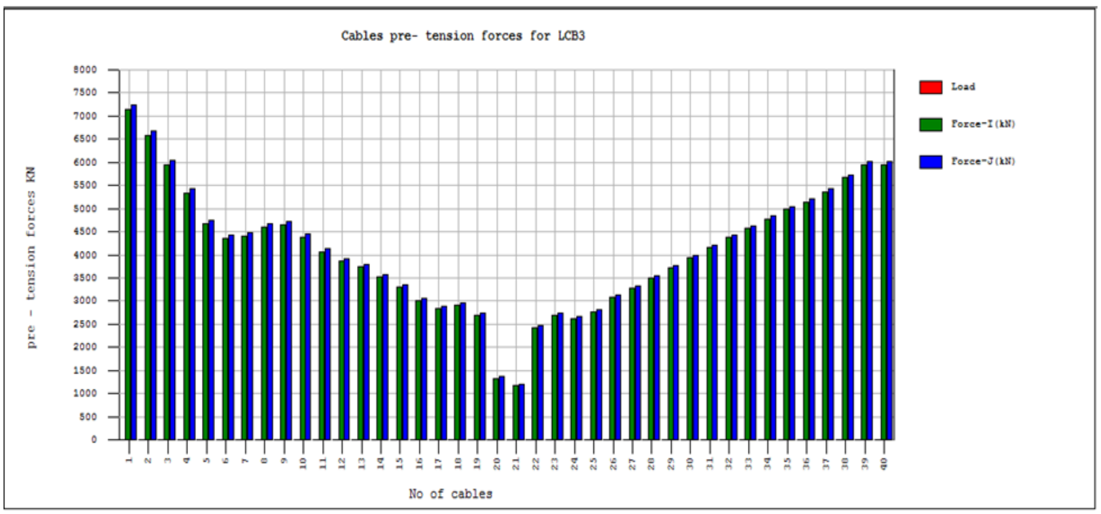

Figure 9: Cable pre-tension Forces Variation Graph for LCB3

Displacement Results

Figures 10 and 11 show the deformed shape of two pylons and main girder including the maximum deflection values. Displacements during LCB2&LCB3 contain the evolution of these displacements according to the stages considered in the analysis.

The maximum values of the horizontal displacements at the top of the Pylons under LCB2, LCB3 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 δ_max= 66000/300= 220mm.

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) in the longitudinal direction.

Stress Results

Figures 12 and 13 indicate the cable stress in N/mm2 for individual cable profile under LCB2&LCB3 respectively. Figures 14 and 15 show graphically the evolution of these stresses according to the stages considered into the analysis and it is indicated that cable stress is maximum in the two long stay cables with maximum pre-tensions of (372 N/mm2) and (410 N/mm2).

The allowable stress under dead load + secondary dead load + live load is 837 N/mm2 (AASHTO-LRFD).

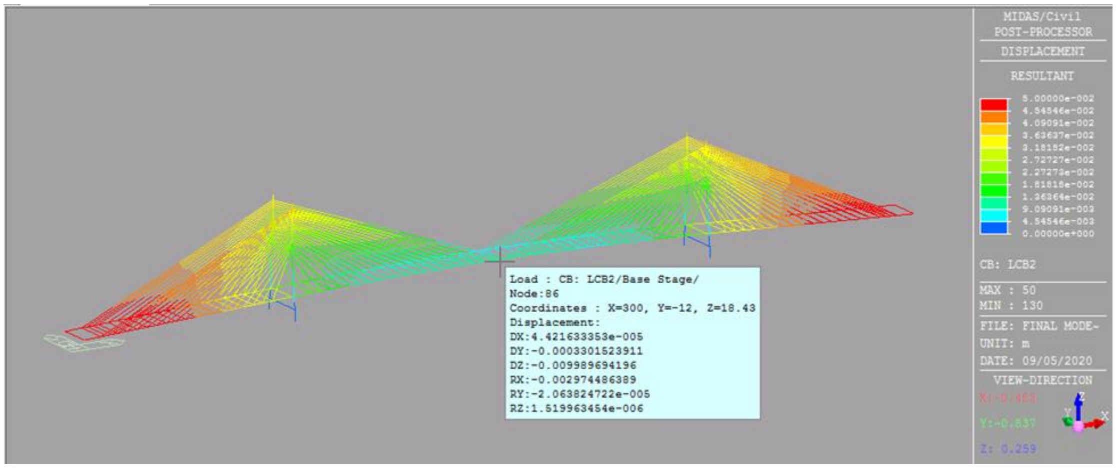

Figure 10: Displacement for cable stayed bridge under LCB2

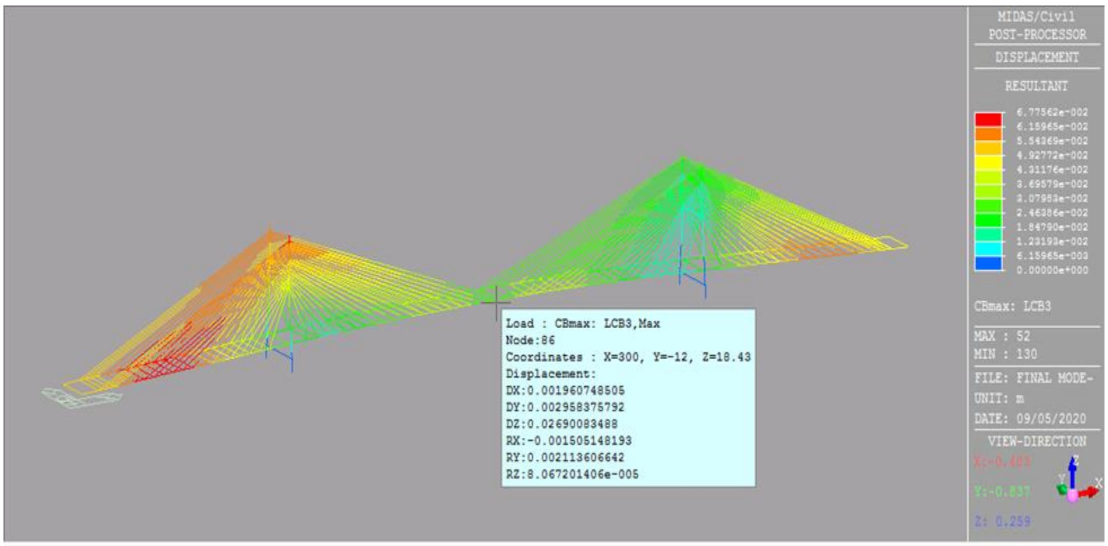

Figure 11: Displacement under LCB3

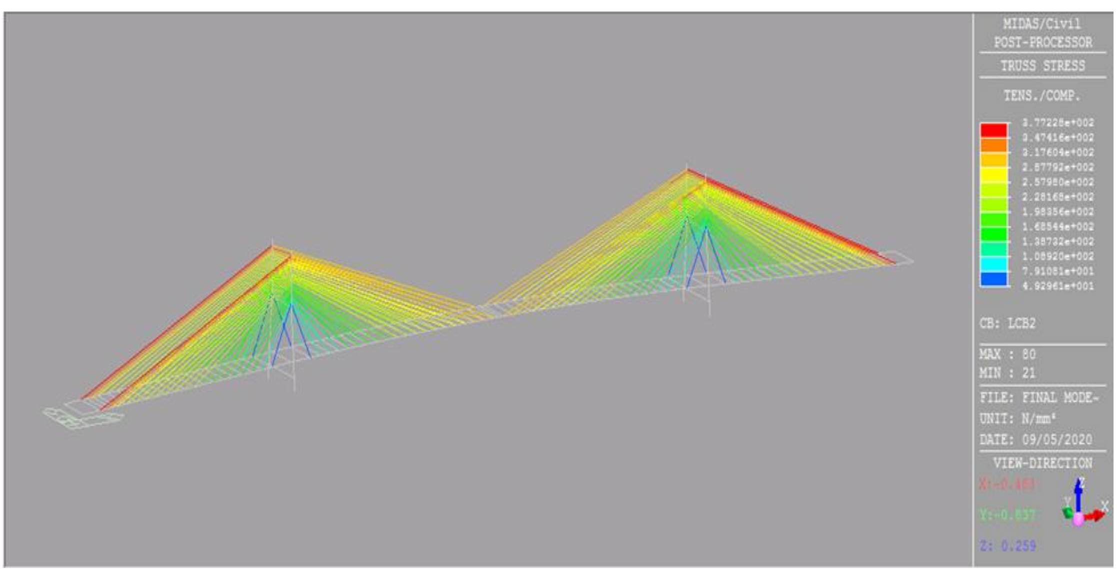

Figure 12: Cables stress under LCB2

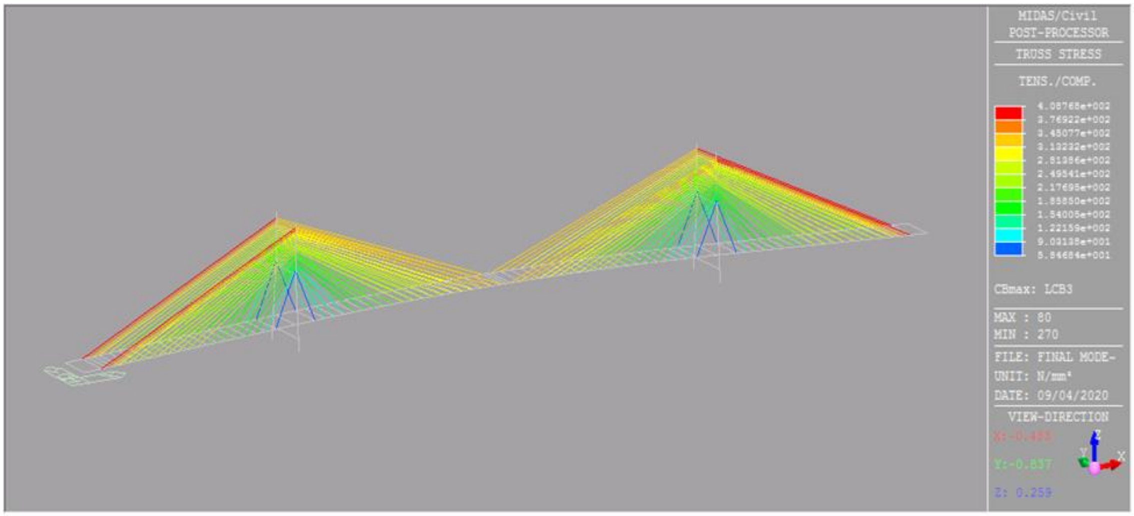

Figure 13: Cables stress under LCB3

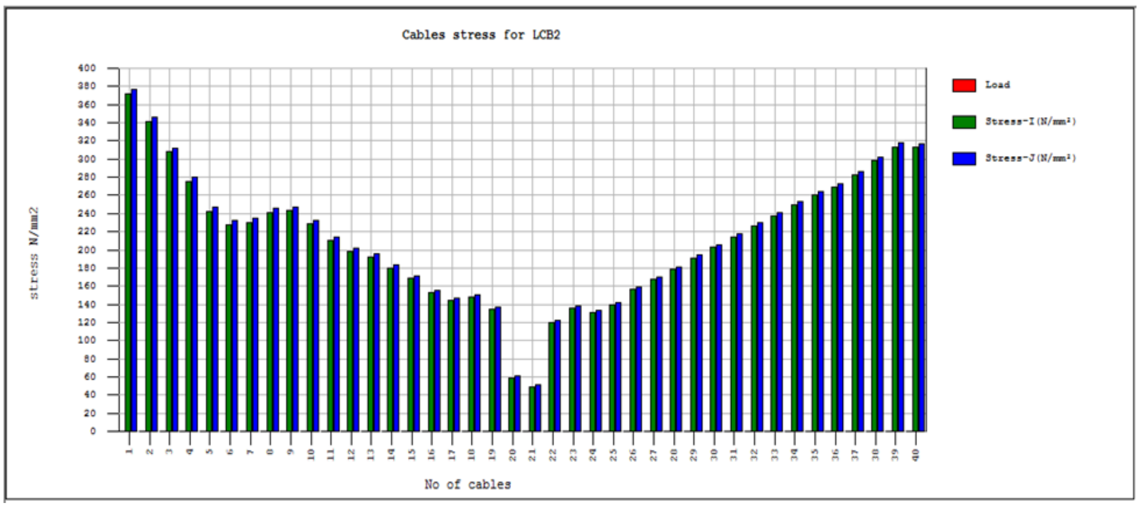

Figure 14: Cables stress variation graphic under LCB2

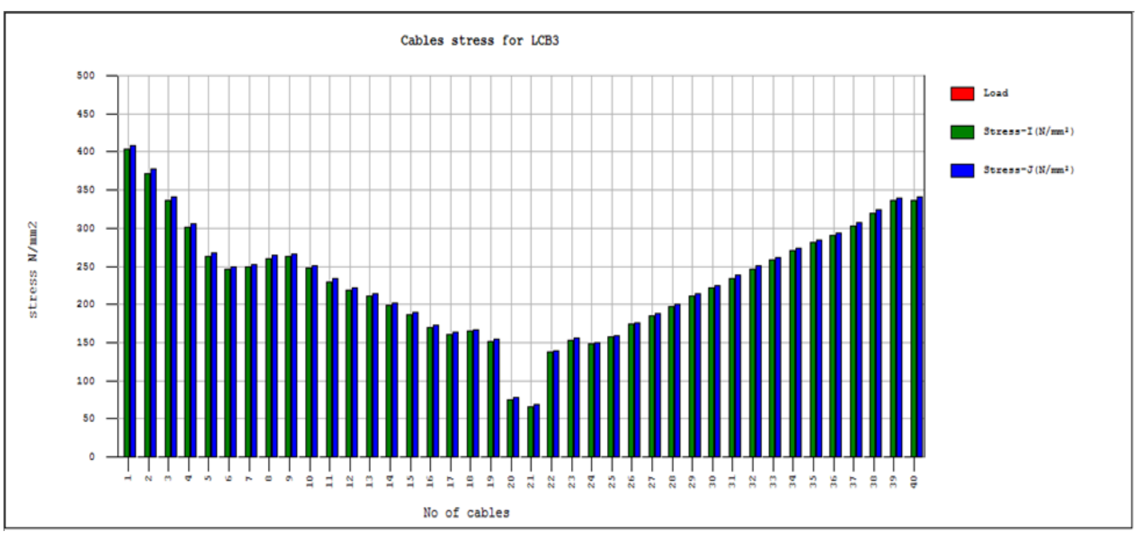

Figure 15: Cables stress variation graphic under LCB3

CONCLUSIONS

A finite element methodology is presented for the model analysis of the TUTI BAHARI cable-stayed bridge. The model analysis has been used to determine the pre-tension force in the cable under different loads conditions. The FEM analysis program MIDAS Civil has been applied in the model of the analysis process.

The ideal state of the structure system has been developed by an appropriate cable pre-tensioning; wherein unknown load factors are applied in the analysis. With the restriction of the moment and vertical displacement, a continuous beam condition for the main girder has been achieved.

The ideal cable pre-tension forces have been determined to achieve an optimal structural performance due to its permanent loads.

The maximum displacement in top of the tower (0.033308m),(0.060295m), and main girder (0.001m) ,(0.026m) under dead load, live load stages respectively, are controlled and are within the allowable range(220mm).

The maximum stress in cable (372 N/mm2) and (410 N/mm2) under load conditions has occurred in the cable with the greatest pre tension force. These stresses are within the allowable range (837 N/mm2).

Acknowledgment

This paper is part of an ongoing study on analysis and design of cable stayed bridges including construction stages of the bridges. The results obtained from the pre–tension forces simulation analysis will serve as the initial data for the main research. The results of this paper can provide reference for the Backward Construction analysis of the TUTI BAHARI cable stayed bridge.

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TABLE 1: MATERIAL PROPERTIES

For Stay Cable Steel	For Concrete
Modulus of Elasticity = 197 GPa	Modulus of Elasticity = 2.76x107 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 2: LOADING DATA OF THE MODEL

Classification	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 LRFD)	

TABLE 3: BOUNDARIES CONDITIONS

Boundaries	No
Support (fixed, pinned, roller)	20
Elastic link	4
Rigid link	8

TABLE 4: SECTION PROPERTIES

No	Name	Area m^2	I_{xx} m^4	I_{yy} m^4	I_{zz} m^4
1	Cable	0.0177	0	0	0
2	Long girder	4.4	2.698	1.775	1.467
3	Transverse 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 5: OPTIMIZED PRETENSION LOAD

Cable Name	Load Combinations	Optimized Pretension Load Kn
Cable Tension 1	LCB2	6670.924041
Cable Tension 2	LCB2	6133.455631
Cable Tension 3	LCB2	5531.530848
Cable Tension 4	LCB2	4960.141608
Cable Tension 5	LCB2	4366.901929
Cable Tension 6	LCB2	4111.069787
Cable Tension 7	LCB2	4153.699496
Cable Tension 8	LCB2	4344.880506
Cable Tension 9	LCB2	4378.645481
Cable Tension 10	LCB2	4106.029361
Cable Tension 11	LCB2	3781.592797
Cable Tension 12	LCB2	3576.922482
Cable Tension 13	LCB2	3457.736178
Cable Tension 14	LCB2	3238.497308
Cable Tension 15	LCB2	3034.995205
Cable Tension 16	LCB2	2759.147190
Cable Tension 17	LCB2	2596.184180
Cable Tension 18	LCB2	2662.053959
Cable Tension 19	LCB2	2430.958754
Cable Tension 20	LCB2	1073.910331
Cable Tension 21	LCB2	914.853003
Cable Tension 22	LCB2	2173.403144
Cable Tension 23	LCB2	2457.236897
Cable Tension 24	LCB2	2368.680407
Cable Tension 25	LCB2	2519.996643
Cable Tension 26	LCB2	2813.879131
Cable Tension 27	LCB2	3015.172401
Cable Tension 28	LCB2	3212.806949
Cable Tension 29	LCB2	3442.335628
Cable Tension 30	LCB2	3640.376023
Cable Tension 31	LCB2	3855.259550
Cable Tension 32	LCB2	4068.631079
Cable Tension 33	LCB2	4268.056633
Cable Tension 34	LCB2	4476.635361
Cable Tension 35	LCB2	4678.992708
Cable Tension 36	LCB2	4836.211000
Cable Tension 37	LCB2	5061.540512
Cable Tension 38	LCB2	5356.495682
Cable Tension 39	LCB2	5623.689377
Cable Tension 40	LCB2	5611.981492

TABLE 6: OPTIMIZED PRETENSION LOAD FOR LCB3

Cable tension name	Load combinations	Force (kN)
Cable tension 1	LCB3	7229.991292
Cable tension 2	LCB3	6673.706319
Cable tension 3	LCB3	6036.925473
Cable tension 4	LCB3	5422.321576
Cable tension 5	LCB3	4746.903585
Cable tension 6	LCB3	4424.220350
Cable tension 7	LCB3	4473.974027
Cable tension 8	LCB3	4673.838100
Cable tension 9	LCB3	4713.045575
Cable tension 10	LCB3	4445.364392
Cable tension 11	LCB3	4128.926235
Cable tension 12	LCB3	3925.205357
Cable tension 13	LCB3	3799.564740
Cable tension 14	LCB3	3570.045027
Cable tension 15	LCB3	3355.695080
Cable tension 16	LCB3	3069.564159
Cable tension 17	LCB3	2897.644305
Cable tension 18	LCB3	2958.287959
Cable tension 19	LCB3	2728.860941
Cable tension 20	LCB3	1369.289362
Cable tension 21	LCB3	1209.809066
Cable tension 22	LCB3	2466.925425
Cable tension 23	LCB3	2747.890615
Cable tension 24	LCB3	2664.933595
Cable tension 25	LCB3	2823.859924
Cable tension 26	LCB3	3124.978694
Cable tension 27	LCB3	3333.482244
Cable tension 28	LCB3	3539.051106
Cable tension 29	LCB3	3777.375972
Cable tension 30	LCB3	3984.741585
Cable tension 31	LCB3	4208.594362
Cable tension 32	LCB3	4429.067642
Cable tension 33	LCB3	4632.017476
Cable tension 34	LCB3	4840.827298
Cable tension 35	LCB3	5044.103239
Cable tension 36	LCB3	5204.101000
Cable tension 37	LCB3	5433.341918
Cable tension 38	LCB3	5734.620526
Cable tension39	LCB3	6015.190314
Cable tension 40	LCB3	6028.155742