FATIGUE LIFE ASSESSMENT OF BOSPHORUS BRIDGE HANGER CABLES

by

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ABSTRACT

FATIGUE LIFE ASSESSMENT OF BOSPHORUS BRIDGE HANGER CABLES

The Bosphorus Bridge, constructed in 1973, is an iconic suspension bridge located in the heart of Istanbul, Turkey serving as an extremely important link of the city transportation network with an average daily traffic of around 190,000 vehicle/day. Recently, the bridge has undergone a major retrofit work which also included replacement of the entire suspender system by changing the inclined orientation of the hangers to vertical. The hanger cables of a suspension bridge are susceptible to fatigue failures under fluctuating traffic loads and wire breaks in cables is a common problem which may also result in replacement of them. This study aims to evaluate the fatigue performance of the newly installed hanger cables under traffic loads. For an existing bridge, it is a meaningful concern for bridge operators as well in terms of asset management and in order to meet the increasing traffic demands through out the years of service. A detailed 3D finite element model of the bridge is developed and the model is verified by using the ambient vibration measurements taken from the bridge. A realistic traffic time history loading scenario is generated by simulating the traffic flow. The resulting stress time histories of the hanger cables are obtained from the analytical model. Fatigue analysis and life prediction of hanger cables under variable amplitude stress history due to traffic loading is performed using a S-N curve based on test results. The factors contributing the damage level such as hanger length and vehicle types are discussed Also, the effect of corrosion on fatigue life is taken into account.

ÖZET

BOĞAZİÇİ KÖPRÜSÜ ASKI HALATLARININ YORULMA DEĞERLENDİRMESİ

Yapımı 1973 yılında tamamlanan Boğaziçi Köprüsü, günde 190,000 araca hizmet eden İstanbul şehrinin trafik ağında kilit öneme sahip ikonik bir asma köprüdür. Yakın zaman önce, köprü kapsamlı bir güçlendirme faaliyetine alınmış ve bu kapsamda çapraz olan bütün askı halat sistemi düşey halatlar ile değiştirilmiştir. Asma köprülerin askı halatları, dalgalanan trafik yükleri altında kaldığından yorulma riskine açıktır ve bu sebepten kaynaklı tel kırıkları oluşabilmektedir. Bu çalışmada, köprüye monte edilen yeni düşey askı halatlarının trafik yükleri altında yorulma performansı değerlendirilecektir. Bu, halihazırda hizmet eden bir köprü işletmecisi açısından da köprünün sürekli artan trafik yükleri altında güvenle hizmet edebilmesi açısından önemli bir endişedir. Bu amaçla köprünün detaylı bir sonlu elemanlar modeli oluşturulmuş ve bu model davranışı sahadan alınan dinamik ölçümler ile teyit edilmiştir. Köprüdeki trafik akışı gerçekçi bir senaryo ile modellenmiştir. Bu etki altnda zaman tanım alanında analiz yapılmış ve halatlardaki gerilme kayıtları elde edilmiştir. Askı halatlarının yorulma analizi ve ömür tayini, testler ile elde edilen S-N eğrileri kullanılarak yapılmıştır. Halatlarda oluşan hasar seviysine etke eden araç tipi, halat uzunluğu gibi faktörler tartışılmıştır. Ayrıca, halatta oluşabilecek olan korozyonun yorulma performansına etkisi de irdelenmiştir.

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LIST OF SYMBOLS

GPa	Gigapascal
Hz	Hertz
kN	Kilonewton
MPa	Megapascal
m	meter
t	tons

LIST OF ACRONYMS/ABBREVIATIONS

Asymm	Anti-symmetrical
CAD	Computer-Aided Drawing
CSI	Computers and Structures Incorporated
DAF	Dynamic Amplification Factor
FE	Finite Element
KGM	Turkish General Directorate of Highways
WIM	Weigh In Motion
S-N	Alternating stress versus the number of cycles to failure
Symm	Symmetrical
2D	Two Dimensional
3D	Three Dimensional

1. INTRODUCTION

1.1. Background

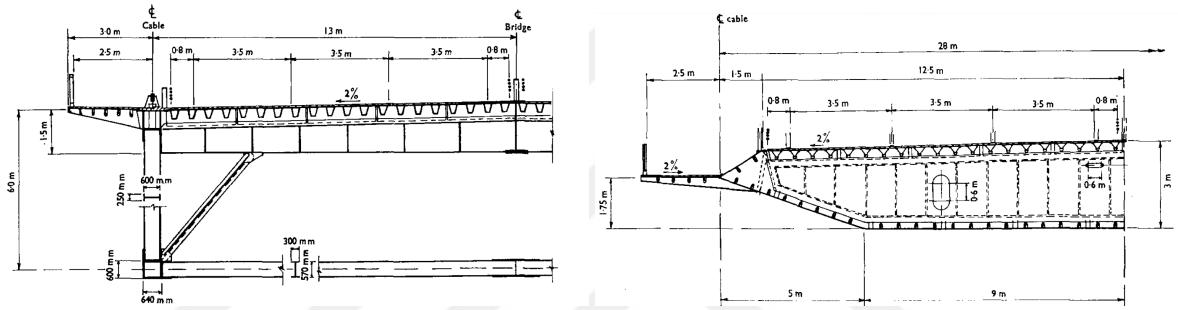
Suspension bridges are generally the primary solution for crossing very long spans. They have been designed and built since 19th century and most of are still in service. These types of bridges have always been considered as elegant looking and monumental structures. Although the idea of building suspension bridges is not new, it was always at the forefront of bridge engineering innovation. The design and construction are continuously evolving throughout the years with the developments in material technology and structural analysis capabilities. The boundaries of bridge engineering are still being pushed to cross longer spans and to increase the durability of suspension bridges. Today, the world's longest suspension bridge is under construction located in Canakkale, Turkey with a record span of 2023 m. Recent studies show that crossing spans up to 5 km will be possible in future [3].

Meanwhile, maintaining these landmark structures and keeping them in good shape is another challenge which bridge engineers and operators face with since they are degrading under loading and environmental effects throughout the years. In order to fulfill and extend the service life of suspension bridges requires continuous effort of maintaining activities.

The Bosphorus Bridge is an iconic suspension bridge with a span of 1074 m constructed in 1973, located in the heart of Istanbul, Turkey. It is the first suspension bridge built in the country and serves as an extremely important link of the city transportation network connecting Europe and Asia over the Bosphorus Strait with an average daily traffic of 190,000 vehicle/day. There are two main structural features of the bridge which needs to be elaborated further on.

Bosphorus Bridge, designed by Freeman Fox and Partners, is one of the first examples of a suspension bridge with a streamlined box girder deck instead of a stiffening

truss which provides significant contribution to aerodynamic performance and at the same time employs structural efficiency and aesthetic beauty by reducing the girder depth and hence self-weight greatly. This can be explicitly demonstrated in Figure 2 by looking at the drawings of two girder design alternatives developed in the design phase of the bridge [2]. Since the girders main duty in this type of structure is to distribute the traffic loads to the hanger cables without causing local failure, it does not need to have very large flexural stiffness. The torsional rigidity required is provided easily by the closed box section.



(a) Truss Girder Alternative (b) Streamlined Box Girder Solution

Figure 1.1. Deck Alternatives considered in Bosphorus Bridge Design

Another very characteristic feature of the bridge is the inverted-V shape or diagonal orientation of the hangers. It is understood that this inclined layout of the hangers are selected in order to limit the longitudinal displacements and benefit from the inherent damping of the cables. However, this design has a major drawback which is the hanger experiencing higher stress amplitudes under traffic loads. In 1982, Homberg criticized the use of this orientation by noting that inclined hangers can have an early fatigue problem due to the high amplitude stress reversals [4].

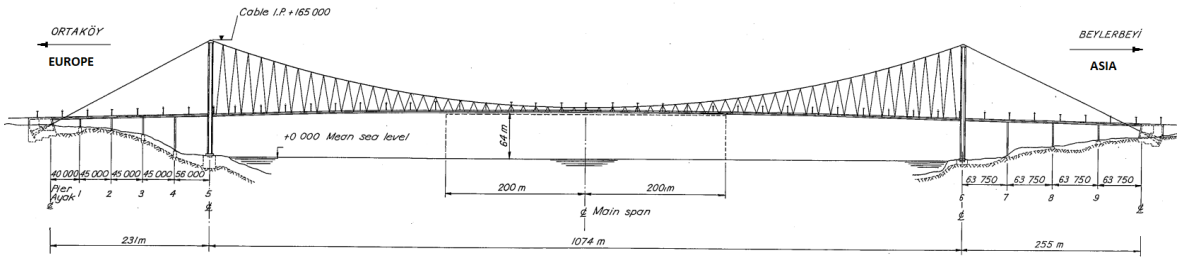


Figure 1.2. Bosphorus Bridge Elevation View - Original Hanger Arrangement

The Severn Bridge constructed in 1966 and Humber Bridge constructed in 1982 have very similar designs to Bosphorus Bridge and sharing the same inclined hanger system. In both of these bridges, hangers are replaced short time after service due to fatigue cracking of wires. This somehow proved Homberg's theory about inclined hangers as well. When the second bridge on bosphorus, namely Fatih Sultan Mehmet Bridge, is completed at 1989, the local authority (KGM) decided to ban truck passage from the Bosphorus Bridge as a protective measure. Definitely, this decision relieved significant loading and extended the service life for the fatigue prone elements. In 2004, a hanger plate failed at the Bosphorus Bridge due to fatigue. This increased the need for examining the bridge closely and after an extensive inspection period, Turkish General Directorate of Highways decided to rehabilitate the bridge. Retrofit designs are prepared by Parsons Transportation Group which included the replacement of hangers with vertical ones. The construction works finished and bridge is under service with the new vertical hangers since 2016.

In a suspension bridge, main load carrying mechanism is tension which is carried by the steel cable elements. In simple terms, the dead and live load of the bridge is distributed by the deck and then taken up by the hanger cables and transferred to the main suspension cables. Main cables pass over to tower saddles which provides vertical support and then terminates at the anchorages. Hanger cable is a critical link in this load carrying system since they experience the local load fluctuations caused by live loads and they are under continuous environmental attack during the service life.

The background on this bridge presented above brings us to the subject matter of this thesis. Fatigue performance assessment of the newly installed hangers under traffic loading is intended in this study. Since the bridge itself and the similar bridges had suffered from the same problem in the past, this study aims to provide an idea about the fatigue performance of the new hangers in use.

1.2. Literature Research

The fatigue assessment of hangers in existing suspension bridges under traffic loading is a very important topic and many studies are conducted to evaluate the hanger fatigue life of critical bridge. The fatigue performance of hanger cables on an existing bridge is a meaningful concern for bridge operators as well in terms of asset management and in order to meet the increasing traffic demands through out the years of service.

1.2.1. Suspension Bridge Computer Modeling

Developing an accurate analytical model is the first step of any stress analysis to be performed. The flexible nature of suspension bridges complicates the structural analysis of the system. When large displacements occur even under self-weight, the stiffness of the structure changes too. This geometric non-linearity precludes the use of a linear elastic approach for the analysis. The famous deflection theory developed for suspension bridges by Melan [5] considers the deformed shape of the structure in the analysis by suggesting a fourth order equation. The additional stiffening effect of the deck deflection reduced the girder stiffness demand. This approach was first adopted by Moisseiff in the design of the Manhattan Bridge. The solution for the force-displacement equation can be solved by numerical methods. The development of finite element method and the use of computers enabled the made the static and dynamic analysis of suspension bridges.

Tezcan and Cherry [6] made early contributions to conducting nonlinear analysis of suspension bridges including the large deflection effect. Using a 3D lumped mass system, they proposed an iterative method to obtain tangent stiffness matrices. This was also the first nonlinear dynamic analysis of a suspension bridge.Abdel-Gaffar [7] formed a computer method for performing dynamic analysis for suspension bridges based on finite element method and using linearized deflection theory. Nazmy and Abdel-Gaffar [8] also developed an computer model capable of performing 3-D non-linear analysis of cable supported bridges including tension stiffening effect of cables.

In modern times, sophisticated finite element softwares are developed and commonly used by engineers to model suspension bridges. The spine-beam modeling method is widely used for capturing the global response of suspension bridges. For capturing local effects, plate models are preferred. Ren *et al.* [9] discussed deeply the methodology to perform a nonlinear 3D finite element analysis with this approach. Bas *et al.* modeled the Bosphorus Bridge using shell elements [10]. Dynamic properties obtained from analysis is a good measure of the reliability of the finite element model. Hong *et al.* [11] and Karmakar *et al.* [12] also developed FE models for performance assessment of suspension bridges and validated their models using the field measurements of vibration frequencies.

1.2.2. Fatigue Resistance of Structural Strands

Cables offer great strength capacities with light self weight and size therefore being widely used in many structural applications. However, fatigue is an important failure mode for cables since they are likely to experience fluctuating loads and prone to vibrations due to their high flexibility and low structural damping [13]. Also, the fatigue mechanism for cables is different than other steel elements. Therefore, the fatigue of structural strands are extensively studied in the last 50 years. Since a strand is composed of many wires, the fatigue behavior is more complicated than a single wire or steel element. For axial fatigue of strands, which is also called T-T fatigue, the additional factors contributing to fatigue mechanism rise from fretting between the wires which is the frictional stress concentrations at inter-wire surfaces. Combined with corrosion, this effect can cause premature failures at strands. The strand termination details are also significant which can introduce local bending effects.

Cullimore [14] did some test to obtain an S-N curve and determine the endurance limit for cables below which no failure happens. 15 mm pre-stressing strands were used in the experiments and a S-N curve is proposed. The author concluded that endurance limit does not exists for the stress ranges considered.

Hobbs [15] performed two series of test on spiral strands with diameters of 16 and 38 mm. The stress range is expressed as a percentage of the minimum breaking load and wire breaks at 2 million cycles are reported. Okugawa [16] tested different diameters of suspension bridge hanger cables and described the failure criteria as 5 percent wire breaks. He found that at 2 million cycles, the corresponding stress range is 160 MPa. Nakamura combined these results and fitted a S-N curve based on first wire break criteria [17]. This curve will be adopted as the main S-N curve in this study.

Raoof and Hobbs [18] developed orthotropic sheet model to obtain contact stress between the wires which plays an important role when the axial fretting of the wires is considered. The construction of the strand (lay angle, number of wires, wire diameter) dictates the inter-wire stresses. The authors took this effect in to account by applying stress concentration factors to the stress ranges used in S-N curves. The authors also proposed applying factors which considers size effects and surface condition of the cable. Alani and Raoof [19] discussed the effect of mean axial load on fatigue life. They also suggested that endurance limit increases with increasing mean axial load.

1.2.3. Fatigue Analysis on Hanger Cables of Existing Bridges

The susceptibility of the structural cables to fatigue failure is a common concern for researchers and efforts are made to determine the resistance of strands to fatigue as summarized above. For suspension bridge operators and engineers, a more important question is the reliability of their hanger cables in service. Wire breaks on hangers are detected in many suspension bridges all over the world and this can cause replacement of strands within the life time of the structure. Inspecting the entire length of the cables is not very easy because of access problems and even it is accessed, only outer wires can be inspected. A solution to this problem may be the use of non destructive evaluation methods such as the magnetic flux leakage sensors. However, assessment of the current condition of cables is not sufficient alone since it does not give information about the future performance. In order to perform a life-cycle analysis for the hangers and find out the remaining service life of hangers, a numerical analysis is necessary considering the load effects and corresponding stress ranges and cycles resulting in cables. This is

recently emerging research topic and there are some attempts to perform such analysis found in the literature.

Nakamura [17] made one of first attempts to evaluate the fatigue life of hangers in a suspension bridge under traffic loading. Author studied the controversial inclined hanger system of the Severn Bridge. The field surveyed traffic data is used to construct a simulation of 20 minutes flow based on Monte Carlo method. The author obtained the stress time histories by using influence lines and calculated fatigue life of hangers according to the S-N curve that is proposed for spiral strands. The results showed that fatigue life of hangers are unsatisfactory and calculated fatigue lives as low as 30 years especially near the mid-span.

Petrini and Bontempi [20] studied the fatigue damage on hangers of a long span bridge under wind and train transit action. A framework for evaluating fatigue of hangers at the design stage of a bridge is presented. The results showed that train load has a bigger influence than wind load when the fatigue of suspenders are considered.

Zeng and Tan [21] performed a reliability assessment of fatigue life of hangers in a suspension bridge. Site specific traffic data on the bridge is collected for 48 hours to develop a statistical model of vehicle load and established a stress spectrum for hangers using finite element method. Fatigue reliability index is calculated and compared with the target index. The damage levels are classified and maintenance activities are assigned to each damage level which can be used to schedule inspection activities . The results showed each hanger can require difference maintenance methods since hangers have different levels of damage.

Wu and Jiang [22] studied the fatigue damage accumulated on an operating three tower suspension bridge within the life time of the structure. The WIM records of an another nearby bridge is used to create a random traffic load model with Monte Carlo method. The traffic model treated each lane separately. A spine beam FE model is used for analysis and hanger stress analysis under live load is performed through influence lines. The influence lines showed that the outer lanes have the biggest effect the load is

shared by 80 hangers that covers 800m. A function is fitted for each hanger's influence line and stress time histories are created by multiplying this function with the load amplitudes at every step distance of flow. Most of the stress cycles are reported to be lower than 15 MPa but the biggest contribution to damage is due to the higher stress ranges with lower number of cycles. Two different S-N curves are utilized in fatigue damage analysis. One of them is the curve suggested by Eurocode 3 and the other one is based on test made on corroded wires. The effect of corrosion on fatigue life is found to be very significant, giving as high as 3000 times of damage.

Deng *et al.* [23] presented a similar work on assessing fatigue life of suspension bridge hangers. This time, the deck is modeled with shell elements to obtain transverse distribution of the load precisely. Instead of influence lines, an influence surface is used to obtain hanger response. The traffic loading data for 8 months is obtained from WIM data and directly used in the analysis. Vehicle weights less than 30 kN is ignored in the analysis. The stress ranges extracted were around 20 MPa. From the results, it is observed that fatigue lives of hangers follow a decreasing trend from towers to mid-span.

Li *et al.* [24] proposed a method for long-term condition assessment on suspenders of Tsing Ma Bridge. The vehicular live load on the bridge is identified through the WIM system installed on the bridge and the train loading is calculated from the strain gages installed on the rail track beams and assigned distributions to them. An updated 3D FE model of the bridge is constructed and stress time histories are calculated by use of influence lines. Instead of S-N curves, the capacity of the hanger cables are estimated by stochastic modeling of deterioration in wires using Monte Carlo method. The reliability index for serviceability and safety is then calculated separately for the hanger.

1.3. Objective and Scope

Suspension Bridges are naturally the most important link in a transportation network like the Bosphorus Bridge being a vital crossing for the Istanbul city. Conse-

quently, for a sustainable service life, these bridges need careful operation and maintenance. In this study, the hanger cables will be the primary focus since they are one of the most vulnerable elements of the structure. In any cable supported bridge, the cables are susceptible to fatigue failure due to cyclic traffic loads and wind induced vibrations in combination with the corrosive environment. When the flexible structural formation with low damping nature of a long span suspension bridge is considered, fatigue mechanism can have a significant effect on service life. Cable wire breaks are very commonly observed in suspension bridge hangers and usually replacement of hanger cables is generally an inevitable activity within the service life of the structure. Bridge operators need to be aware of this situation and plan the inspection and retrofit campaigns accordingly.

In this view, fatigue performance assessment of the newly installed hangers on the bridge is intended in this thesis. This study will provide an idea about the reliability of hangers in use and give an idea about the future plans for inspection and cable replacement activities. Since there is no structural health monitoring system available for the present state of the bridge, stress time histories which are necessary for fatigue analysis will be obtained analytically. However, this is a complex engineering problem and by nature involves high degree of unknowns. A meticulous engineering approach is necessary to obtain the correct structural output for the cables, representing the actual case. The first uncertainty is the traffic loading on the bridge. Accurately modeling the load effects borne by traffic flow is extremely difficult. Engineers usually benefit from the WIM systems installed on bridges in order to develop a specific traffic model for the bridge under investigation. Unfortunately, such system is not available in Bosporus Bridge. This problem will be overcame by assumptions based on observations and available data on the traffic flow which will be described in Chapter 3. Another difficulty in such an analysis is capturing the actual structural behavior. Due to the high non-linearity included in suspension bridge structural system, a very precise FE model will be developed and dynamic analysis results are compared with previous studies for verification. Finally, the fatigue phenomenon in structural strands is well explained analytically so the S-N curves based on laboratory test are used to estimate accumulated damage.

After taking in to account all the unknowns as necessary with the tools created, a fatigue life assessment of the hangers will be performed. The developed FE model will be used for analysis of the structure under defined traffic loading scenario. Taking advantage of the time history analysis tool provided by the FE software, stress histories resulted in hangers will be obtained. Based on the fatigue damage theory adopted, service life for the hangers will be estimated. The effect of the location of the hanger on fatigue life will be also discussed. Finally, the effect of corrosion on fatigue life will be presented.

1.4. Thesis Outline

This thesis is structured on 5 main chapters. In chapter 1, background information is presented of the structure to be studied. The literature survey about suspension bridge modeling, fatigue damage theory, cable S-N curves and fatigue analysis on hanger cables are introduced.

In Chapter 2, the modeling technique used for the bridge is described. Details of the input parameters are presented and structural analysis type selected is described. The methodology used for initial shape finding is described and initial tension values obtained for hangers and main cable elements are calculated. The primary natural mode shape and frequencies are presented and compared with previous studies.

In Chapter 3, the developed traffic simulation model for using in the time history analysis is described. The vehicle types and properties, together with the traffic flow information on the bridge is presented.

In Chapter 4, the fatigue assessment of the hanger cables are performed. Here, stress time histories are obtained by a time history analysis performed based on the traffic model developed thanks to the capabilities of the analysis software used. Stress amplitude spectra for every hanger is extracted with the help of a Rainflow counting algorithm. Afterwards, the results are compared with the actual measurements taken from site. Finally, the corresponding accumulated fatigue damage is calculated based

on two different S-N curves and reliability of hangers is discussed.

At the end, In Chapter 5, a brief summary of the study is presented. The concluding remarks on the thesis are made together with the recommendations for future research on the subject.

2. FINITE ELEMENT MODELING OF THE BRIDGE

This chapter will focus on developing an analytical model for the Bosphorus Bridge. In order to capture the realistic structural behavior of the bridge and perform a condition assessment for fatigue on hangers, a precise computer model is generated with a rigorous approach. The FE model of the bridge is developed using the commercial CSI Bridge Software Version 20 based on design drawings, previously published papers and site observations. In Figures 2.1 and 2.2 below, the elevation and isometric view of the generated global 3D model of the bridge is presented. The same model will be used for static and dynamic analysis.



Figure 2.1. FE Model Elevation View.

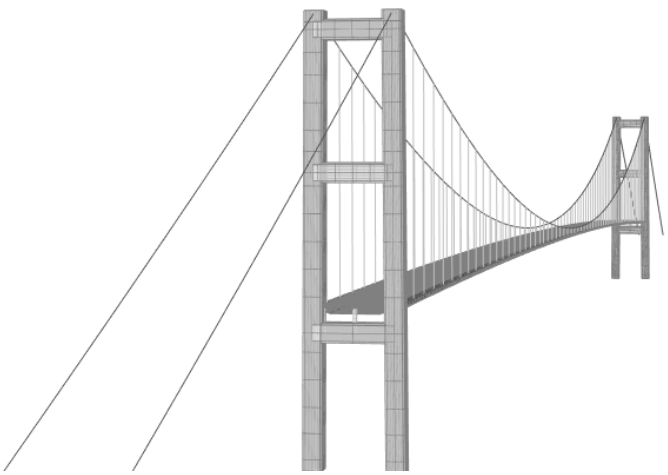


Figure 2.2. FE Model 3D View.

Since large displacements occur for a flexible structure like this even under self-weight, the stiffness of the structure changes too. This geometric non-linearity pre-

cludes the use of a linear elastic approach for the analysis. Due to the geometrical non-linearity of the structure, a nonlinear static analysis needs to be performed under dead loads taking into account the deformed shaped of the structure. The program uses Newton-Raphson iterations to obtain stiffness matrix of the structure at equilibrium. Once the equilibrium state is reached, other linear analysis can be performed using the stiffness obtained by this analysis. Material non-linearity is not considered in the analysis since elements of this type of structure is designed to remains within the proportionality limit. The P-Delta and large displacement effects need to be taken into account in the analysis especially for the cables.

The main span of the bridge is 1074 m with a tower height of 165 m. The back spans are unequal with lengths 231 m and 255m respectively. The sag of the main cable is 93 m which corresponds to a sag to span ratio of 0.086. Including the side spans, the bridge total length is 1560 m.

A spine model approach is preferred for deck modeling consisting of longitudinal beam elements representing the box girder and transverse weightless rigid link elements for load transfer to cable connections (see Figure 2.3).

The deck has a camber of 8 meters and this has been taken into account in modeling since it will affect the hanger lengths. Based on the geometry, it will be useful to fit a polynomial function to describe the deck profile which is presented in Equation 2.1. In this equation, x is the longitudinal distance to mid-span and y is the elevation in meters.

$$y - 0.000027938402x^2 + 68.245 \tag{2.1}$$

Since the mass and stiffness of the deck has crucial importance for the behavior of the bridge, deck is modeled precisely using the as-built drawings of the bridge. The CAD drawing of the deck section is prepared and imported into software with exact plate thicknesses and including longitudinal bulb stiffeners and V-ribs. Hanger gusset

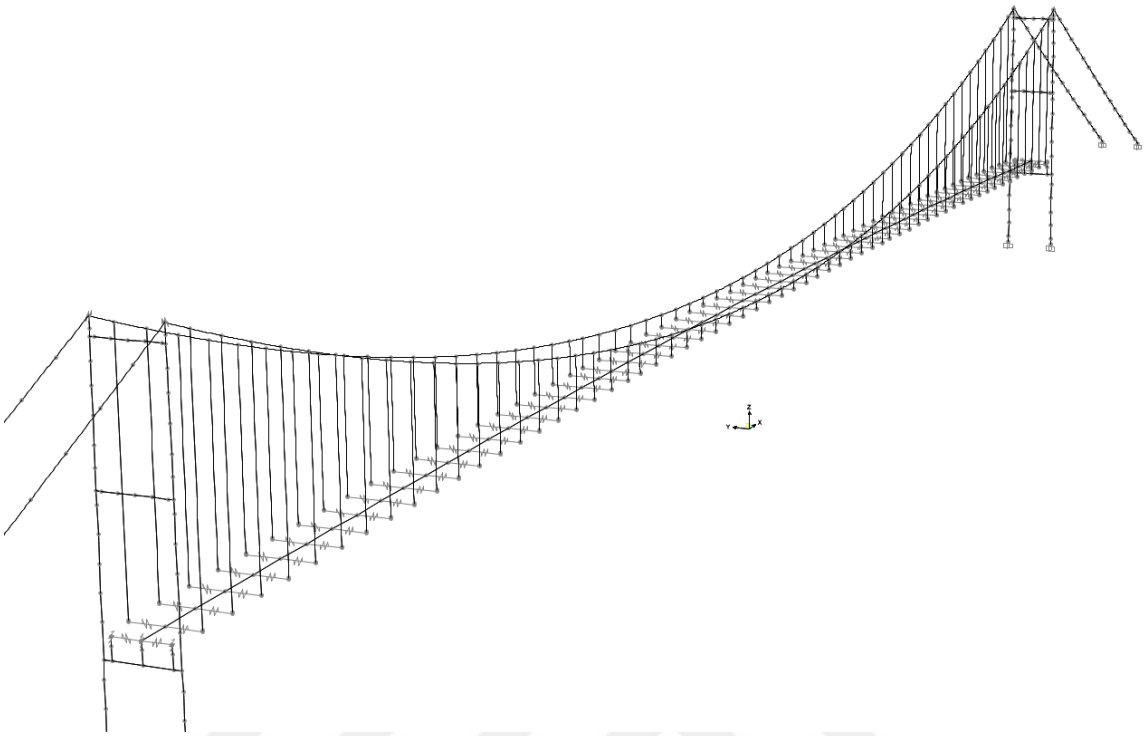


Figure 2.3. FE Spine-Beam Model Structure.

plates are also modeled separately using frame elements with the intention of correctly defining the hanger lengths. In Figure 2.4, an assembly consisting of deck, hanger, main cable and gusset plate elements is shown.

The side spans are not modeled because they are not suspended but rest on vertical piers and also completely separated from the main span deck. Hence, modeling side span is not likely to contribute in the structural behavior of the main span. It may affect the tower dynamic behavior due to added mass but this lies beyond the scope of this study.

Towers are modeled using beam elements as well. The steel box cross section is defined in the analysis software including the longitudinal bulb stiffeners. This was done again by generating CAD drawing of the section based on the as-built drawings and then importing the CAD drawing as a section file in CSI Bridge. In the real structure, tower sections have a slight taper on one side but this is ignored in the

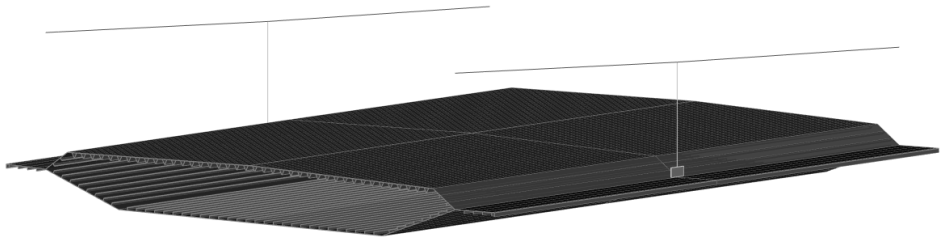


Figure 2.4. View of Modeled Deck Section and Hanger Connection Detail

model.

Hangers and main cable elements are modeled using the cable object in the software which is a nonlinear element which adopts elastic catenary formulation and takes the tension stiffening of cable into account. The cable element is defined as a tension only member with no flexural stiffness. There are 30 hangers of different length in each quarter of the bridge which makes in total 120 hangers Actually, at each hanger location, a pair of hanger cables exists on the bridge but they are modeled as one element having the total cross section area for convenience. This is a very reasonable assumption since the two cables share the same connection member at both ends and they are very closely spaced so that the loads will be shared equally. Main cable nodes are taken from the as-built drawings of the bridge which means they are defined at the deformed state of the bridge under dead load. This approach will require incorporation of initial tension at those elements in order to have zero deflection under dead load when the analysis is performed. This process will be broadly described in the coming sections.

Since the main cable assumes a parabolic shape under dead load, it is easy to fit a function to tha main cable location as done for the deck profile. This function is



Figure 2.5. View of New Vertical Hangers of Bosphorus Bridge

presented in Equation 2.2.

$$y = 0.000322x^2 + 72 \tag{2.2}$$

The deck-tower connection detail is made of two rocker links that provides vertical restrain and a wind shoe preventing transverse displacement. These members are modeled using link elements. The base of towers have fixed against all 6 degree of freedoms and the cable ends are fixed against translation at anchorages.

2.1. Model Input Parameters

In this section, the parameter inputs used during the creation of analysis model will be given. The material and section properties used will be of extreme importance since they could easily effect the structural behavior especially in these kind of bridges.

The material properties used for the sections are presented below in Table 2.1. All members of the bridge are made up of structural steel. The young modulus used in the analysis will also have a significant effect on the force – displacement behavior especially when cables are considered. For the main cable, a Young's modulus of 193 GPa is used whereas for hangers, a modulus of 165 GPa is taken according to industry standards for corresponding cable types.

Table 2.1. Material Properties.

Member	**Young's Modulus**	**Poisson's Ratio**	**Unit Weight**
Deck	205,000 MPa	0.3	78.5 kN/m^3
Tower	205,000 MPa	0.3	78.5 kN/m^3
Main Cable	193,000 MPa	0.3	78.5 kN/m^3
Hanger	165,000 MPa	0.3	78.5 kN/m^3

The cable type for the new hangers installed on the bridge is 55 mm diameter open spiral strand which is made up of 127 helically laid individual hot dip galvanized high strength round wires. A sample view of the strand is given in Figure 2.6.

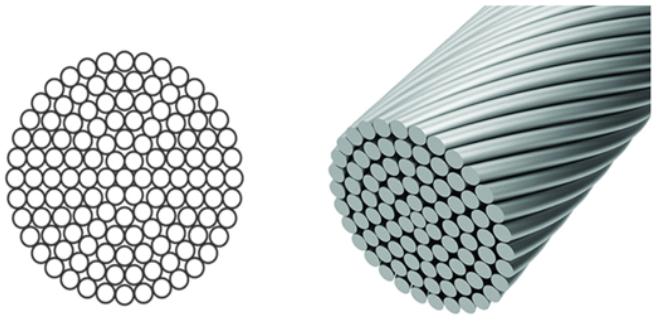


Figure 2.6. Open Spiral Strand View (Image taken from Redaelli Catalogue Cut).

The main cable is formed by parallel wires strands. The main cable has a different cross section at the back span and main span. This is due to the different inclination

of the main cables at both sides of the tower saddle. Main span section consists of 10,414 number of 5 mm diameter round wires where the back span has 11,176 wires. The section view of the main cable is presented in Figure 2.7.

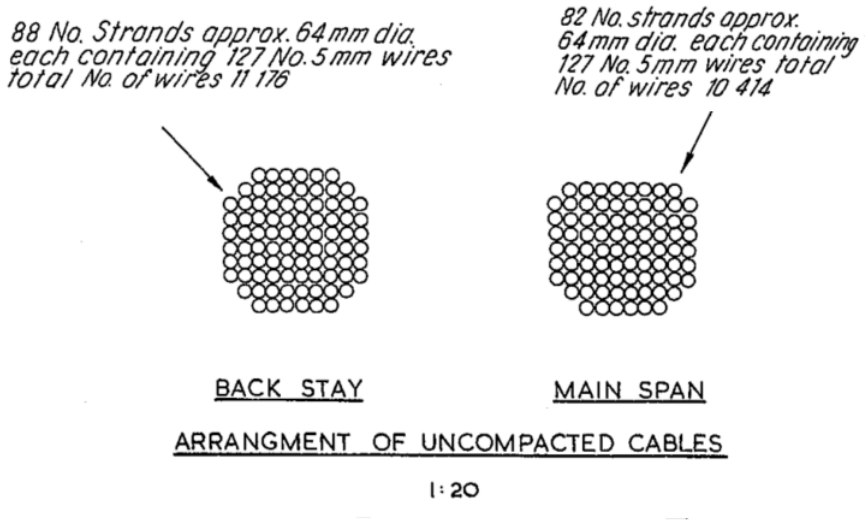


Figure 2.7. Dead Loads of Components of Bosphorus Bridge [1].

The section properties used in the model for all members are presented in table 2.2 and 2.3. The dead load on the bridge needs to be carefully determined for a

Table 2.2. Frame Section Properties.

Member	**Area**	**Moment of Inertia,I_{yy}**	**Moment of Inertia,I_{zz}**	**Torsional Constant**
Deck	0.8689 m^2	65.972 m^4	1.256 m^4	3.509 m^4
Tower	0.7228 m^2	2.852 m^4	4.915 m^4	4.873 m^4
Gusset	0.025 m^2	1.3x10^{-6} m^4	2.1x10^{-3} m^4	5.13x10^{-6} m^4

suspension bridge model. This is important not just for getting the correct static results but also for obtaining the right stiffness for the modal and traffic load analysis that will be performed afterwards. The dead load of the structural components are calculated

Table 2.3. Cable Section Properties.

Member	**Area**	**Moment of Inertia**	**Torsional Constant**
Main Cable	0.2044 m^2	3.325x10^{-3} m^4	6.649x10^{-3} m^4
Back Span	0.2195 m^2	3.834x10^{-3} m^4	7.668x10^{-3} m^4
Hanger(double)	0.003703 m^2	1.091x10^{-6} m^4	2.183x10^{-6} m^4

automatically by the analysis software using defined sections and material unit weights. The additional loads of pavement, auxiliary members(handropes, barriers,services etc.) are also taken into account according to the as-built drawings and the design paper [2] as presented in Figure 2.8. The effect of these superimposed loads are included in the analysis by applying a modification factor to the deck section unit weight. From here, the total distributed dead load for the suspended deck is calculated as 110 kN/m.

	Main span	Ortaköy sidestay	Beylerbeyi sidestay
Cables	3·32	3·90	3·86
Cable wrapping	0·08	0·09	0·09
Cable bands	0·11	0·03	0·03
Handropes	0·02	0·02	0·02
Protective treatment	0·01	0·01	0·01
Hangers and sockets	0·16		
Suspended box steelwork	8·02		
Roadway surfacing	2·25		
Footway surfacing	0·08		
Parapets and crash barriers	0·20		
Services	0·22		
Protective treatment	0·07		
Total design dead load	14·54	4·05	4·01

Figure 2.8. Dead Loads of Components of Bosphorus Bridge [2].

2.2. Initial Tension of Cables

Unlike any other structure, precisely modeling the geometry,section properties and boundary conditions are not sufficient to establish a robust structural analysis model for a cable supported bridge. If one performs a static analysis at this step, the deformation of the bridge will be unrealistically high. In our case, the mid-span vertical deflection of the deck went beyond 9 meters which is not reasonable.

As a matter of fact, in a suspension bridge, the main source of stiffness is the tension in the cables. Therefore, before any analysis, this tension has to be present in the cables at the dead load state of the structure. There exists two ways to get this initial tension in the cables. One of them is developing a construction stage analysis model, by which every step of construction is simulated. This will require the knowledge of initial catenary shape of the main cables and many precise details about the superstructure erection process. At the end of such analysis, one can obtain the finished geometry of the bridge with the corresponding tension in cables. This is the general method applied by the designer of a cable supported bridge.Such process is also called shape finding [25].

However, when dealing with an existing bridge, the initial geometry of the cables are not known. The bridge is modeled based on the completed state which is actual the deformed shape under dead load. Therefore, the cable tensions needs to be entered to obtain the tension stiffness so that when a static analysis is performed under dead loads, the structure should be at equilibrium with no deflection meaning preserving the final geometry [9]. This can be done by performing a nonlinear static analysis and using the resulting cable tensions as a pre-strain input for cables then repeating the analysis again. This trial and error process is done until a minimal deflection of the deck is obtained. Kim [26] proposed a similar method based on the Newton-Raphson method to analyze a cable supported structure at the target configuration under dead load.

In this study, a feature of the analysis software is utilized called CSI Load Optimizer which follows a similar approach same described above. Once the target displacement criteria is defined at the selected nodes which in this case the deck nodes at hanger locations, the program adjusts the cable tensions automatically under the dead load case. Since this is an approximation through a trial and error algorithm, having exactly zero displacement at the nodes should not be expected. The deck deflections obtained after this analysis are around 3 cm which can be considered as negligible.

Another key point while defining the target criteria is the rocker links (vertical deck support frames located at towers). These members has to be in compression under at dead load state of the bridge. If a shape finding run is performed asking for zero deflection of the deck, the rocker links will take no load. Therefore, a separate criterion needs to defined as a minimal axial compression value for these members. Based on previous experience and engineering judgment, a value of 300 kN is adopted in this study. The dead load tension values are obtained by this way for each hanger cable (one of each pair at one hanger location) are presented in the Figure 2.9.

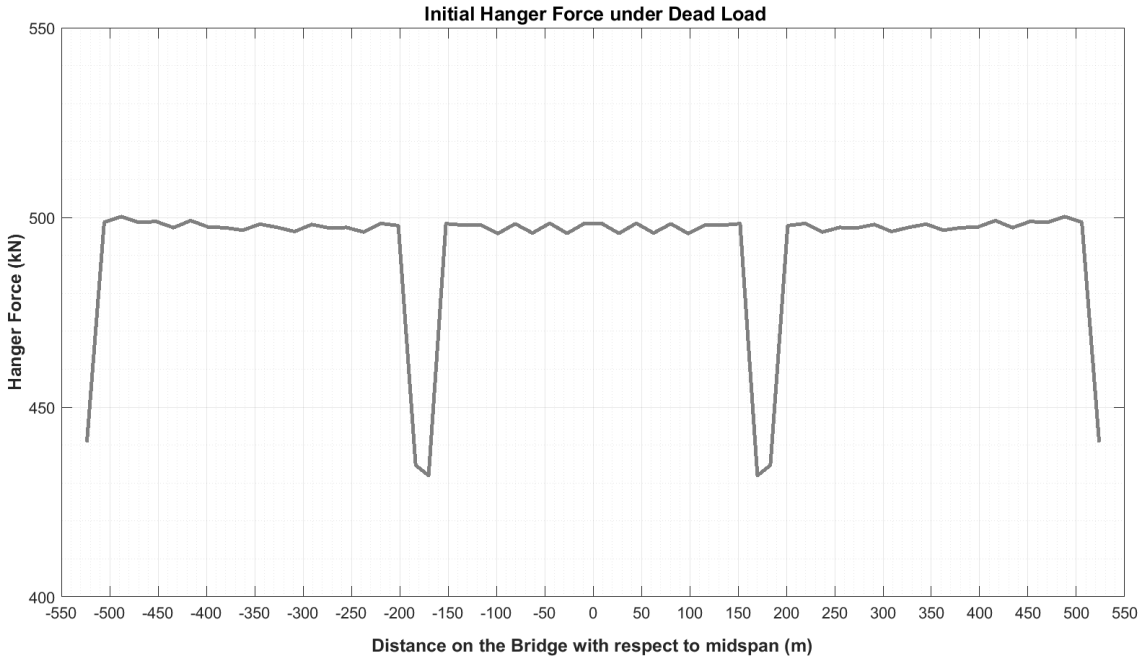


Figure 2.9. Hanger Tension at Dead Load State.

As can be seen from the figure, the tension values are around 500 kN for each hanger. The first hanger has a lower tension due to the load sharing of the rigid vertical support provided by the tower and the distance to the tower is shorter than the regular hanger spacing of 17.9 m. Also, hanger numbers 20 and 21 have lower tension because of the at that location, the hanger spacing is 13 meters.

In the Figure 2.10, the initial tension in the main cable under dead load is given. From this figure, it is seen that the cable tension at the main span is lower than the tension of back span and decreases towards the mid-span. Another outcome from the figure is that the back spans have different amount of tension. This is expected because the the length of the back spans and hence the inclination of the back span cable is different at each side.

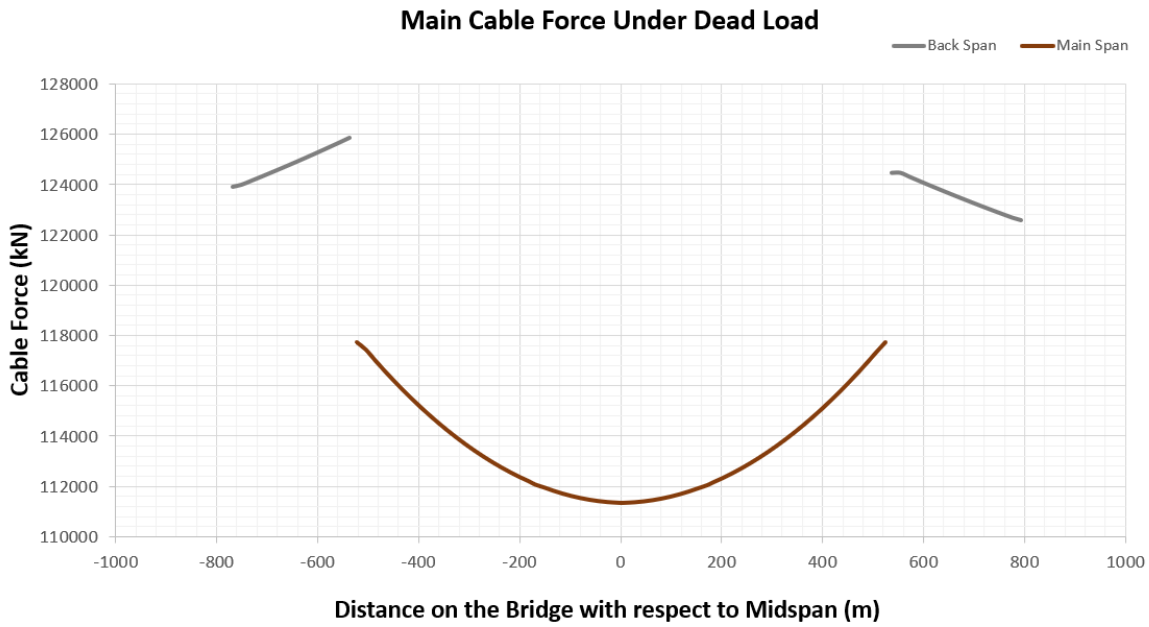


Figure 2.10. Main Cable Tension at Dead Load State.

2.3. Modal Analysis

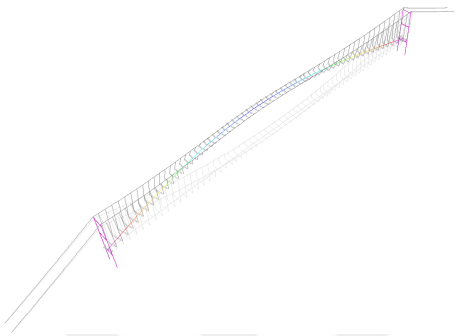
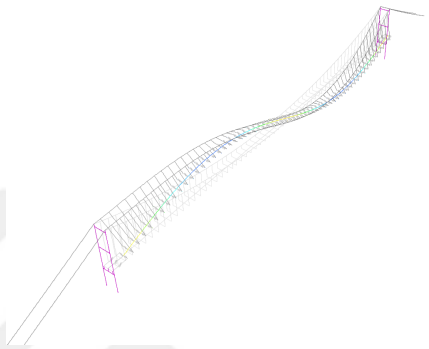
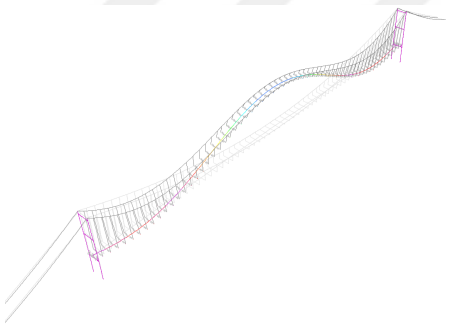
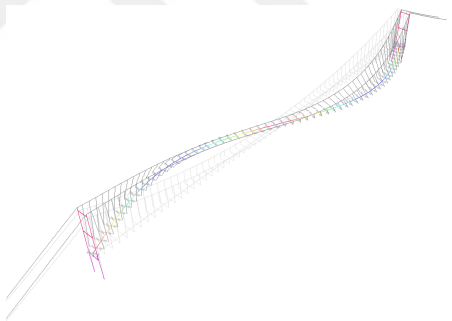
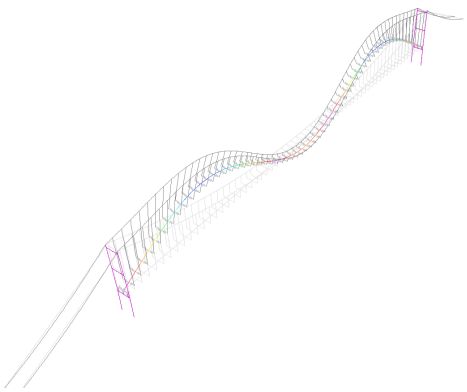
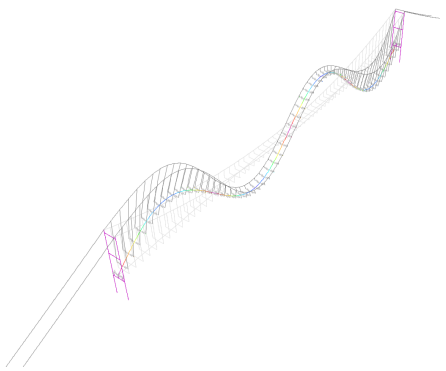
After static equilibrium is reached under dead loads and initial strains are defined at cable elements, the model has correct stiffness and is ready for further analysis.

Modal analysis needs to be performed using this stiffness as initial state since otherwise the cables would be unstable and insignificant mode shapes and frequencies could be obtained. In Figure 2.4 below, the main lateral and vertical mode shape and frequencies of the deck are shown. Recently, field vibration measurements is performed on the bridge after the vertical hangers are installed [27]. The results are presented in Figure 2.11. Comparing with the field identification results, the modal frequencies calculated by the FE model developed is found satisfactory, especially the vertical modes which will affect traffic load analysis, and therefore no model updating will be performed.

Mode	Before Retrofitting	During Retrofitting	After Retrofitting
1 (1st Lsym)	0.074 Hz	0.076 Hz	0.081 Hz
2 (1st Vasym)	0.137 Hz	0.137 Hz	0.125 Hz
3 (1st Vsym)	0.159 Hz	0.154 Hz	0.149 Hz
4 (2nd Vsym)	0.217 Hz	0.215 Hz	0.201 Hz
5 (2nd Vasym)	0.276 Hz	0.262 Hz	0.243 Hz

Figure 2.11. Identified Modal Frequencies.

Table 2.4. Primary Mode Shapes

1st Transverse Symm f=0.067 Hz	1st Vertical Asymm f=0.108 Hz
	
1st Vertical Symm f=0.152 Hz	1st Transverse Asymm f=0.193 Hz
	
2nd Vertical Symm f=0.204 Hz	2nd Vertical Asymm f=0.248 Hz
	

3. TRAFFIC LOAD ANALYSIS

In this chapter, the details of traffic simulation that fatigue analysis will be based on are given. Since we are trying to perform a condition assessment,a realistic and site specific traffic loading which will represent the service load effects on cables, has to be used instead of code-based load models. It is very complicated to precisely simulate the traffic flow since different types of vehicles passing through the bridge with random speeds and vehicle spacing. Also, during the day, the traffic flow at morning and night times is variable. A stochastic model of the traffic flow can be built by assigning probability distribution functions to number of vehicles passing, axle loads, vehicle speed for each lane. However, fur such analysis, a data set is required for each of this variables in order to fit a distribution to them.



Figure 3.1. A View of Traffic Flow on Bridge.

WIM systems are widely installed on critical bridges to provide extremely valuable information about vehicle speed, arrival time, number of vehicles and weight of vehicles. Such information will ease the process of developing a realistic traffic flow model for performing analytical condition assessments. This data can be used to fit probabilistic distributions to variables or one can directly use them in the analysis. Unfortunately, such system is not present on the Bosphorus Bridge. Under this circumstances, a traffic model will be generated based on the available average data. Here, we will simplify the analysis by assuming a deterministic traffic flow based on the daily average volume of traffic for each group of vehicle.

The bride accommodates 3 lanes in each direction with an average daily traffic (ADT) flow of 190,000 vehicle/day according to current situation. This is almost 1.5 times of the design traffic flow of the bridge which was 130,000 vehicles/day which means that it is operating way above the design value. In Figure 3.2, the deck cross section and lane configuration is shown.

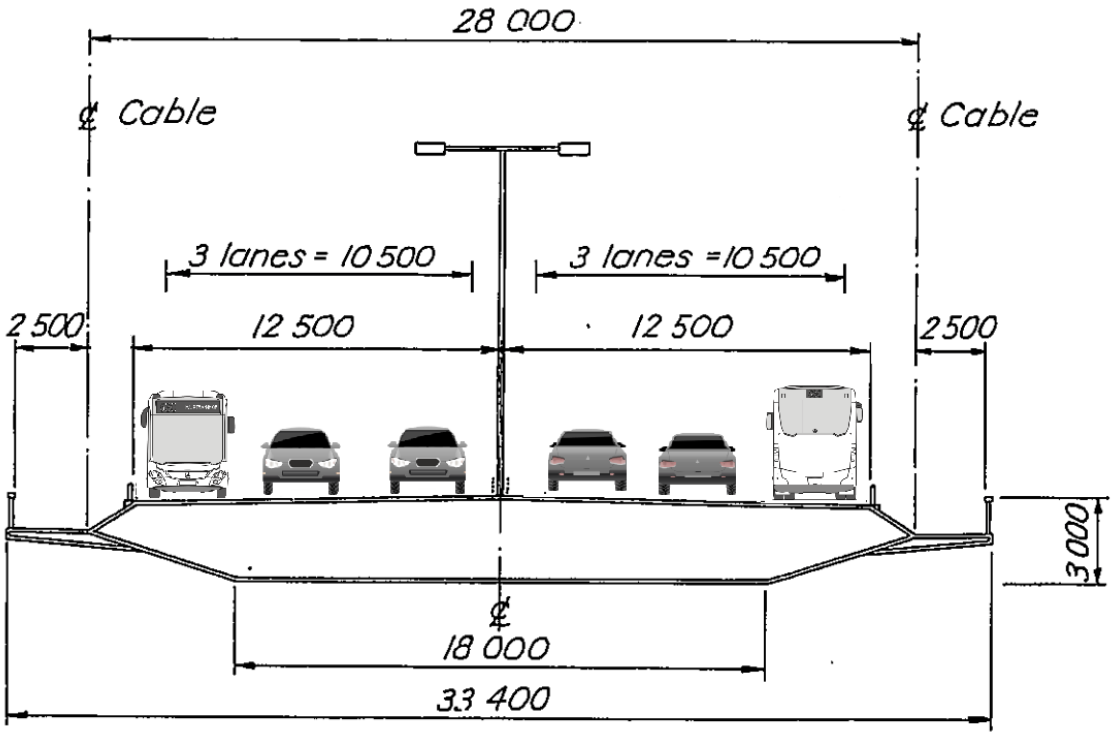


Figure 3.2. Deck Cross Section And Lane Arrangement [1]

When the second bridge on Bosphorus, namely Fatih Sultan Mehmet Bridge, was completed in 1989, the local authority (KGM) decided to ban truck passage on the Bosphorus Bridge as a preventive measure. Therefore, only passenger cars, public transportation buses (metrobus) and standard buses are using the bridge. The daily timetable of these buses is obtained from the municipality database. According to this, the daily volume of flow is for metrobus 3288 and for bus 259. The rest of the flow is composed of passenger cars. Typical articulated bus using the bridge and corresponding axle loads are presented in Figure 3.3. This vehicle is expected to create the major part of stress ranges with the total axle load of 365 kN. A convoy is generated based on the percentage of buses using the bridge over total traffic flow. The mean speed of the flow is taken as 40 km/hr in the analysis. It is assumed that vehicles moves with a constant speed and does not change lanes on the bridge.

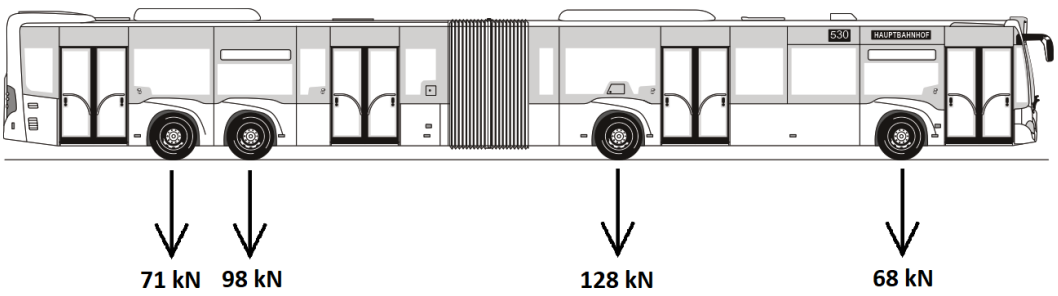


Figure 3.3. Articulated Bus Axle Loads Used in the Analysis

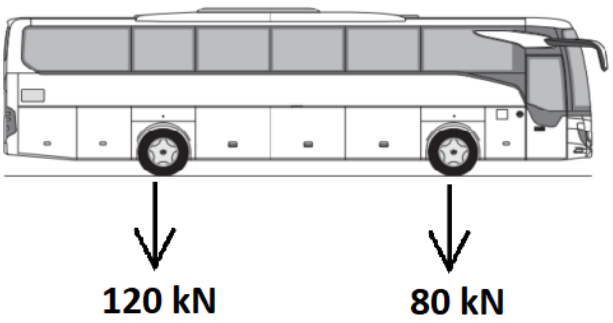


Figure 3.4. Standard Bus Axle Loads

Apart from this, standard 20 t coaches shown in Figure 3.4 are defined in the traffic scenario. Also, 1.5 t passenger cars with two axles are included in the convoy. By analyzing the daily trip numbers provided by municipality, it is found that roughly 10 percent of the total traffic flow consists of buses that are traveling in the outer lanes. The artificially generated convoy is then marched over the bridge and 10 minutes of traffic flow is generated in the analysis software. The details of the stress analysis will be given in the next chapter. The resulting stress time histories of the hangers are recorded for fatigue analysis.

4. FATIGUE ASSESSMENT OF CABLES

A fatigue assessment is aimed at hanger cables under variable stress fluctuations caused by traffic loading of 10 minutes. Many of the past studies used influence lines to perform this analysis but, in this study, we benefited from the analysis software capability of creating a time history live load case automatically based on the convoy defined and results of the time history analysis are obtained by direct integration. The program uses ramp loading functions that goes from zero to one in a time step and unloads to zero in the next step. Time step used in the analysis is selected as 0.2 second. Based on the traffic model established, a time history analysis is then performed by direct integration with Newmark method to obtain stress time histories at hangers. This way, a time signal for the loading is generated based on the defined vehicles.

Dynamic effects due to vehicular loads will be taken into account with this approach as well therefore there is no need to apply a dynamic amplification factor (DAF) separately. However, in a long span suspension bridge like this, significant dynamic amplification is not expected because the bridge has vertical frequency around 0.1 - 0.2 Hz as shown in Section 3 whereas vehicle frequencies generally are in the order of 2-5 Hz.

In Figure 4.1, the hanger response obtained from the analysis due to traffic loading only is shown for the longest (blue line) and shortest hanger (red line). It is clear from this figure that under the same loading scenario, different hangers experience different force levels and fluctuations. This effect will obviously result in different fatigue lives for hangers.

The fatigue tests are generally performed with constant amplitude stress cycles and S-N curves are generated combining the data of different tests performed at various stress levels. In real life, structural components like hangers experience cycles of variable stress amplitudes under service conditions. This complex stress time history of the hanger or the stress signal needs to be separated into constant stress ranges and

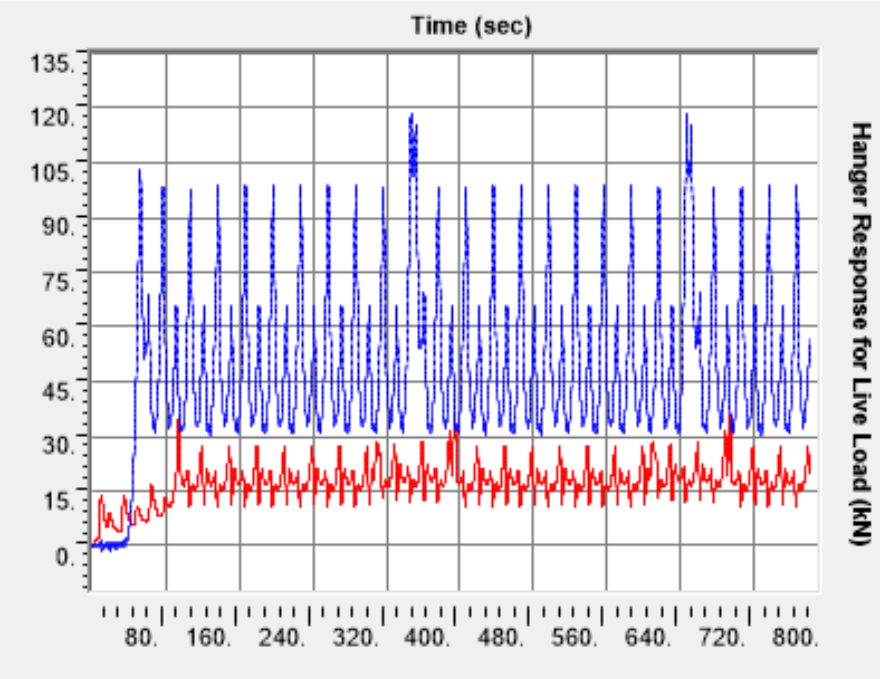


Figure 4.1. Axial Force Response of the Shortest and the Longest Hanger

corresponding cycles. These can be extracted by the record by making use of a classical Rainflow counting algorithm developed in Matlab. This method gives the same results with the Reservoir method. An example of a stress time history and corresponding Rainflow counts are presented in Figure 4.2 for hanger number 1. Hangers are numbered from 1 to 30 where 1 is the longest hanger located next to the tower and 30 is the shortest hanger at mid-span.

A temporary monitoring campaign was held on hanger cables and it provided the acceleration time history data for six months (data taken from the retrofit Contractor). Using the taut string theory [28] given in Equation 4.1, the tension in the cable is obtained based on the natural frequency and, the stress signal of the cable is artificially obtained. This data is valuable in the context of verifying the simulation results. As can be seen from Figure 4.3, the mean and fluctuating stress levels given for Hanger No.1 (next to tower) are in good agreement with the simulation results. Both results show that the stress reversals are below 10 MPa and the mean stress on cable is around

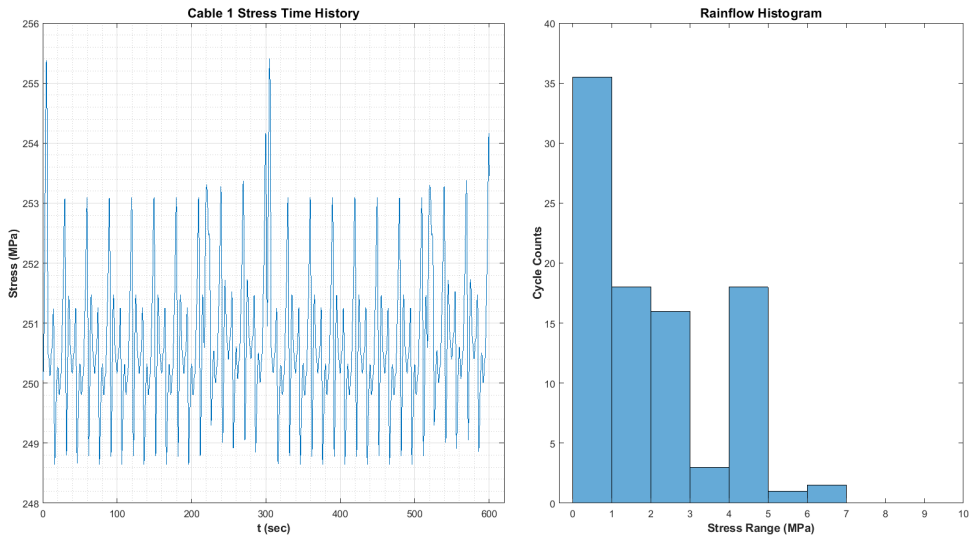


Figure 4.2. Simulated Stress Time History and Cycle Counts for Longest Hanger

235-250 MPa.

$$T = 4mL^2(\frac{f_n}{n})^2 \tag{4.1}$$

The provided data can not be used for direct fatigue analysis because the calculation of frequency has to be done over a time period. In this case, the monitoring system was reporting frequencies for every 10 min window. However, recent studies show that obtaining real time stress time history from cables is possible by calculating the tension in an other way instead of frequency [29]. This way, a fatigue analysis mat be possible from transverse acceleration measurement of cables which is relatively easy to measure on an existing bridge compared with other methods to obtain hanger forces.

After obtaining the stress cycles for every hanger, Miner's cumulative fatigue damage model is used to evaluate accumulated damage level. This method assumes that every stress cycle consumes a fraction of the fatigue life of a structure. This method assumes linear damage rule and ignores the effects of loading sequence and

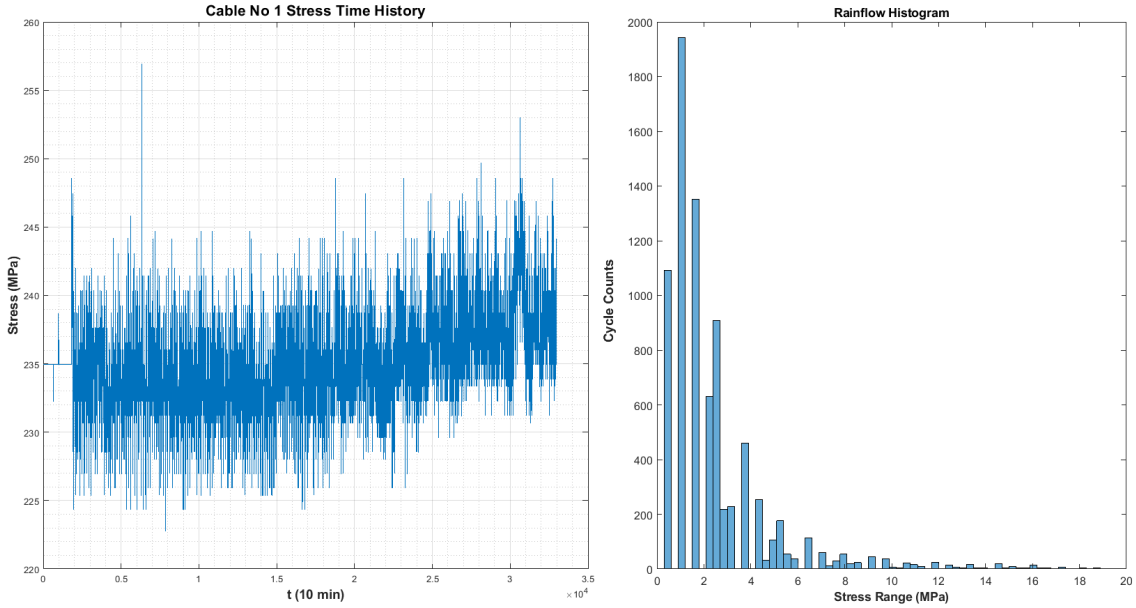


Figure 4.3. Measured Stress Time History and Cycle Counts for Longest Hanger

interaction. Some nonlinear damage theories also exist but Miner's method widely used in the field because it is very practical for variable amplitude fatigue. This relation is shown in the Equation 4.2 below. Here, D represents total fatigue damage, n_i is the experienced number of cycles and N_i is the total cycles to failure for corresponding i^{th} stress amplitude. When the damage is equal to one, the failure occurs.

$$D = \sum_{i=1}^{k} \frac{n_i}{N_i} \tag{4.2}$$

To calculate Ni, a fatigue life model needs to be used. In this study, we will focus only on axial fatigue of the cable and ignore bending effects. This is a reasonable approach since the hanger pin bushings on the bridge are equipped with spherical bearings which will help to prevent bending effects at cable terminations. In axial fatigue mechanism, fretting between the wires becomes significant in structural strands for fatigue life therefore structural configuration (number of wires, lay angle, shape of wires) of the strand matters. For example, spiral strands tend to have lower fatigue life than parallel wire strand due to the fact that contact stresses between the wires is

higher in a helical configuration. Corrosion of wires also triggers early fatigue problems and wire breaks in strands. The cable type for the new hangers installed on the bridge is 55 mm diameter open spiral strand which is made up of 127 helically laid individual hot dip galvanized high strength round wires. A sample view of the strand was given in Chapter 2 (see Figure2.6).

The fatigue life model for strands is not analytically defined yet therefore it can only be determined through laboratory testing. At this point, we will benefit from bi-linear S-N curves drawn in the light of laboratory testing. The S-N curve proposed by Nakamura will be used in the analysis [17]. This curve is based on many tests performed on open spiral strands with different diameters. The S-N curve is represented by Equation 2 below. The failure criteria of the S-N curves may differ from study to study and naturally has a significant effect on the calculated fatigue life. In this case, the failure criterion is first wire breakage in the strand. It is meaningful to note that this behavior model considers axial fretting fatigue but not the effect of corrosion. Unlike normal steel structures, there is no cut-off limit for strands that we can talk about below which no fatigue takes place [13]. This means that even the small stress fluctuations can initiate cracks and cause failure.

$$logN = 3.0 - 3log\Delta\sigma^* \tag{4.3}$$

The term $\Delta\sigma^*$ in Equation 4.3 represents the normalized stress range by the ultimate strength of the strand. Mean stress on the strand is another aspect which has an influence on fatigue damage and is taken into account by modifying the stress range with the Goodman relation given in Equation 4.4 [19].

$$\Delta\sigma^* = \frac{\dfrac{\Delta\sigma}{\sigma_u}}{1 - \dfrac{\sigma_m}{\sigma_u}} \tag{4.4}$$

Here the ultimate tensile strength, σ_u is taken as 1550 MPa and σ_m is the mean stress which varies between 240-280 MPa depending on the hanger location. The term $\Delta\sigma$ represents the stress range obtained from the stress time history record. The average value of the stress reversals is observed as 13 MPa and the peak value is 27 MPa.

For each hanger the mean stress associated to stress ranges and cycle numbers can be viewed by a three dimensional rainflow matrix to be used in the Goodman relation equation given above. In Figure 4.4, an example of this rainflow matrix is presented for Hanger No. 1.

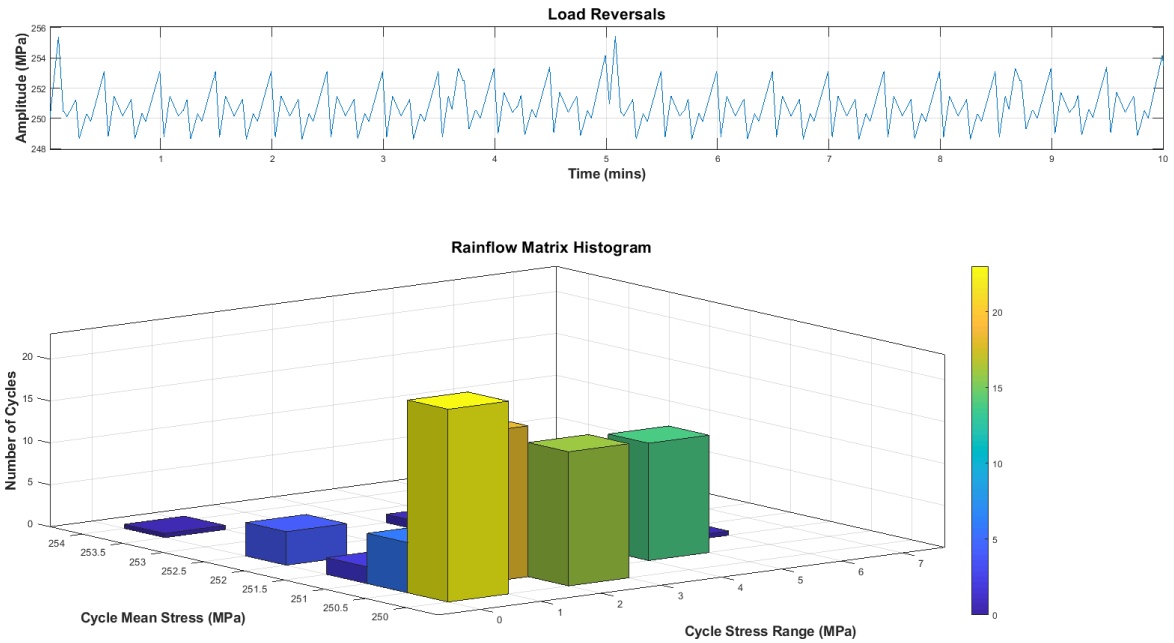


Figure 4.4. Rainflow Matrix Histogram for the Longest Hanger

The maximum stress ranges observed in each hanger is presented in Figure 4.5. It is seen from the figure that stress ranges increase towards the mid-span.

The cycles and corresponding stress ranges obtained in the analysis of 10-minute traffic flow is used to calculate fatigue damage by using S-N curve equation given

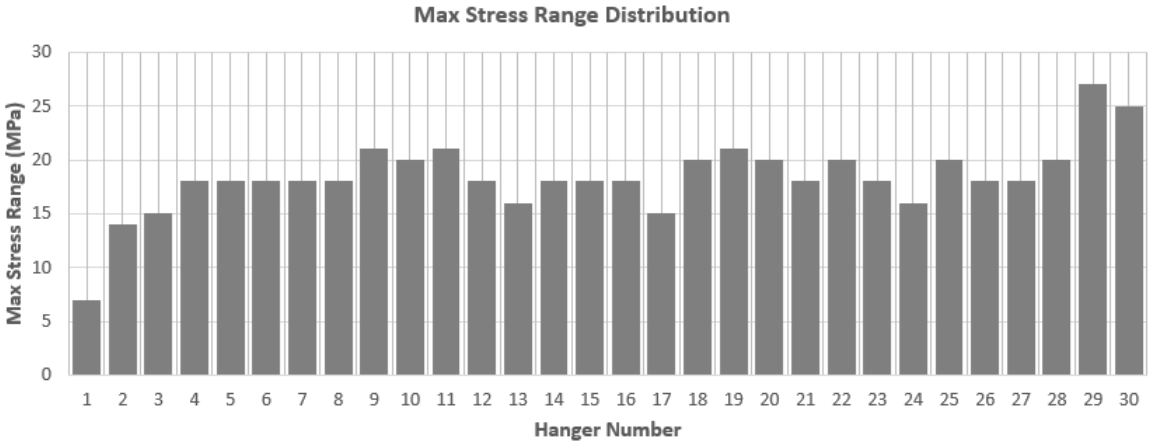


Figure 4.5. Max Stress Range in Hangers

and Miner's cumulative damage theory as described above. The calculated 10-minute damage is then extrapolated to get fatigue life, T in years with the equation 4.5.

$$T = \frac{1}{Dx6x24x365} \tag{4.5}$$

The calculated fatigue life of the hangers varies between 200 years to 11157 years as shown in Figure 4.6. The first observation made from examining the distribution of fatigue life among hangers is that the hanger next to the tower has a much longer life than other hangers. This can be explained with the contribution of the rocker link support near this cable which can take up a large portion of the live load effects.

In Figure 4.7, the accumulated fatigue damage on the cables in a potential 100-year service life is shown. This is another way of interpreting the outcomes of the analysis. This approach obviously assumes that current modeled traffic conditions will remain constant for the entire period considered. When the fatigue damage ratio shown in the figure reaches unity, it means that failure occurs. It is seen from the figure that the highest damage is experienced by the shortest hanger located at mid-

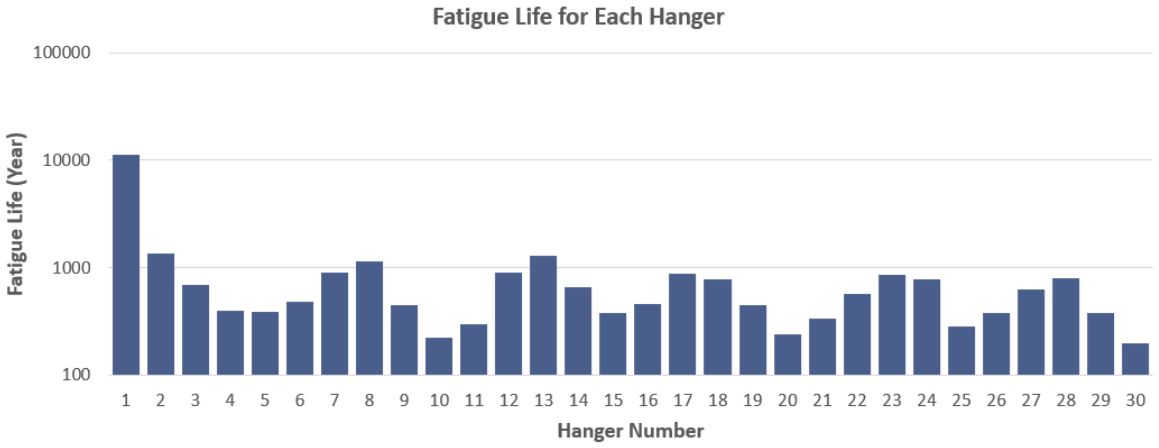


Figure 4.6. Fatigue Life Calculated for Hangers based on Traffic Simulation

span with a utilization ratio around 0.5. It is necessary to reemphasize, in this case, that failure corresponds to the first wire breakage on the strand as defined in the S-N curve adopted.

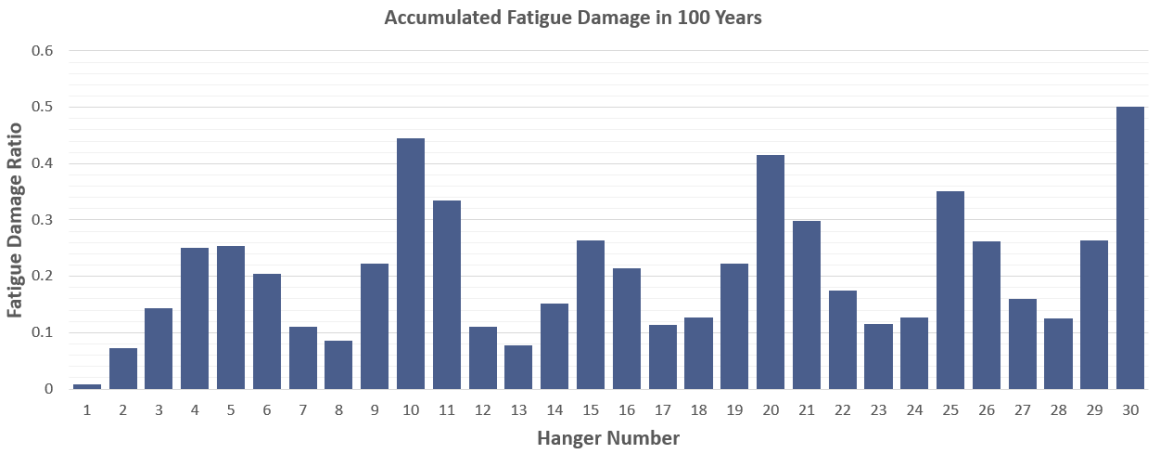


Figure 4.7. Fatigue Damage Ratio for Hangers Accumulated in 100 Years of Traffic Simulation

If the damage spectrum is highlighted for a hanger, like hanger 30 as shown in Figure 13, one can see each stress range contribution to total damage. As displayed above, the damage utilization ratio in 100 years for this specific cable was 0.5. From

examination of Figure 4.8, it is clear that stress ranges less than 10 MPa has a small effect on total damage where-as stress range of 20 MPa constitutes 70 percent of total damage occurred which shows that heavy vehicle passage is much more critical even though it happens less frequently. This is a rather expected result because in the S-N equation, the stress term has an exponential factor of 3.

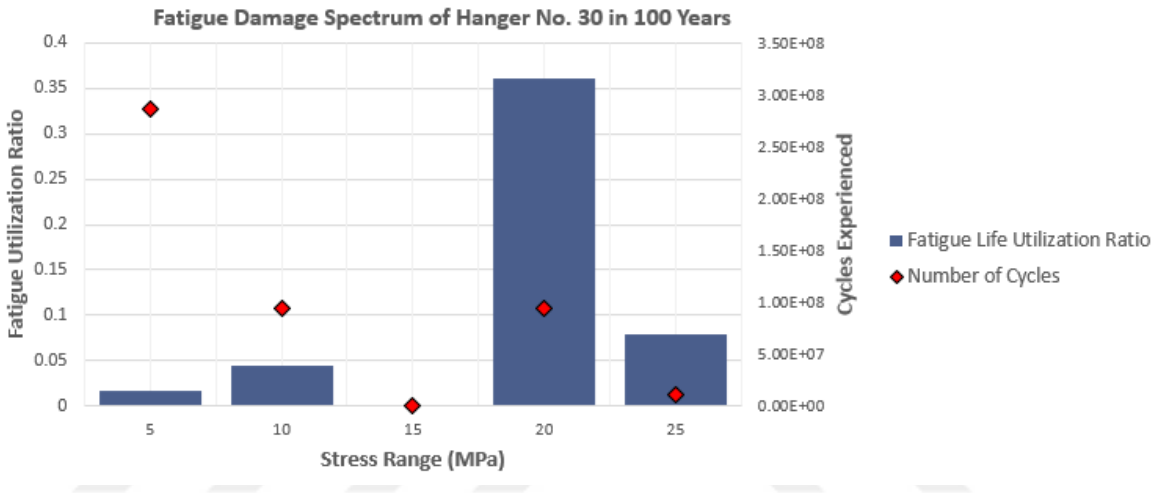


Figure 4.8. Damage Spectrum of Hanger No 30 and Corresponding Cycle Counts

As reported before, the studies show that corrosion could drastically reduce the fatigue life of the cables [22] by interacting with fretting mechanism. Since the hanger cables of the Bosphorus Bridge have no sheathing, corrosion of the strand can be expected during the service life therefore it will be meaningful to consider corrosion when calculating fatigue damage. For this purpose, same analysis will be performed but this a different S-N curve will be used which is developed on the basis of tests performed on corroded wires. The S-N curve relation is given in Equation 4.6 [22]. This relation is based on parallel wire type of strand which is not the same with the Bosphorus Bridge hangers but there is no available S-N curve for spiral strands so this will be used in the analysis. It should be kept in mind that spiral strand is expected to have a lower fatigue resistance than a parallel wire strand. The reason for this is

that helical nature of the spiral strand caused higher contact forces between wires.

$$logN = 11.729 - 2.51log\Delta\sigma \tag{4.6}$$

In Figure 4.9, the logarithmic plots of both S-N curves used in the study is shown. It is seen from the figure that the corroded wire has a steeper slope thus resulting in lower fatigue resistance especially for low stress ranges. This effect was also proved with laboratory testing in a recent study [30]. The surface notches and pits caused by corrosion increases stress concentration in within the wire results in initiation of fatigue cracks. As the degree of corrosion increases, the pits get deeper and fatigue resistance further decreases [31]. Therefore, inspection of wires is crucial to prevent premature wire breaks on the cable.

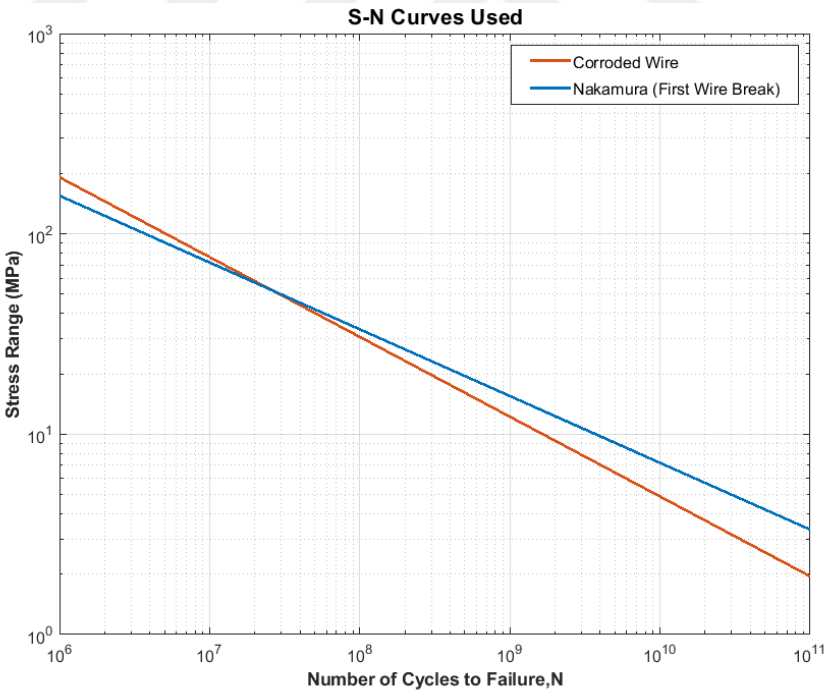


Figure 4.9. S-N Curves Used in the Analysis

In Figure 4 , some example photos of corroded wires observed in the industry are shown. The corrosion pits are easily observed on the surface of the wires.



Figure 4.10. Views of Some Corroded bridge Wires

The same stress ranges and cycles are used with an other S-N curve relation will result in a different levels of fatigue damage and fatigue lives. In Figures 4.11 and 4.12, the corresponding damage and fatigue life plots are presented. From inspecting these figures, it is seen that the damage is almost double due to the effect of corrosion on fatigue life of the strand. For the shortest hanger (No.30), the fatigue life is reduced from 200 to 134 years and for the longest hanger (No.1), fatigue life of the strand reduced from 11157 to 3738. Note that for the longest hanger which has the lowest stress range as shown in Figure 4.5, the influence of corrosion is higher than the shortest hanger which has a greater stress range.

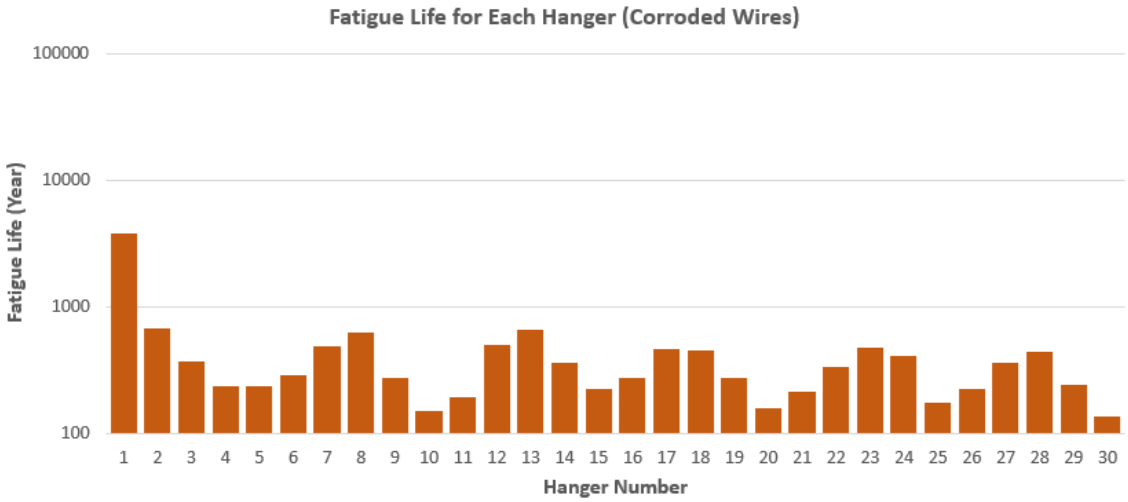


Figure 4.11. Fatigue Life Calculated for Corroded Hangers based on Traffic Simulation

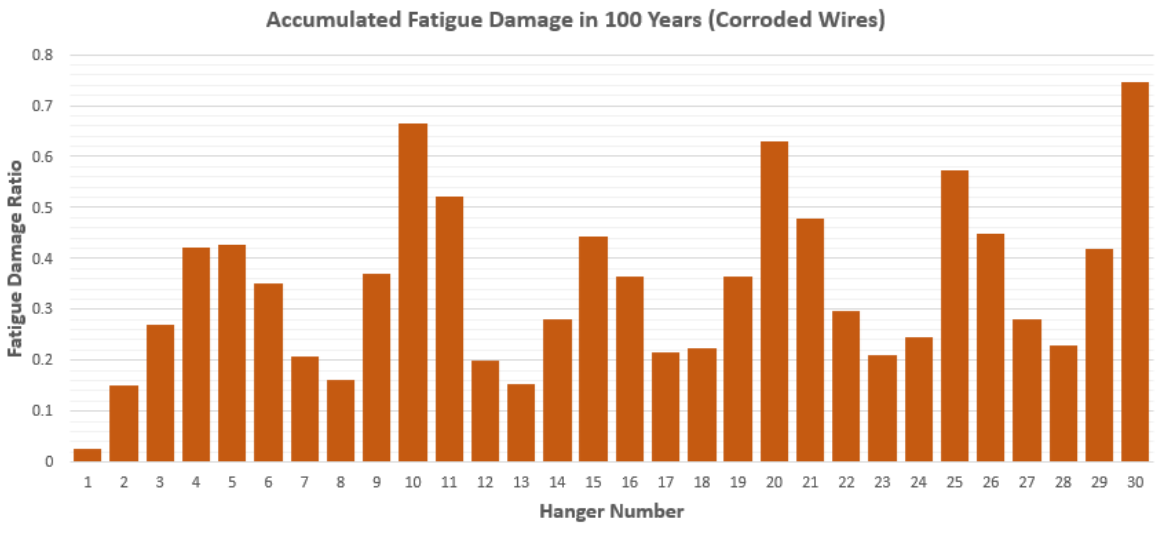


Figure 4.12. Fatigue Damage Ratio for Corroded Hangers Accumulated in 100 Years of Traffic Simulation

5. CONCLUSION

5.1. Summary and Discussion of Results

Throughout the paper, a methodology is proposed to perform a fatigue performance assessment for cables of an existing suspension bridge based on a simulated traffic flow. Hangers of suspension bridges are prone to fatigue failure under fluctuating vehicular live loads. A numerical study is attempted in order to understand the fatigue behavior of hangers and quantify the damage they suffer throughout the service life.

A refined analytical FE model of the bridge is developed as a first step which has paramount importance to capture structural response when dealing with a highly nonlinear cable supported structure. The modeling of such structure is not straightforward but rather requires a rigorous approach. The methodology for such process and principles are discussed in detail. Dynamic proper-ties of the structural model should be verified with field vibration measurements.

The traffic flow scenario is developed based on average daily traffic volumes and timetables of public transportation buses. A deterministic model had to be created since a distribution could not be fit to vehicle speed, axle weight and distance due to lack of data. This shows us the importance of structural health monitoring tools like the WIM system in this case, which can allow to evaluate numerically the risks for the bridge.

Stress analysis on the new vertical hangers is calculated based on 10 min long traffic time history analysis is obtained through the FE model. Benefiting from a Rainflow algorithm, stress reversal ranges and corresponding number of cycles are extracted from the structural response of cables. A S-N curve based on first wire break criteria is adopted as a fatigue damage model. Utilizing Miner's rule, the damage for each stress range group is summed for every hanger and accumulated damage is

obtained.

The calculated 10-min damage is than projected over a year by assuming uniform traffic flow. The fatigue damage accumulated in a hypothetical service life of 100 years is estimated together with the fatigue lives of hangers. The results show that vertical hangers do not fully utilize their fatigue life in a 100-year period. The highest fatigue life is obtained at the longest hanger near the tower and on the other hand, the shortest life of 200 years is observed at mid-span hanger. Analysis of the results also reveals that stress fluctuations can go up to 27 MPa and most of the cycles are below 10 MPa. The stress cycles below 10 MPa contributes less to fatigue damage but the cables are found to be sensitive to stress fluctuations greater than 20 MPa. This shows the effect of heavy articulated buses using the bridge and provides a valuable information when planning for public transportation vehicle types to meet for future demands. This fact also shows that the preventive measure taken by the local authority to ban the crossing of heavy trucks from the bridge was rightful and extended the service life of the bridge.

To see the effect on corrosion on fatigue life, the calculations are repeated for a S-N curve which is based on corroded wires. The results show that fatigue life of a cable can reduce to a third when the wires are corroded. This is mainly due to the stress concentration increase in the wires due to the presence of surface pits. As the degree of corrosion increase, this effect will be more significant. This shows the importance of the routine inspection and maintenance of cables to prevent early wire breaks. From fatigue point of view, the central hangers should have the priority in such efforts.

Another inference to be made from this study is that performance of the vertical hangers is much better than the previous inclined system. From literature, it is seen that similar efforts to estimate fatigue life of inclined hangers resulted in fatigue lives as low as 30 years [17].

5.2. Future Work

Based on the work presented and the conclusions drawn, some work can still be done to improve the study by following future work.

In order to numerically compare the vertical and inclined hanger orientations of the bridge under same traffic conditions, the analysis can be repeated and resulting hanger fatigue lives will be obtained. This will provide a valuable information about the effect of changing the hanger system on fatigue life. Also, the effect of traffic flow on one way off the bridge can be analyzed and compared between the two scenarios.

Instead of a deterministic traffic load model, a stochastic model can be developed where each parameter has a distribution assigned. To do that, a data set of vehicle types, crossing speeds, numbers and axle weight is needed. Since there is no WIM system installed on the bridge, alternative methods can be used such as vision based identification of vehicle types, flow and speed data with the help of an image processing algorithm by using the surveillance camera records on the bridge. This way, a distribution of the vehicle numbers, types and speed can be obtained to set up a probabilistic load model.

Since all the fatigue damage and life calculations comes down to the S-N curve adopted, using a representative one will improve the reliability of the results. In this view, a strand with exact diameter and construction of wires could be tested.

If the force time history for the hangers is obtained with a small time step (0.2s in this study), the fatigue assessment can be made directly from the records without the need of a FE model. For an existing bridge, it is not very easy to install force transducers to the cables but acceleration methods to identify the real-time tension in the cables can be exploited [29]. Vision based methods to obtain acceleration can be combined with this approach.

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