ABSTRACT

Title of Document: CABLE RELIABILITY OF CABLE-STAYED BRIDGES

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Directed By: Research Professor Chung C. Fu

Department of Civil and Environmental Engineering

Redundancy is achieved by studying critical load paths and finding alternatives to implement them so that system's function is not altered after the failure of one or several structural members. Structural Failure can be caused by the application of large live loads, sudden loss of structural elements due to fatigue, brittle fracture, or an accident due to the collision of trucks, ships or just debris. Depending on the degree of redundancy considered for the system, its reliability is greatly affected. Reliability is considered during system operation and helps greatly in monitoring system's health by applying techniques to the data collected and predicting failure of components of the system. Primarily focussed on cable-failure, James River Bridge, which is a harp-shaped cable-stayed bridge located 20 miles south-east of Richmond, Virginia, is selected in this study. A commercial bridge analysis tool CSIBridge was used for developing a linear elastic model of the bridge to analyze the effect of a moving load and recording cable response. ANSYS Mechanical APDL (ANSYS Parametric Design Language) which was further used to create a reliability framework by developing a tool to integrate with MATLAB and calculate the probability of limit state exceedance. The findings of this study will benefit engineers in understanding the behaviour and implementation of redundancy and reliability concepts to cable-stayed bridges.

CABLE-RELIABILITY OF CABLE STAYED BRIDGES

By

Shashank Ganesh

Thesis submitted to the faculty of the graduate school of the
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Advisory Committee:
Professor Chung C. Fu – Chair
Professor Amde M. Amde
Professor Brian M. Phillips

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Lastly, I would like to thank my family for the support they have provided me over the last few months of this study. Their encouragement kept me going till the end of this project.

Contents

1. Introduction

1.1. Background

Reliability is the art of formulating a mathematical model to help determine the behaviour of a system given a set of properties such as geometric dimensions, material properties etc.

Reliability is a relatively new concept in the field of structural engineering. Until the late 80's, structural reliability was only in research papers and not yet in practice. Originated as a tool in the mechanical and aerospace industries to improve productivity and reduce downtime, the reliability process also helps in taking care of defect elimination and extending the life of equipment and structures. [1]

Since in engineering we deal with problems that cannot be answered solely by theoretical consideration and solutions, it is important to understand how a system behaves in total deterministic control in order to appreciate the nature of the problem in an illustrative manner. The important question that is raised amidst all these is about safety. In the olden days, the accumulated experiences of professional builders and engineers were taken into considerations and used to make the necessary value assessment of safety margins. But it is important that the tools to assess the safety of a system should not merely be dependent on a person’s (or a professional's) experience but it must be an established process that will determine this objectively. [2]

Redundancy is defined as the availability of alternative critical load paths in the structural system. Since, structural failure can be caused by the application of large live loads or sudden loss of structural elements due to fatigue, fracture or an accident due to the collision of trucks, ships or just debris. When a structure is deemed non-redundant, the entire system will collapse due to the failure of a single critical element.

Segmental erections, prestressed and concrete box girders and cable-stayed support system are relatively recent developments in bridge technology. These techniques were not employed in bridge design and construction until the early 1970's due to their low strength-to-weight ratio and the complexities of design. Innovations over a period of time lead to design simplifications and innovative construction techniques helped speedy erection and efficient use of high-strength materials, as well as pleasing aesthetics.

Influence lines can be defined as a variation function for shear, moment, reaction and also stresses, when a unit load is moving over the structure. Usually, x is the distance and y the ordinate when analysing an influence for a beam. Influence lines are good enough only for a 1-D model. For longer span bridges i.e. cable-stayed bridges, suspension bridges, 3-D FE models are built for accurate results so that material and geometrical non-linearity are accounted for. In such cases, we employ influence surface technique, where, x and y are surface coordinates and z is the ordinate. [3]

There are various factors that make the aforementioned type of bridges difficult to analyse and design. One of the major factors is that there are several continuous spans with multiple supporting cables making these structures highly statically indeterminate. Secondly, the degree of complexity is raised by a notch by the introduction of multi-stage post-tensioning, stay cable non-linearity, connections and residual stresses, including, time-dependent phenomena such as creep and shrinkage. Lastly, mechanical and thermal stresses cannot be neglected as they also affect the behaviour of the structure to a considerable extent.

1.2. Purpose and Scope

This thesis proposes a method for evaluating the effects of cable snapping of cable-stayed bridges i.e. the influence of each cable and identifying the most critical member. Two analysis software were involved – (1) CSIBridge17 that is commercially available and in this project was used for non-linear static analysis of the bridge and to obtain influence surfaces. (2) ANSYS is used to develop a finite element model of the bridge to understanding the damage due to the absence of cables and redistribution of forces on the bridge. Normal probability distributions characterize the parameters of cable snapping scenarios and applied to the bridge model and the distributions also include the uncertainties in the consequences. For random variable sampling, the Latin Hypercube method (LHS) was used as opposed to the traditional Monte Carlo sampling, and the failure probability and reliability index was simulated using Direct Monte Carlo Simulation (DMCS) LHS is a method of stratified sampling used on multiple variables and used to reduce the number of runs necessary for a Monte Carlo simulation to achieved a reasonably accurate random distribution. (https://mathieu.fenniak.net/latin-hypercube-sampling/). From the simulated data, the reliability index and failure probability were then calculated.

The finding of this study will benefit bridge engineers on understanding the behaviour and implementation of redundancy and reliability concepts to cable-stayed bridges. This process can be extended to other components of the bridge or even any structural system as a tool for structural engineers in general so that predictive analysis can be performed on various structures to understand various failure scenarios and their economic effects. Reliability index is a measure of mean margin of safety in the units of standard deviation (β). Failure probability of the structural system can be written as: $P_f = 1 - \Phi(\beta)$; where Φ is cumulative distributive function of the standard normal variate. [4]

1.3. Document Organisation

This thesis document has nine chapters and an appendix. Chapter 1 gives some background on cable stayed bridges and the importance of incorporating cable failure during bridge designs. It also introduces the concept of reliability and, reliability and redundancy in structures along scope and the purpose of this study. Chapter 2 contains a literature review, which provides an overview of works that has been over the last few decades. This includes papers that discuss reliability as an established domain in mechanical and aerospace engineering and how it can be implemented in the structural engineering; along with some implemented methods in this area, implementation of finite element methods in cable stayed bridges and model updating, Chapter 3 and Chapter 4 discuss the structural specifications, boundary conditions, analysis methods, software used and case definition. Chapter 5 covers different cases in which cable influence is studied as part of this case study. Chapter 6 is an extension of chapter 5 where we complete the dynamic analysis of the structure as per the information obtained from cable influence diagrams. In chapter 7, Monte Carlo simulation is done on a performance function dictating the dynamic load on the cables. Chapters 8 and 9 are about the interpretation of results, conclusion and scope of extending this study.

2. Literature Review

2.1. Reliability and Redundancy

Ghosn et al. (2016) discussed that the member-oriented approach to the design may not lead to the efficient utilization of resources while reliability assessment procedures are in place due to the management of existing deteriorating structures. Therefore, the system-level characteristics, such as reliability, redundancy, robustness, resilience and risk, are stressed upon, refined and extended to establish a performance-based and reliability-based design that is relevant for assessing and managing system-level risk. Structural redundancies and robustness are also discussed as specific examples. [2]

Michael Bruneau (1992) published an introductory paper in the domain of structural reliability in the ASCE journal that was in the year 1992 deducing that failure of individual members does not necessarily lead to structural collapse or the complete loss of system functionality. It is one of the initial papers that discussed the implementation of probabilistic system reliability methods, especially the first-order second moment (FORM) method, leading to more economical, reliable and rational structures. Linear approximations of the performance function have been used as the most likely failure point. The study was divided into two different parts – (1) ductile cable systems categorized as a series system whose structural system failure will only be caused due to formation of a collapse plastic mechanism, and (2) brittle cable systems, which are a mix of parallel and series systems. This is because even after the removal of material non-linearity under pre-stretching operations the stress-strain diagram shows a genuine plastic curve. But the plastic strains of the cable steel may not be enough for the plastic collapse mechanism to dominate the failure mode of the entire structure to which is why mixed system was necessary for their analysis. Reliability has been measured using the safety index, β, from which probability of failure can be calculated. [5]

Feng Jian and Cai Jianguo (2009) published a paper about progressive collapse of major bridges based on their performance. The authors flag this is an important aspect of bridge design post 9/11 era and discuss whether existing design standards can be applied to cable-stayed bridges. The paper illustrates two cases – The first is based on an incident involving a school bus and two trucks on Mezcala bridge in Mexico when one of the cables caught fire whose sheathing was poorly resisted to suit fire. This affected the integrity of the bridge leading to its closure until deemed safe for operation and in hindsight; multiple cable at the same could have undergone the same fate. The second example is based on an incident on the Quingzhou Min Jiang Bridge, China where a 1000-ton crane brought down 3 cables. Very little damage was found occurred to the strands of the cables. However, the entire cables were replaced due to uncertainty of varying forces and potential of the cable strands. The authors further discuss risk mitigation strategies, for abnormal load combinations, recommend the following equation:

$$P[C] = P[C|LD]\,P[LD|H]\,P[H] \qquad (2.1)$$

P[C] is the probability of structural collapse; P[C|LD] is the probability of collapse provided there is local damage; P[LD|H] is the local damage given there is a hazard and P[H] is the probability of hazard. The focus is on limiting P[C|LD] to control P[C] in order to develop an alternate load-path design strategy.

The research paper follows the PTI recommendations which a provision for a dynamic force to compensate the snapping of the cable:

$$1.2DC + 1.4DW + 0.75(LL + M) + CLDF \qquad (2.2)$$

where, DC = Dead load of structural components; DW = Dead load of wearing surfaces and utilities; LL + IM = full vehicular live load placed in actual striped lanes and dynamic load allowance; CLDF = Impact force due to cable failure. The authors' discuss having a reservation on the PTI recommendation stating it considers only the loss of a single cable and not multiple ones. Given that due to technological advances and the use of multiple cable strands for the modern stay-cable makes it internally redundant and therefore, multiple cable loss is an extreme condition, intentional attack scenarios like terrorist attacks or high velocity vehicle impact can prove otherwise. [6]

Li, H et al (2010) introduced the limit state function for evaluating the failure probability. The limit state function was represented as a stochastic performance function:

$$Z\,(\tau) \;=\; g[R(\tau), S(\tau)] \;=\; R(\tau) \;-\; S(\tau) \qquad (2.3)$$

where $R(\tau)$ & $S(\tau)$ are the stochastic processes of resistance and load effects (strength) subsequently, leading to the evaluation of the probability of failure P_f. This is done by implementing FORM (First Order Reliability Method) and SORM (Second Order Reliability Method).

This reliability estimation method was employed on the Tianjin Yonghe Bridge where strain gauges were installed at multiple locations – near cracks on the concrete-box girder and corroded stay supports of the stay cable. A 3-D FEM model was established on ANSYS based on the engineering drawings that consisted of BEAM44 element type for unsymmetrical tapered beam, LINK10 element for tension-only/compression-only elements, MASS21 and COMBIN14 for structural mass and spring-damper elements respectively. The bridge and structural parameters were updated based on the modal parameters cable forces obtained from the strain sensors.[7]

Han, Sung Ho (2011) utilized the Taylor series expansion to define the stability functions to numerically stabilize structural and material property variables of the bridge members. The author then discusses about two methods – Direct Monte Carlo Simulation (DMCS) and a Stochastic Finite Element Program (SFEAP), each of which computed the probability of failure and reliability index by incrementing the load step and running probability structural analysis. An analytical model was initially developed to perform the shape analysis of the bridge with multiple load schemes that involved the cable tensile force, dead load and evenly distributed lived load with different percentage correlation, since these have a significant effect on the structural response of the bridge. [8]

Biodini, F. et al (2001), published a paper on the use of fuzzy criterion to perform reliability assessment. The authors also stress on the fact that due to numerous uncertainties in the geometrical and mechanical properties defining the bridge, the problem cannot be deterministic. They define seven limit states – due to the strains of concrete, reinforcement steel and prestressing steel, and collapse of the cross-section that occurs when the strains reach their ultimate state. A non-linear analysis is performed by considering material non-linearity and geometric non-linearity. Based on the results, due to high degree of non-linearity, it is imperative to have the right sample size and also the method of sampling. [9]

Zheng, X. et al (2018) worked on the redundancy analysis of cable-tied arch bridge systems. As discussed earlier in Chapter 1 that redundancy is having alternative load paths in place during the failure of one or more structural components, this paper delves into more detailed methodologies of using the limit state functions to determine the degree of redundancy of the structure. The AASHTO LRFD specification suggested including modifiers that could reflect ductility, redundancy and other properties to ensure smooth operation of the bridge. The limit states considered were

1. Member failure (LF_1): when the first member fails while incrementing the load in addition to dead load and 2 HS-20 trucks.
2. Ultimate limit state (LF_u): the bridge is subjected to serious damage (or the bridge collapses), simulated by incrementing the load on a non-linear structural model in addition to the standard dead load and 2 HS-20 trucks.

3. Functionality limit state (LF_f): capacity of a structure to resist a live load displacement in a main longitudinal member equal to the span length/100.

According to the NCHRP (1998) report on ‘Redundancy of highway superstructures’, on the basis of the above limit states, redundancy indices and reliability indices were defined. This step-by-step procedure was performed on the Jinghang Canal Bridge in Jiangsu, China, which is box girder, arch tied bridge. It was concluded that, the steel box resists large deformation and approaches functionality limit state after member failures. [10]

Zhou, Y. et al. (2016) developed a reliability framework for long-span cable stayed bridge to study cable breakage events. This is a very holistic study which considers multiple random variables for input, ranging from structural and geometrical parameters to a multitude of traffic flow parameters. Non-linear dynamic analysis was performed to for different load cases to obtain information on the condition of the cable i.e. loss of cable material, reduction in cross-section and whether the cable material enters the inelastic range and they will be eliminated from the FE model if they meet the failure criterion. The ultimate limit states for failure are considered to be the same as what Zheng, X. et al [10] had considered. Non-linear dynamic analysis was performed in each experiment for simulating cable-breakage, the sources of non-linearity and dynamic coupling effects from traffic and wind are considered. Finally, a fragility analysis was conducted for cable-breakage events in ultimate limit states and serviceability limit states. It was concluded that the failure of the cables leading the structural instability of the structure with plastic hinge formation on pylon elements is the dominant mode of failure. Secondly, traffic and wind excitation may increase the probability of failure of the bridge. [11]

2.2. Bridge inspection and model updating

Hua, Ni, Chen and Ko (2009) discussed detecting bridge damage by measuring cable force. The damage identification techniques, being the key topic researched in the paper, was formulated as an optimization problem where the structural parameters were obtained by using direct differentiation and finite differentiation methods. System identification is about FE model updating or parametric identification. [12]

Raftoyiannis, Konstantakopoulos and Michaltsos (2013) describe cable-stayed bridge as an alternative to suspension bridge for longer spans. The analyses conducted on the bridge in question here, is very similar to the one being discussed as part of this thesis i.e. a single line of cables anchored along the central axis of the deck's cross-section. The paper specifically discusses about the dynamic response of a cable-stayed bridge due to the failure of stay. According to the authors, different factors leading to the failure of cables are – corrosion, continuous friction or abrasion, progressive and extended crevice created by fatigue. [1]

Cai et al (2012) claimed that progressive collapse is dynamic event that is initiated by a localized structural damage and creates a chain reaction. Progressive collapse scenario is generally overlooked in case of bridges because it is assumed that structural efficiencies are better due to the presence of a single major axis. Therefore, this has never been a major consideration during bridge design. The primary aim of this article is to discuss the effect on the bridge due to the failure of a single cable. Since, this is categorized as a dynamic event; it is considered that along with the failure of a single cable some of the adjacent cables will also rupture due to overloading. The authors then consider one failure mechanism i.e. loss of one cable and adjacent cables; in four analysis techniques, (1) linear static (2) non-linear static (3) linear dynamic (4) nonlinear dynamic. The analyses are performed on SAP2000, which is commercially available structural analysis software that is developed by CSI Inc. SAP2000 provides the user the ability to place plastic hinge on the structural model for non-linear analysis. The Post-Tensioning Institute published a set of recommendations in 2001 that considered different scenarios that are not included in the conventional design of bridges [13] and it dictates a dynamic amplification factor for the linear static and linear dynamic analyses. But the non-linear static and non-linear dynamic analyses were completed taking large displacement and material non-linearity into consideration. The demand to capacity ratio (DCR), which is the ratio of the axial force to the ultimate capacity of the cable, was used to compare results of the four techniques. The results of the project suggested that the nodal displacement decreases as the lost cables are near the pylon. The article then concludes by stressing on the importance of considering cable failure due to various circumstances (accidents and other dynamic events) that can lead to progressive collapse as a potential mode of bridge failure. [6]

Ren, Peng and Lin (2005) performed a modal analysis on the Qingzhou Bridge, China to understand the dynamic characteristics of the structure. They completed analytical and experimental modal analysis of the bridge, the results of which were compared and collaborated to establish a comprehensive investigation. ANSYS APDL was to develop the Finite Element (FE) model of the bridge. The different elements used in the ANSYS model were:

(1) BEAM4 – used for beams and girders; allows for tension, compression, torsion and bending capabilities.

(2) LINK10 – is a 3D element that has a unique bilinear stiffness matrix that results in tension only element; this element is used for the stay cables.

(3) SHELL63 – is used for the concrete slab; possess both bending and membrane capabilities and both in-plane and normal loads are permitted;

(4) SOLID45 – assigned to piers and platforms in the model, is primarily used for the three-dimensional modelling of solid structures. The physical properties include plasticity, creep, swelling, stress stiffening and large strain capabilities.

The experimental setup provided accurate and reliable information on the global parameters of the structure. The ambient vibration tests were carried out on the opening day of the bridge with the help of accelerometers cables and a signal amplifier. The modal parameters were identified using the stochastic subspace identification because simple peak picking method (on the signal amplifier) cannot identify all mode shapes for such a large bridge. The correlation between the analytical and experimental results was defined in the form of MAC which is expressed as:

$$\mathrm{MAC}_{jk} = \frac{|\phi_{mj}^{T}\phi_{ak}|^{2}}{(\phi_{ak}^{T}\phi_{ak})(\phi_{mj}^{T}\phi_{mj})}. \tag{2.4}$$

Higher values of MAC, better the correlation between the identified and calculated mode shapes except the anti-symmetric bending modes which are associated with the lower MAC values. [14]

The National Cooperative Highway Research Program (NCHRP, 1998) issued a report titled ‘Redundancy in Highway Bridge Superstructures’ which discusses the importance of redundancies in bridge structures. It describes redundancy as a function of types, geometry and ductility and comments on the existing procedures, about them considering only the effect of an individual element to the system and not the system as an entirety, e.g., the failure of a member may not lead to the failure of the entire system but the failure of the entire system might prove it to be inadequate for truck traffic. It is important to consider the system reserve ratios (R) to be able to measure structural redundancy. When R=1.0, then the ultimate capacity of the structural system is equivalent to the ability of structure to resist the failure of its most critical member, deeming it non-redundant. As the value R is greater than 1.0 and increasing, the redundancy also increases. [15]

3. Structural Specification and Case Definition

This thesis stresses the importance of cable reliability and redundancy to give bridge engineers idea of structural behaviour during the failure of one or more cables. As explained in the earlier chapters, the reliability of cables is overseen with respect to the rest of cable-stayed bridge structure which in retrospect, face similar risks of getting damaged and causing concerns to the structural integrity to the structure. A harp-shaped cable-stayed bridge was adopted for studying cable behaviour based on a report published by the Virginia DOT. There are some structural details missing from the report which is also mentioned here which has led us to make some assumptions but goes into a detailed description in the modelling section, Chapter 4.

3.1. Structural Dimensions

The case study is based on a parallel cable-stayed box girder, precast, segmentally erected, a post-tensioned bridge that carries Interstate 295 over James River. The bridge is approximately located about 15 miles southeast of Richmond, Virginia.

The following is taken directly from V-DOT report:

There are 28 individual spans, including approach spans. The focus was mainly on a 7-span continuous section that includes the main span of 630ft long and three approach spans of 150ft on either end of the main span. The spans are supported by a system of 52 cable stays, held on by two 290-ft pylons located on either side of the river. The bridge is composed of 2 parallel girders joined by a closure pour along the central line of the structure. The forces from the cable stays are transferred to the twin box girders through a series of precast delta frame assemblies located between the girders at each stay location. The main span of the bridge was constructed by the cantilever method with a closure pour at mid-span. The segments are joined together using epoxy cement and post-tensioned strands. The box girders are externally post-tensioned by a system of multiple tendons anchored within the deck segments.

Completed in 1990, this bridge was the first cable-stayed bridge in Virginia. The material required in the box girders required was less than what was required for a single box girder of the same width, which was quite innovative at the time which resulted in the significant reduction of special construction equipment. [16]

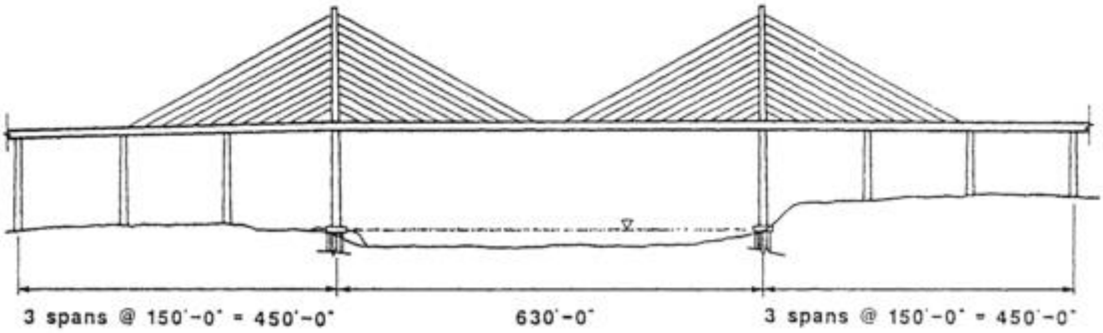

Figure 3-1 Elevation of the I-295 Bridge[16]

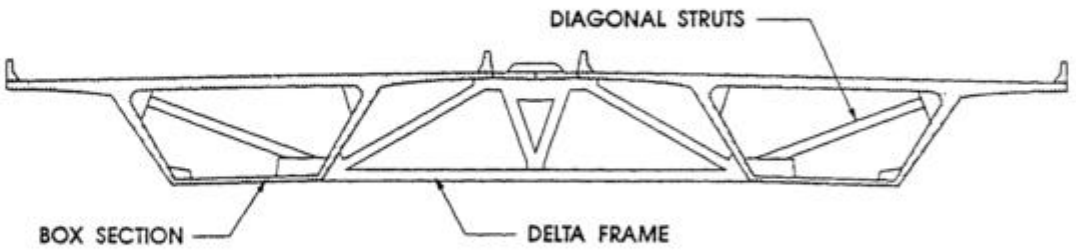

Figure 3-2 Girder cross-Section of the I-295 Bridge at the cable supports [16]

The report consisted of minimal information about the cables, cross-section of the pylons (Figure 3-1) and cross-section of the girder (Figure 3-2) - especially about the delta frame and diagonal struts dimensions. The report suggests that the pylon is a cast-in-place (CIP) as shown in Figure 3-1. Therefore, to complete the model on both CSIBridge and ANSYS, there were some assumptions made. The objective of this project is to study cable response, identify critical cable members and analyze them under factored load. Subsequently, the probability of limit state exceedance was evaluated. [16]

3.2. Model Definition and Proposed Methodology

This thesis proposes to conduct a reliability analysis and a redundancy study of a cable-stayed bridge during the event of a cable-failure. The flowchart depicted in *Figure 3-3* illustrates the step-by-step process undertaken in course of this study. Each step is discussed in further detail in Chapters 4-7 where we go deeper in bridge modelling, assumptions and the behaviour of the bridge during different conditions.

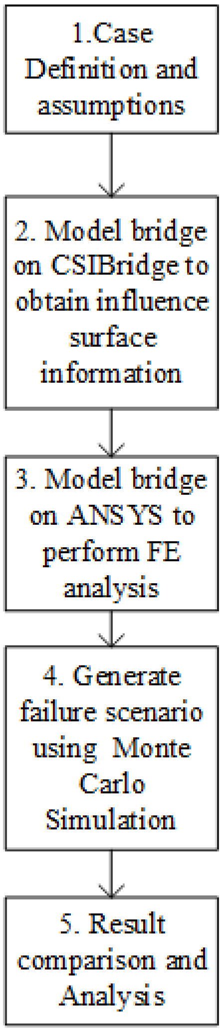

Figure 3-3 Flowchart to illustrate the process flow of this project

We started with the modelling of the proposed James River Bridge on CSIBridge. The purpose of this was to perform the structural analysis of the structure. The challenge here is to get the model accurate to that of the real structure, especially the cross-sectional dimensions of the deck. After the model was completed, the static analysis was completed only due to, the dead load of the structure to calculate the natural frequency of the structure. With this, we can check the closeness of the model to that of the real structure. This is followed by a live load analysis from which we obtain the information of the cables.

The second step involves performing a finite element analysis of the structure. Since CSI bridge gives minimum freedom to modify pre-processing and meshing of the model, in this step we use Mechanical APDL. APDL helped us to analyse the structure and look at the redistribution of forces, stress concentrations and to assess the capacity of the structure in the event of a cable snapping incident.

4. Influence of cables

The James River Bridge model was developed on CSIbridge to understand the cable influence of the bridge. The first model was developed close to the original design of the bridge. Modal analysis was performed following which a non-linear static analysis was performed.

This chapter will discuss the CSIBridge model in detail, assumptions considered and influence of cables.

4.1. CSI bridge model

CSIBridge is a specialized commercially available bridge analysis and design software package tailored for the engineering of bridge systems. This software is developed by Computers and Structures, Inc (CSI) that provides software tools for structure and earthquake engineering globally. It offers powerful capabilities for the design of concrete and steel bridges. Advanced features for modelling and analysis techniques include dynamic effects, inelastic behaviours and geometric non-linearity. Some pre-defined templates in the tool can help users right from bridge model definition through design optimization and output report generation. (https://www.csiamerica.com/about)

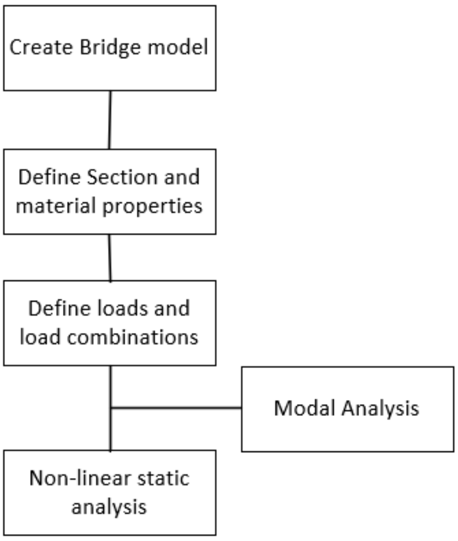

Figure 4-1 Process flow in Chapter - 4

4.2. Model Construction

Instead of developing a full-scale 3-D cable-stayed bridge model, bridges with a central frame, it is adequate for them to be analysed under vertical dead load and live traffic load. Therefore, to obtain a cable-stay forces and axial, shear forces and bending moments are calculated.

1. Girders

Three cross-sections were initialized to replicate the original girder cross-section as shown in Figure 3-2. The girder is 100-ft wide and divided into three parts, right deck, middle deck and the left deck. Apart from the overhang dimensions, the right and the left decks are just mirror images with the girder depth (12ft), slab thickness (8in-10in) where cell-wall thickness (8in) stays the same.

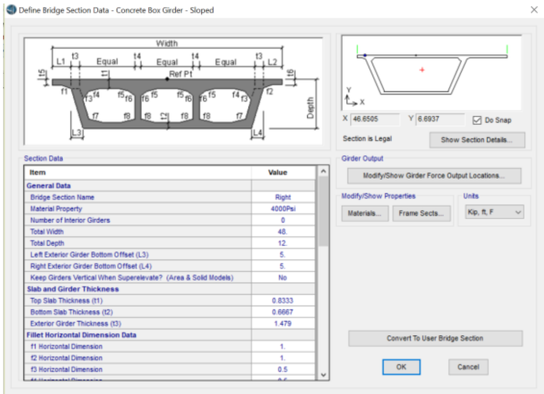

Figure 4-2 Right side girder

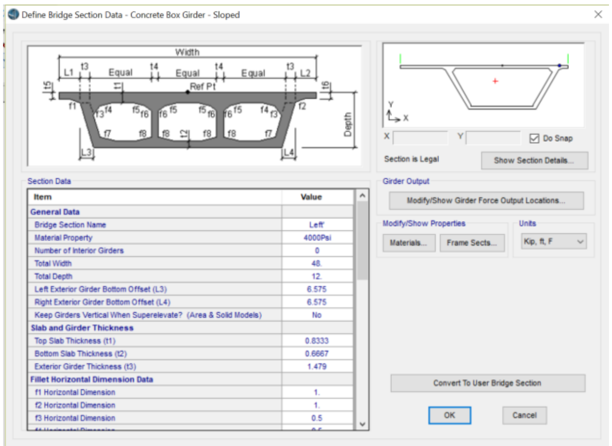

Figure 4-3 Left side girder

The right and left side girders are connected at the centre line with a middle slab which is 4ft wide and 0.8ft thick. This part of the deck signifies the central frame of the bridge along which the cables are tied and also as a separator of the two lanes.

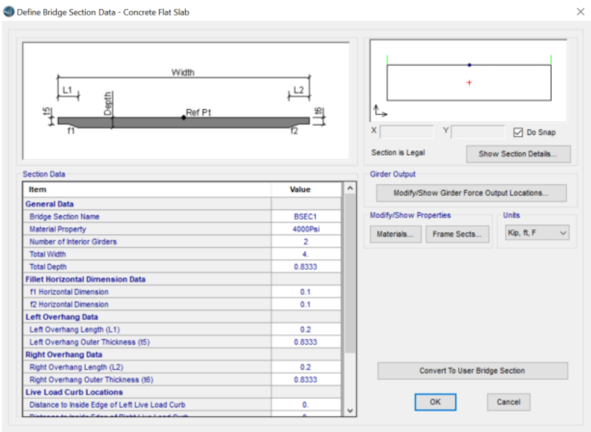

Figure 4-4 Middle deck

<u>Assumptions</u>: As shown in Figure 3-2, the diaphragms and stiffeners in the girder cross-sections are considered in the CSIBridge model. These components would more useful to be detailed during non-linear dynamic analyses studying the effects of transverse forces especially due to wind and earthquake.

2. Pylons

The dimensions of the pylons are taken to be of cross-section 8ft x 8ft. The original bridge is designed to have a CIP (cast-in-place) below the deck and further stacked with concrete segment up to a height of 208ft, hence the cross-sectional dimensions of the columns is an **assumption** for this model. On a trial and error basis, in order to match the fundamental frequency of this model on Mechanical APDL, 8ft x 8ft was deemed to satisfy requirements.

3. Cables

Cables were originally modelled as ASTM A722, which are parallel-bar cables. These comprise of a set of steel rods/bars (16mm-36mm) in a metal duct held by polyethylene spacers. Threaded couplers are used to connect multiple segments.

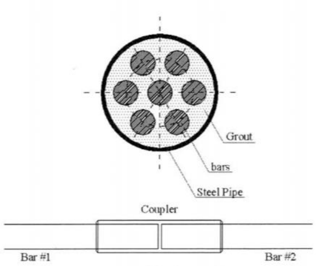

Figure 4-5 Parallel-wire cable (ASTM A722)

In theory, the cables used in cable-stayed bridges are similar to that used in the post-tensioning of concrete girders [17]. But in CSIBridge, there needs to be an assignment of tension and compression limits because the cables are a tension-only member. Therefore, the cables are initialized as a frame member to which the compression limit is as assigned as 0 and the tension limit assigned as a positive quantity.

4.3. Loads and Load combinations

According to AASHTO LRFD specifications, vehicular live loads are denoted by HL-93 where HL means highway loading and 93 is the year in which this convention was adopted. The three categories of loading defined under HL-93 are:

1. Design Truck

The design truck has three axles. As illustrated by the figure, the first axle has a loading of 8 kips, the second and third axles have 32 kips. The first and the second axles are spaced at 14 ft and the second and third axles are separated by a distance that varies between 14 ft and 30 ft. The axle spacing considered by CSIBridge is where the maximum loading occurs. [18]

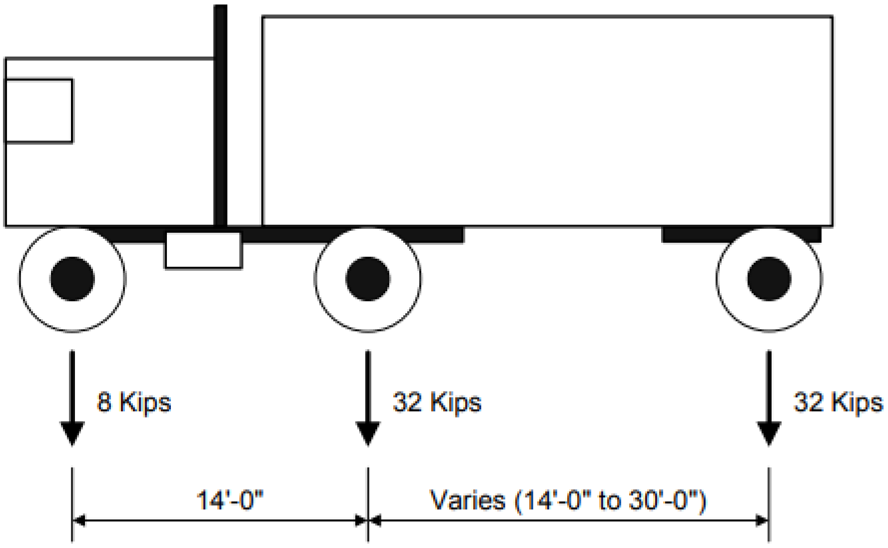

Figure 4-6 AASHTO LRFD Design truck specification

2. Design Tandem

   The design tandem loading type has two axles with 25 kips each spaced at a distance of 4 ft. [18]

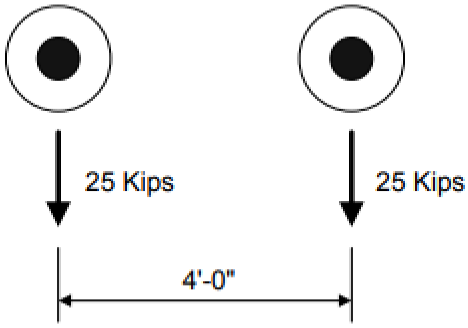

Figure 4-7 AASHTO LRFD Design tandem specification

3. Design Lane Load

The lane loading as per AASHTO LRFD code is a uniform load of 0.64 kips per linear foot distributed in the longitudinal direction. While evaluating, the lane loading applies to only the part of the span where the forces and moments are being calculated e.g. while calculating the forces and moment for the centre span, the lane loading is applied on the centre span and not on the side spans. This is how CSIBridge evaluates as well. [18]

The bridge model in the project has the considered two lanes and a design truck for analysis as shown in *Figure 4-8*.

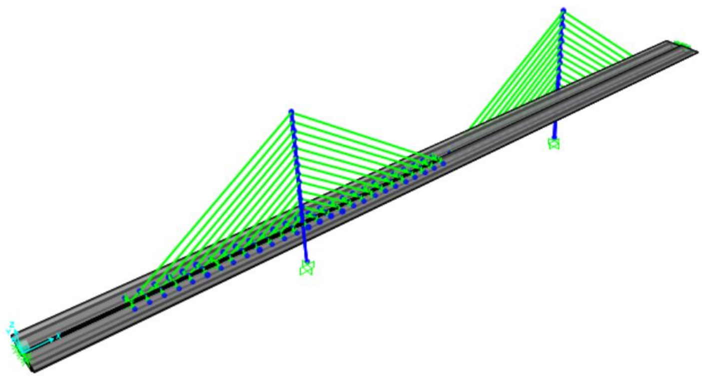

Figure 4-8 CSIBridge model showing two lanes

The list of standard vehicles according to AASHTO LRFD is described in *Table 4-1*

Vehicle Class	Description
HL93-K	Regular truck plus lane load.
HL93-S	Double truck plus lane load for negative superstructure moments and reactions at interior piers.
HL93-M	Military vehicle (tandem) plus lane load
HL93-F	Fatigue truck intended for fatigue loading, the magnitude and configuration of which is based on AASHTO LRFD 2007, Article 3.6.1.4.1.

Table 4-1 AASHTO LRFD Standard vehicle list

4.4. Modal Analysis

The fundamental frequency of the bridge coming out of modal analysis is 0.40421 Hz, shown in Figure 4-9. Multiple models were generated by removing a single-cable, to check integrity of the bridge while creating a cable-snapping scenario. Since, each cable-system has 26 cables on each of the two pylons, due to symmetry, it is sufficient for us to generate only 26 such scenarios.

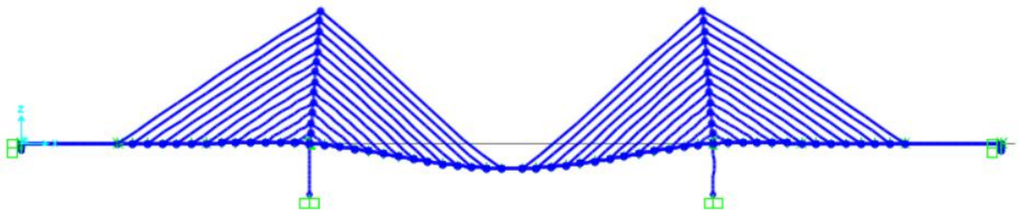

Figure 4-9 Fundamental modal shape

The fundamental frequencies in all 26 scenarios were nearly the same with the average frequency being 0.40313Hz and standard deviation being 0.0008. Figure 4-10 illustrates the cable numbers for reference. Total tensile force in the cables include the dead load and moving load.

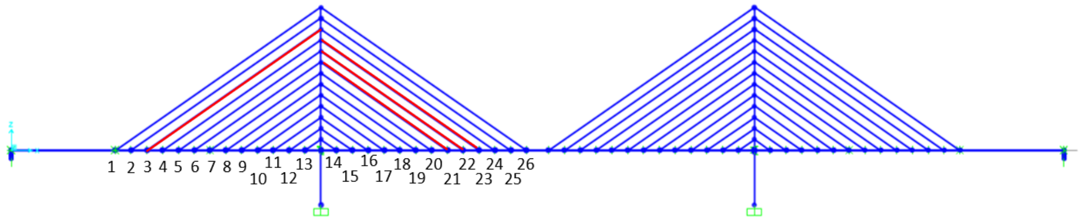

Figure 4-10 Diagram showing cable numbering

Cable-Snap	Natural Frequencies
Undeformed	0.40421
Cable-1	0.40119
Cable-2	0.40419
Cable-3	0.40174
Cable-4	0.40195
Cable-5	0.40216
Cable-6	0.40238
Cable-7	0.40265
Cable-8	0.40298
Cable-9	0.40333
Cable-10	0.40336
Cable-11	0.40392
Cable-12	0.40410
Cable-13	0.40419
Cable-14	0.40420
Cable-15	0.40412
Cable-16	0.40397
Cable-17	0.40375
Cable-18	0.40348
Cable-19	0.4032
Cable-20	0.40295
Cable-21	0.40274
Cable-22	0.40262
Cable-23	0.40259
Cable-24	0.40266
Cable-25	0.40282
Cable-26	0.40306

Table 4-2 Fundamental Frequencies when a single cable was snapped

4.5. Moving load analysis

After the modal analysis, a moving load was performed with a vehicular live load. CSIBridge is able to perform a moving load analysis only with the HL-93F vehicle class.

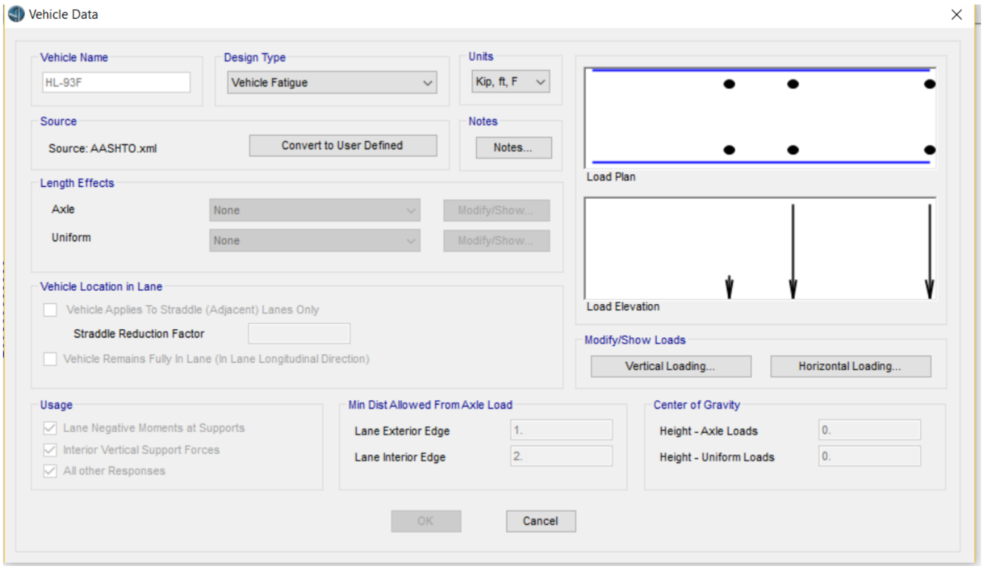

Figure 4-11 HL-93F vehicle class load description

A moving load case is created on the analysis tool, where the type of analysis is set to non-linear, solution type is set to direct integration and not considering any geometric non-linearities. The number of output steps is set to 300 and output time size is set to 0.1 where a damping ratio of 0.5 is considered for all the calculated modal frequencies. With 300 time-steps, the total time taken by the vehicle to cross the entire length of the bridge is 30 secs (300 x 0.1). Therefore, the speed of the vehicle is 51 ft/sec or 34.7 mph.

4.6. Results

After the moving load analysis, cable response during the time taken for the truck to cross the bridge is recorded and illustrated in *Figure 4-12*. The x-axis denotes the time taken for the vehicle to move across the bridge and y-axis shows the force due to the moving load in the cable.

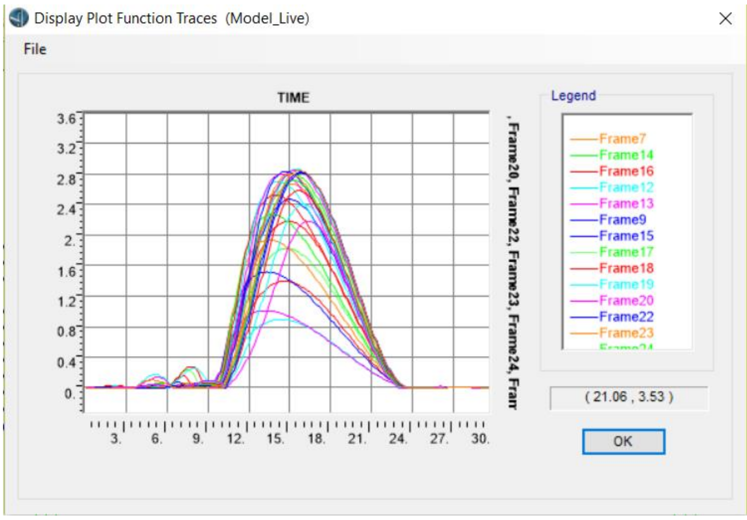

Figure 4-12 Time response of a single cable-system due to the moving load

Dead load analysis showed that the end cables, cable 1 and cable 26 have 286 kips and 236 kips of maximum tension force respectively. It is observed that, the cable forces in the centre cables are more than that of the end cables.

The graph in Figure 4-13 illustrates the typical response of a cable during a moving load analysis. The x-axis shows the time, which can be interpreted with the position of the bridge and the y-axis shows total tensile force on the cable. If the peak is attained at t = 14 secs, which means that the vehicle was at 714 ft from the starting of the bridge.

Cable	Cable-force, kips		Peak attained, ft
	Dead Load	Total Tensile Force (max)	
Cable-1	286	288	841.5
Cable-26	236	239	841.5
Cable-3	294.03	296.7	765
Cable-21	334.6	337.5	714
Cable-22	328.03	331	739.5
Cable-23	314.34	317.2	790.5

Table 4-3 Cable forces summary

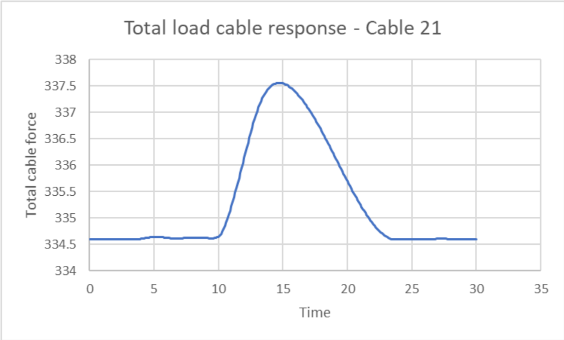

Figure 4-13 Graph depicting typical response of a cable under total load

It can be seen from *Table 4-3* that the cables in the centre-span system are in more tension than the cables in the end-span system. Cables 21, 22 and 23 were found to have the more tensile force than any other cable in the entire single pylon system. Their peaks were attained when the vehicle was at 714ft, 739 ft and 790 ft resp. Therefore, from this analysis it can be concluded that these three cables are the most critical and their snapping would lead to the re-distribution of forces. Further analysis would consider only these three cables.

5. Finite Element Analysis

5.1. Introduction

ANSYS mechanical APDL is a commercial tool that is used for the construction and analysis of finite element models. It is also easier to update/modify models as per requirements and run the analysis in batch runs.

Instead of developing a full scale 3-D model with solid elements for girders and pylons, it is sufficient for us to create a skeleton model with only the cable planes. Bridges with multiple cable-planes will have to be modelled in 3-D to indicate the skew in the geometry and forces. The bridge involved in this study has only a single cable frame along the central axis. It is adequate to perform the analyses under vertical dead load and traffic load. Cable-stay forces, axial forces, shear forces and bending moments are calculated.

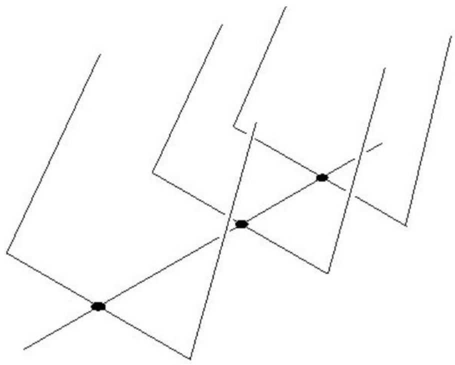

Figure 5-1 Example of a double-cable plane to a box girder [3]

The number of longitudinal beam elements denotes the number of girders/boxes in the bridge. Transverse elements are created to integrate the multiple planes (either for girder or cable planes) in the bridge model. This type of bridge model is called a skeleton model or the fish-bone model as it resembles shape of the model resembling that of a fish bone. The longitudinal element denotes a girder or a box of a girder. In modelling terms, for a bridge this is called a grillage. The transverse elements denote the cross-beams or the diaphragms. [3]

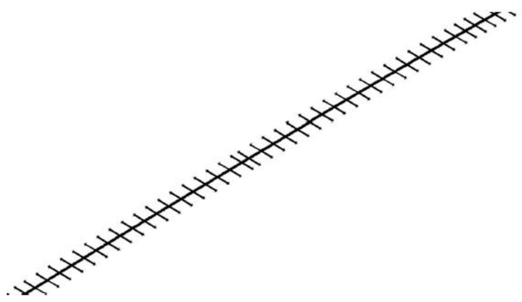

Figure 5-2 An example of a Fish-bone model [3]

Defining section properties and material constants for the elements is part of the bridge model creation process. The elements are then divided and meshed before the meshing. The boundary conditions are applied and independent DOFs are coupled. Figure 5-3 illustrates the process flow involved in this section.

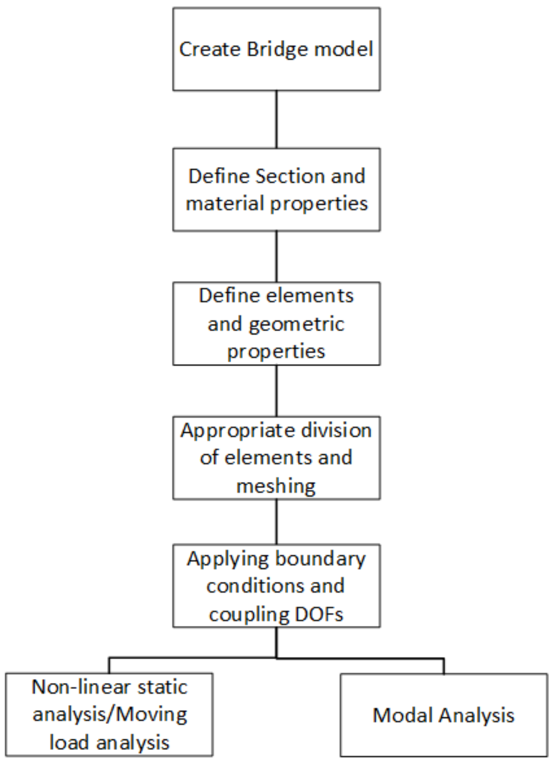

Figure 5-3 Process flow to analyse a bridge model on ANSYS

5.2. Elements defined in the model and constants used.

The James River Bridge model is created by writing a set of instructions in form of a code instead of using the graphical user interface (GUI). APDL reads code in the standard FORTRAN format that can be written on a text file and imported to APDL. Figure 5-4 illustrates bridge model in ANSYS Mechanical APDL. The central longitudinal element of the model represents the axis for the central frame which is also along the cable plane and the other two elements along the length of the bridge depict the two cells of the box girder.

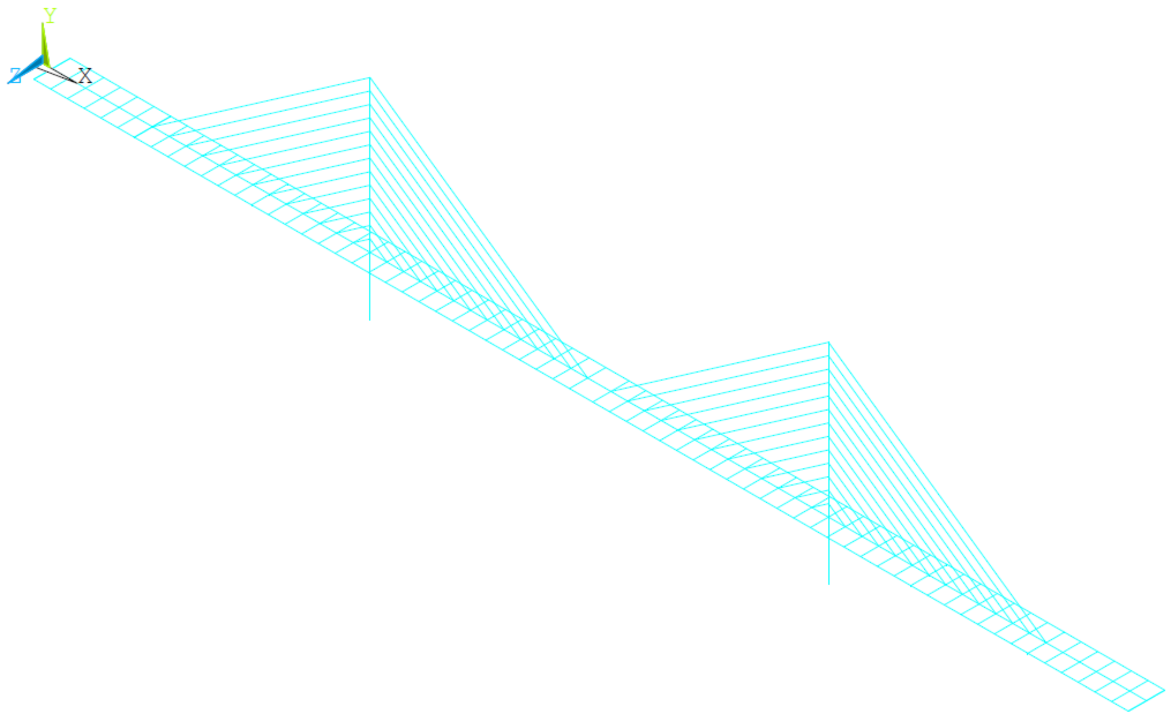

Figure 5-4 Model layout of bridge on ANSYS APDL

The following elements were used in the FE model:

1. **Link180**: This element type is used for the modelling of the cables stays in the bridge. . This element is typically used in the modelling of tension and compression members but a non-linear iterative solution procedure is necessary for these effects to show in the simulation. In this study, Link180 is used only as a tension-only member. The command

only requires for us to input the area of cross-section of the cable apart from the material properties.

There is difference between the link members in CSIBridge and in Mechanical APDL. The CSIBridge links have bending which is why they were used as transverse elements along the girder (or cross beams) in the model. In APDL, link elements can face only axial force and cannot bend out plane. Therefore, the transverse elements in the FE-model are modelled as BEAM188 elements.

2. **Beam188**: is suitable for the analysis of beam structures based on the Timoshenko beam theory which includes shear-deformation effects and allows slender to moderately thick beams to be analysed. It is a two-node beam in 3-D having linear, quadratic or cubic capabilities but for the purpose of this study only the linear part has been used. SECD and SECT commands follow after the initialization of elements for assignment of the type of section and section properties. In this case, for the girder, values of the section properties were obtained from the CSIBridge model for ASEC (arbitrary cross-section).

The transverse members are also modelled as BEAM188 for them to be able undergo bending and but since they are rigid members, their stiffness is assigned to very high quantity.

3. **Beam189:** used for modelling pylons in this study. They are similar in function and feature as compared to BEAM188, the only difference being a mid-node providing orientational freedom. This can be constrained to limit to rotation to behave like BEAM189.

Section	Section Type	Element
Girder	ASEC	BEAM188
Pylon	RECT	BEAM189
Cable	-	LINK180

Table 5-1 Elements of structural components in the model

5.3. Analysis

For this study, two types of analysis were performed on ANSYS APDL:

1. **Modal Analysis**: To confirm the natural frequency of the FE model with the CSIBridge model and establish the similarity in the model. This was completed by using the Block Lanczos solver.

2. **Non-linear static analysis:** To perform a non-linear analysis to obtain influence surface diagrams, while there is a moving load in the bridge. A non-linear static analysis was carried out using the non-linear solver in ANSYS Mechanical APDL. A factored truck load was applied along with factored lane loading on the bridge to emulate moving load for static analysis. The load combination used in this analysis is:

$$1.25D + 1.75LL \qquad (5.1)$$

. Therefore, the responses of the cables were calculated as shown in Figure 5-5.

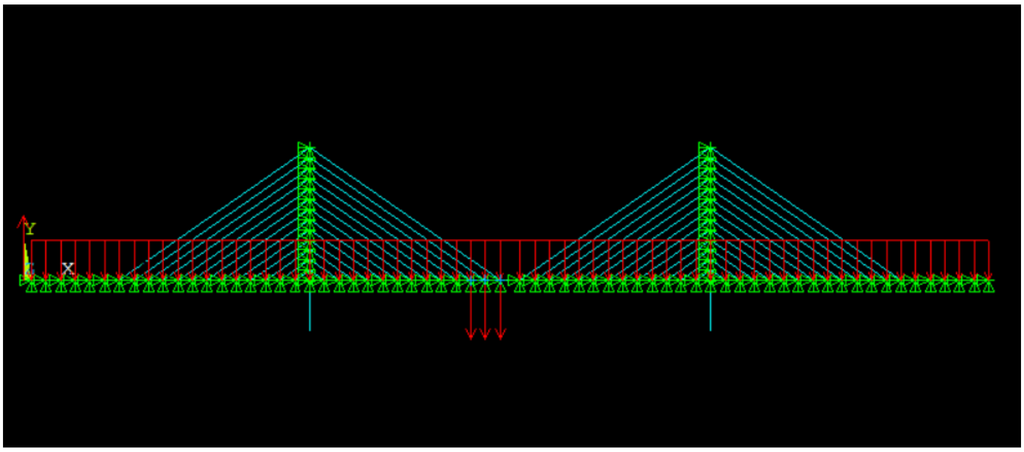

Figure 5-5 Non-linear static analysis performed on ANSYS

5.4. Results

The natural frequency was obtained to be 0.426 Hz, as compared to the CSIBridge model which had 0.4041 Hz, as show in Table 5-2. Figure 4-9 and also Figure 5-7 show the fundamental modal shape of the bridge model.

	CSIBridge	ANSYS	Error %
Fund. Freq.	0.4041	0.426	0.05141

Table 5-2 Fundamental frequencies in different models

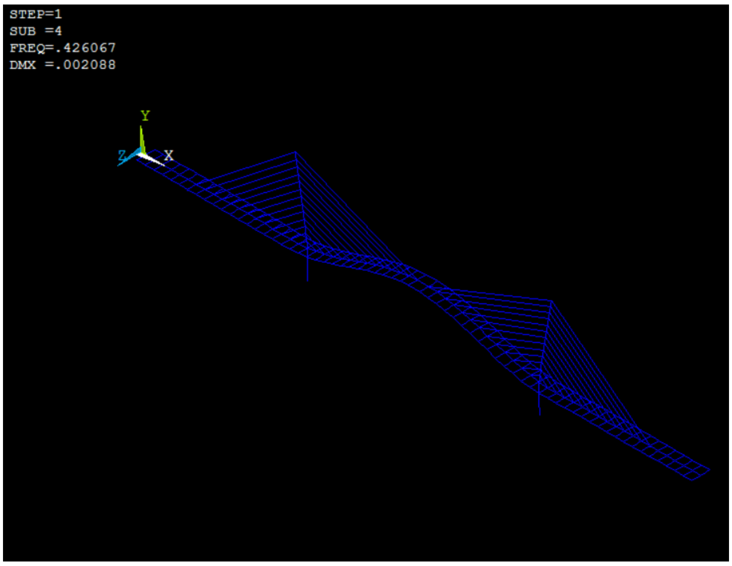

Figure 5-6 Fundamental mode as shown in ANSYS APDL

In the non-linear static analysis, a factored truck load along with lane loading applied to emulate an HL-93K truck as moving a load.. The number of load steps was chosen to take the nodal spacing in consideration and so that in every load steps the point load it applied directly on the node. From this analysis, we get the value of total cable force in the member.

The response of cables obtained from this analysis is similar to that obtained from in Chapter 4. The peak value of force also occurs in the similar region.

From the CSIBridge analysis, cables 21, 22 and 23 were deemed the critical members. In the non-linear static analysis in ANSYS, these members are revisited and the total cable force is determined.

Cable	Total force, kips	
	ANSYS	CISbridge
21	369.372	337.5
22	372.136	331
23	365.024	317.2

Table 5-3 Comparing force values of CSIBridge and ANSYS

6. Reliability Analysis

6.1. Procedure of implementation

To implement a reliability process flow, it is important to develop a framework. Reliability analysis is performed to calculate the P_f and β of different components of the system. Figure 6-1 illustrates the process involved in the development of the framework.

The procedure starts by generating random numbers for different variables involved in the analysis. The parameters recognised importance for the analysis were modulus of elasticity and yield strengths of both concrete and steel, geometrical and section properties of the girders, pylons and cables. This is because during structural installation, there may be a variation in the material strength, section properties and quality control measures affecting the structural properties and capacities. By setting the means and the COV for each of these parameters, random variables are generated and written on a separate text file. With the help of this text file, a FE model is generated on ANSYS to perform modal and moving load analyses. The axial loads of the cables obtained from each run are recorded and a Direct Monte Carlo simulation is run to obtain the P_f .

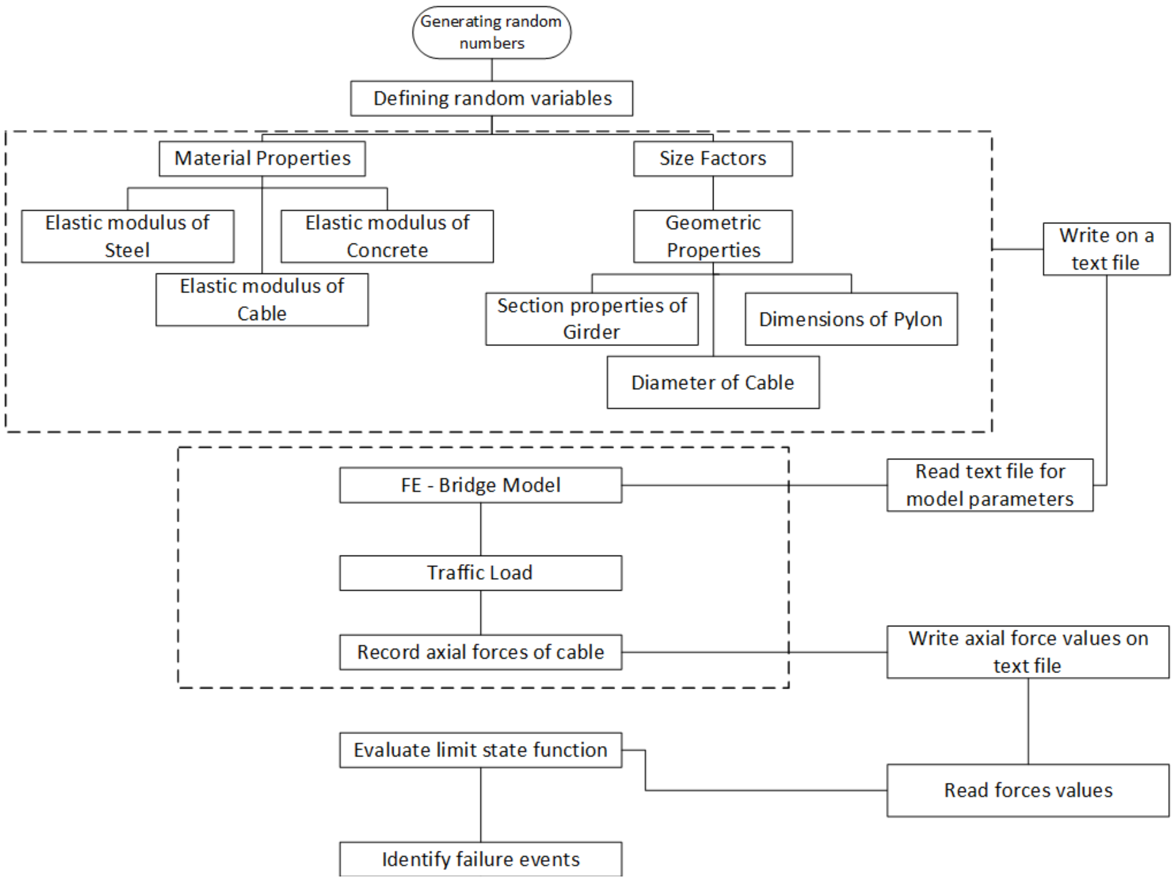

Figure 6-1 Flowchart describing the reliability analysis of the bridge subjected to cable failure events

6.2. Sampling of data

Latin hypercube sampling (LHS) is a form of stratified sampling that can be applied to multiply variables. This method is commonly used to reduce the number of runs necessary for a Monte Carlo simulation to achieve a reasonably accurate random distribution. LHS can be incorporated into an existing Monte Carlo model fairly easily, and work with variables following any analytical probability distribution. (https://mathieu.fenniak.net/latin-hypercube-sampling/).

This technique's origin dates back to the 1980's when computational challenges were immense, the distributions in models were modest, and simulations took in the order of days to complete. It was then that LHS was introduced to allow users to calculate accurate results while using fewer simulation cycles than simple MC sampling. (https://www.linkedin.com/pulse/20140708131747-483951-the-pros-and-cons-of-latin-hypercube-sampling)

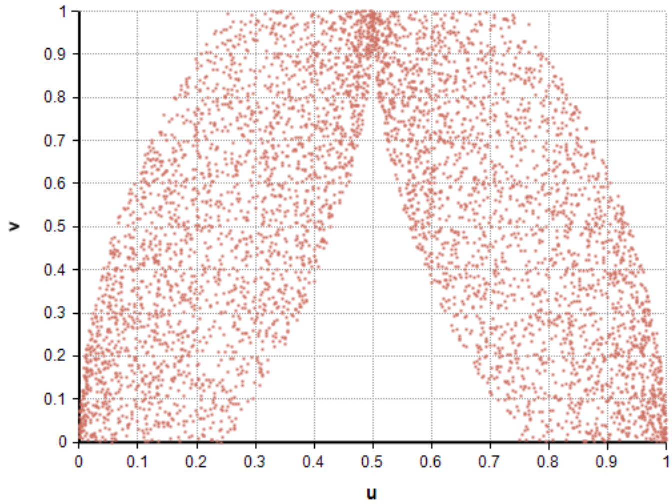

Figure 6-2 Latin Hypercube Sampling Technique

In LHS, the sample points are more evenly spread across all possible values. This happens by partitioning each input distribution into N intervals of equal probability. Then, one sample from each of those intervals is selected, to ensure that a random, yet well distributed set of random values are used in the simulation.

The LHS process can be discretely broken into two parts: sampling and grouping. For each simulation cycle, the cumulative probability is divided into segments. A probability is randomly picked from each segment and then mapped to the appropriate probability value in the actual distribution. This stage is known as the sampling and produces a stratified sample space. Once the variables are sampled, a random grouping is done for every distribution which in most cases is done by reversing the probability function process or by using random permutations.

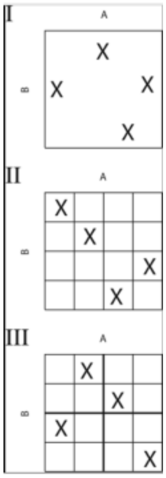

Figure 6-3 Illustrating LHS

There are two types of LHS – Random Latin Hypercube (R-LHS) and median Latin Hypercube (M-LHS). In R-LHS, the model selects a random point within each interval whereas, the M-LHS uses the medial value of each equi-probable interval.

Due to the presence of uncertainties in the structural properties and capacities, it leads to variation s in material strength, section properties, measurements and processing. The parameters associated with the model are characterized by different type of distribution but following the sampling methods used by Zhoun, Y et al (2017), the most common distribution types were used in this analysis. [11]

6.3. Limit States

Discussed in the literature review, multiple research papers have considered three possible limit states:

1. Ductile failure
2. Structural Failure
3. Functional failure

In this analysis, we shall only discuss about ductile failure of the cables. When the forces in the cables exceed ultimate yield strength of the material, i.e. steel in this case, we can therefore define a limit state function Z as:

$$Z = 1 - \frac{F}{F_u} \tag{6.1}$$

Where, F is the axial force of the cable and F_u is the ultimate yield force. When Z < 0, it means that the axial force has exceeded the ultimate yield strength of the cable and therefore, the cable has failed. Z was calculated for all cables during the event when the cable exceeds the limit state value, i.e. that value of Z < 0.The mean and standard deviations were calculated for the axial force value and N simulation cycles were run. P_f is introduced as the probability of exceeding limit state and calculated as:

$$P_f = \frac{Number\ of\ failure\ events(N_f)}{Total\ number\ of\ simulation\ cycles(N)} \tag{6.2}$$

where N_f is number of failure events.

MATLAB was used to generate random variables by using LHS method for the sampling process for each of the parameters. The parameters considered for the analyses are shown in*Table 6-1*. X1 and X2 are random variables for the elastic modulus of concrete (for girder and pylon) and steel (for cable) respectively and they assumed to follow the normal distribution. The coefficient of variance value (COV) of 0.15 is considered through references. [11]

Size factors X3-X10 are for sections properties of the cables, girders and pylons. By multiplying the size factor mean value of the size, area, moment of inertia and plastic section modulus with the corresponding first, second, third and fourth order respectively. The COV value is considered to be 0.05 to allow minimal variations in the value of the geometric properties and prevent considerable changes in size of the randomly generated models. The size factors of the girders include I_{yy}, I_{yz}, I_{zz}, Plastic Modulus Z and the centroid of the girder. The size factors of the pylons and cables include dimension of the rectangular cross-section and the diameter of the section respectively. All the section property values are borrowed from the CSIbridge model discussed in 4. [11]

The yield strength is assumed to follow a lognormal distribution with variables X11 for concrete and X12 for steel and similar to variables X1 and X2, the COV is taken to be 0.15. [11]

Random Variables	Distribution Type	Mean	COV
Elastic Modulus of girder/pylon, X1	Normal	5.76×10^8 lb/ft^2	0.15
Elastic Modulus of cable, X2	Normal	4.176×10^8 lb/ft^2	0.15
Size Factor (Cable), X3	Normal	1	0.05
Size Factor (Girder), X4 - X8	Normal	1	0.05
Size Factor (Pylon), X9 - X10	Normal	1	0.05
Yield Strength (Girder/Pylon), X11	Lognormal	3.13×107^7 lb/ft^2	0.05
Yield Strength (Cable), X12	Lognormal	5.76×10^8 lb/ft^2	0.05

Table 6-1 Probability distributions of parameters associated with the bridge structure

A model was created for each set of random variable generated in the program. For this to happen, it was important to integrate ANSYS with MATLAB.

6.4. Results

The three cables on which the analysis was performed in the previous chapters, Z was calculated for 250 simulations and the number of times the axial force on the cables exceeded the ultimate tensile strength value was calculated. A P_L of 0.004 was observed for cables 21 and 23 and a P_L of 0.008 was observed for cable 22. The number of simulations must be increased to 500 or 1000, such that, the value of P_L attains convergence and is more accurate. Cables are pre-tensioned during bridge construction and safety factors are taken into consideration to prevent the cables forces to reach the limit state value. The cables considered in this experiment are the most critical members in the cable-system but the values of P_L are very less and is an event that is bound to happen rarely.

	μ_{Fi}	σ_{Fi}	P_L
Cable - 21	369.3728	15.7	0.004
Cable - 22	372.1368	12.6	0.008
Cable - 23	365.0248	16.7	0.004

Table 6-2 Results of Reliability analysis

7. Conclusion

7.1. Summary

In this study, three different models of the James River Bridge were discussed. From the original design of the bridge, there were assumptions that went into the model before starting the analysis, all mentioned in Chapter 3.

The first model was developed on CSIBridge, where time history response diagrams were obtained after performing a moving load analysis with the help of HL-93F truck load. A modal analysis was first done to obtain the natural frequency and fundamental mode shape of the bridge. The moving load analysis was initially preformed for an undeformed model and the 3 critical cables were identified on further analysis was performed. The value of maximum force in the cable was found by adding the dead load to the moving load response in each time step.

A FE model was then created on ANSYS Mechanical APDL, to perform the non-linear static analysis. A modal analysis was first completed to check its accuracy with the CSIBridge model. The key is to decide the size of each time step which helps the solver to position the load during the analysis. In this analysis, a factored truck load was applied along with factored lane load to emulate the moving load analysis. From the data obtained, the value of the total force in the cable was found by multiplying it with the axle force with the cable force obtained through analysis along with the dynamic amplification factor.

A reliability analysis is an integral part of this project. The FE model developed in the case study before was modified such that it takes the random variables generated by a MATLAB program as input. The same MATLAB script also runs the ANSYS script multiple times which is equal to the number of random variables generated and also to the total number of simulations involved. Under factored load conditions, the probability of the cable exceeding its limit state was evaluated and was concluded that the occurrence of that event is rare. Progressively, the number of simulations achieved in this project must be increased sufficently to get a convergence on the value of $P_{L.}$ To complete a failure study of the cables, a redundancy analysis should be performed where the load is increased with every time step until the member fails.

7.2. Future work

As mentioned at the end of Chapter 6, to approach limit states, the concept of reliability analysis should be coupled with the concept of redundancy analysis, which goes a step further than moving load and non-linear static analyses. In redundancy analysis, with the information we have from this study, the load of the HL-93F truck should be incremented until the first member fails and continued so forth until limit states are achieved. On the basis of the three limit states mentioned at the beginning of this document, redundancy ratios are calculated to quantify the bridge’s degree of redundancy.

PTI Recommendations (2001), includes term in the load combination for cable loss, CLDF (Cable Loss Dynamic Factor). The term CLDF signifies that during a cable-failure event, there is compensating dynamic force that applied to the bridge and is necessary to include when calculating cable-failure.

This framework is rudimentary and can be developed into a multi-faucet tool and analysed for different structural components. Interfaces can be created to generate and simulate models to obtain the failure criteria in various scenarios. Since both of them are conservative approaches, a non-linear dynamic analysis is necessary to approach the problem statement in a holistic manner. This is presently being used in FE model updating where based on the design drawings of a given bridge an FE model is created before the completion of the bridge construction. Initial analysis through strain gauges and accelerometer installed at specific location of the bridge is required to calibrate the model. Once the model is perfected, it can be used to simulate and predict the behaviour of the bridge during various load cases and extreme scenarios.

8. Appendix

a. ANSYS Code

```
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/clear

/prep7

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!k,5,765,0,0
k,6,450,208,0
k,7,450,-80,0
k,8,781,0,0
k,9,1080,0,0
k,10,1379,0,0
k,11,1517,0,0
k,12,1080,208,0
k,13,1080,-80,0

k,14,13,0,-25
k,15,151,0,-25
k,16,450,0,-25
k,17,749,0,-25
!k,18,765,0,-25
k,19,781,0,-25
k,20,1080,0,-25
k,21,1379,0,-25
```

```
k,22,1517,0,-25

k,23,13,0,25
k,24,151,0,25
k,25,450,0,25
k,26,749,0,25
!k,27,765,0,25
k,28,781,0,25
k,29,1080,0,25
k,30,1379,0,25
k,31,1517,0,25

*do,i,1,13,1                !Keypoint on the 1st pylon
k,100+i,450,16*i,0
*enddo


*do,i,0,19,1                !Keypoint on the 1st grider segment for 1st cable set
k,200+i,450-23*i,0,0
*enddo

*do,i,1,13,1                !Keypoint on the 2nd grider segment for 1st cable set
k,300+i,450+23*i,0,0
*enddo


*do,i,1,13,1                !Keypoint on the 2nd pylon
k,400+i,1080,16*i,0
*enddo
```

```
*do,i,0,13,1                  !Keypoint on the 2nd girder segment for 2nd cable set
k,500+i,1080-23*i,0,0
*enddo


*do,i,1,19,1                  !Keypoint on the 3rd girder segment for 2nd cable set
k,600+i,1080+23*i,0,0
*enddo

*do,i,0,19,1                  !Keypoints for links on the left (-z) of the 1st girder
segment for 1st cable set
k,220+i,450-23*i,0,-25
*enddo

*do,i,0,19,1                  !Keypoints for links on the Rgiht (+z) of the 1st girder
segment for 1st cable set
k,240+i,450-23*i,0,25
*enddo

*do,i,1,13,1                  !Keypoints for links on the left (-z) of the 2nd girder
segment for 1st cable set
k,320+i,450+23*i,0,-25
*enddo

*do,i,1,13,1                  !Keypoints for links on the Rgiht (+z) of the 2nd girder
segment for 1st cable set
k,340+i,450+23*i,0,25
*enddo

*do,i,0,13,1                  !Keypoints for links on the left (-z) of the 2nd girder
segment for 2nd cable set
k,520+i,1080-23*i,0,-25
```

```
*enddo

*do,i,0,13,1                    !Keypoints for links on the Right (+z) of the 2nd girder
segment for 2nd cable set

k,540+i,1080-23*i,0,25

*enddo

*do,i,1,19,1                    !Keypoints for links on the left (-z) of the 3rd girder
segment for 2nd cable set

k,620+i,1080+23*i,0,-25

*enddo

*do,i,1,19,1                    !Keypoints for links on the Right (+z) of the 3rd girder
segment for 2nd cable set

k,640+i,1080+23*i,0,25

*enddo

!*do,i,1,9,1                    !Keypoints for links on the Centre of the 1st girder
segment

!k,255+i,15*i,0,0

!*enddo

!*do,i,1,9,1                    !Keypoints for links on the Right of the 1st girder
segment

!k,266+i,15*i,0,25

!*enddo

!*do,i,1,9,1                    !Keypoints for links on the Left of the 1st girder
segment

!k,277+i,15*i,0,-25

!*enddo
```

```
!*do,i,1,9,1                    !Keypoints for links on the Centre of the 3rd girder segment

!k,655+i,1530-15*i,0,0

!*enddo


!*do,i,1,9,1                    !Keypoints for links on the Right of the 3rd girder segment

!k,666+i,1530-15*i,0,25

!*enddo


!*do,i,1,9,1                    !Keypoints for links on the Left of the 3rd girder segment

!k,677+i,1530-15*i,0,-25

!*enddo


K,901,151,-1,0

K,902,151,-1,-25

K,903,151,-1,25

K,904,312,-1,0

K,905,312,-1,-25

K,906,312,-1,25

K,907,1218,-1,0

K,908,1218,-1,-25

K,909,1218,-1,25

K,910,1379,-1,0

K,911,1379,-1,-25

K,912,1379,-1,25


l,1,2       !L1

l,2,3       !L2

l,3,4       !L3

!l,4,5      !L4
```

```
l,4,8        !L4
l,3,6        !L5
l,3,7        !L6
!l,5,8       !L7
l,8,9        !L8
l,9,10       !L9
l,10,11      !L10
l,9,12       !L11
l,9,13       !L12 (L11)

*do,i,1,13,1          !L12-L24
l,100+i,200+i
*enddo


*do,i,1,13,1          !L25-L37
l,100+i,300+i
*enddo


*do,i,1,13,1          !L38-L50
l,400+i,500+i
*enddo

*do,i,1,13,1          !L51-L63
l,400+i,600+i
*enddo

*do,i,1,3,1 !L64-66
l,13+i,13+i+1
*enddo
```

```
l,17,19                  !L67

*do,i,6,8,1 !L68-70
l,13+i,13+i+1
*enddo

*do,i,1,3,1 !L71-73
l,22+i,22+i+1
*enddo

L,26,28                  !L74

*do,i,6,8,1 !L75-77
l,22+i,22+i+1
*enddo

l,1,14        !L78-80
l,2,15
!l,3,16
!l,4,17
!l,5,18
!l,8,19
!l,9,20
!l,10,21
l,11,22

l,1,23        !L81-83
l,2,24
!l,3,25
!l,4,26
```

```
!l,5,27
!l,8,28
!l,9,29
!l,10,30
l,11,31

*do,i,0,19,1        !L84-123
l,220+i,200+i
l,240+i,200+i
*enddo

*do,i,1,13,1        !L124-L149
l,320+i,300+i
l,340+i,300+i
*enddo

*do,i,0,13,1        !L150-177
l,520+i,500+i
l,540+i,500+i
*enddo

*do,i,1,19,1        !L178-215
l,620+i,600+i
l,640+i,600+i
*enddo

!*do,i,1,9,1        !L220-228
!l,266+i,255+i
!l,277+i,255+i
!*enddo
```

```
!*do,i,1,9,1            !L229-237
!l,666+i,655+i
!l,677+i,655+i
!*enddo

l,901,2                 !Pesudo_members for supports !L216-227
l,902,15
l,903,24
l,904,206
l,905,226
l,906,246
l,907,606
l,908,626
l,909,646
l,910,10
l,911,21
l,912,30


ET,1,LINK180
R,1
MP,EX,1,4.176E9
MP,PRXY,1,.3
MP,DENS,1,15.23
SECT,1,LINK
SECD,0.0135
SECCONTROL,6.07,1

ET,2,BEAM188
R,2
MP,EX,2,5.76E8
```

```
MP,PRXY,2,.3
MP,DENS,2,4.66

SECT,2,BEAM,ASEC,"Girder12"
SECD,83.553,13009.084 , -654.8274, 1686.6953,, 3924.8681, 7.7151,,,,,
SECOFFSET, ORIGIN
SECCONTROL,7.2e10,,
!area of girder = 480 ft2

SECT,3,BEAM,RECT,"Middle_Deck"
SECD,0.8333,4
SECOFFSET, ORIGIN
SECCONTROL,9.76e10,,,
!area of girder = 480 ft2



!SECT,3,BEAM,RECT,"Middle_Deck"
!SECD,0.8333, 4
SECCONTROL,7.2e10,,,
!area of middle deck = 3.332 ft2

R,4
MP,EX,4,9E18
MP,PRXY,4,.3
SECT,4,BEAM,CSOLID,"Link_Girder"
SECD,1,,
SECCONTROL,6.023e10,,,
!Link griders

ET,3,BEAM189
R,2
```

```
MP,EX,3,5.76e8
MP,PRXY,3,.3
MP,dens,3,4.66
SECT,5,BEAM,RECT,"Pylon_12"
SECD,8,8
SECCONTROL,653.85e10,0
!area of pylon = 4.359 ft2

LSEL,,,,1,4
LATT,2,2,2,,,,3

LSEL,,,,5,6
LATT,3,3,3,,,,5

LSEL,,,,7,9
LATT,2,2,2,,,,3

LSEL,,,,10,11
LATT,3,3,3,,,,5

LSEL,,,,12,63
LATT,1,1,1,,,,1

LSEL,,,,64,77
LATT,2,2,2,,,,2

LSEL,,,,78,227
LATT,4,4,3,,,,4


DK,1,,,,,UY,UZ,ROTZ
```

```
DK,23,,,,,UY,UZ,ROTZ
DK,14,,,,,UY,UZ,ROTZ
DK,901,,,,,UY,UZ,ROTZ
DK,902,,,,,UY,UZ,ROTZ
DK,903,,,,,UY,UZ,ROTZ
DK,904,,,,,UY,UZ,ROTZ
DK,905,,,,,UY,UZ,ROTZ
DK,906,,,,,UY,UZ,ROTZ
DK,907,,,,,UY,UZ,ROTZ
DK,908,,,,,UY,UZ,ROTZ
DK,909,,,,,UY,UZ,ROTZ
DK,910,,,,,UY,UZ,ROTZ
DK,911,,,,,UY,UZ,ROTZ
DK,912,,,,,UY,UZ,ROTZ
DK,11,,,,,UY,UZ,ROTZ
DK,22,,,,,UY,UZ
DK,31,,,,,UY,UZ
DK,7,ALL
DK,13,ALL

LSEL,,,,1
LESIZE,ALL,,,6

LSEL,,,,2
LESIZE,ALL,,,13

LSEL,,,,3
LESIZE,ALL,,,13

LSEL,,,,4
LESIZE,ALL,,,1
```

```
LSEL,,,,5
LESIZE,ALL,,,13

LSEL,,,,6
LESIZE,ALL,,,5

LSEL,,,,12,63
LESIZE,ALL,,,1

LSEL,,,,7
LESIZE,ALL,,,13

LSEL,,,,8
LESIZE,ALL,,,13

LSEL,,,,9
LESIZE,ALL,,,6

LSEL,,,,10
LESIZE,ALL,,,13

LSEL,,,,11
LESIZE,ALL,,,5


LSEL,,,,64                    !Side_girder_Left
LESIZE,ALL,,,6

LSEL,,,,65
LESIZE,ALL,,,13
```

```
LSEL,,,,66
LESIZE,ALL,,,13

LSEL,,,,67
LESIZE,ALL,,,1

LSEL,,,,68
LESIZE,ALL,,,13

LSEL,,,,69
LESIZE,ALL,,,13


LSEL,,,,70
LESIZE,ALL,,,6


LSEL,,,,71                    !Side_girder_Right
LESIZE,ALL,,,6

LSEL,,,,72
LESIZE,ALL,,,13

LSEL,,,,73
LESIZE,ALL,,,13

LSEL,,,,74
LESIZE,ALL,,,1

LSEL,,,,75
```

```
LESIZE,ALL,,,13

LSEL,,,,76
LESIZE,ALL,,,13

LSEL,,,,77
LESIZE,ALL,,,6


LSEL,,,,78,227                    !Girder_links
LESIZE,ALL,,,1



!Fk,53,FY,-32.24
!Fk,41,FY,-32.24
!ACEL,,-32.2

ALLSEL,ALL,lines
LMESH,ALL

allsel,all
cpintf,all

!pivcheck,off



/SOLU
ANTYPE, STATIC
NLGEOM,on
```

```
CNVTOL, F, 10e20,0.05,,
CNVTOL, M, 10e20,0.05,,
CNVTOL, U, 10e10,0.05,,
KBC,1
NEQIT,50
NSUBST,10,50,1
ACEL,0,1.25*32.15,0
ESEL,S,CENT,Y,0,0
ESEL,R,CENT,Z,0,0
SFBEAM,ALL,2,PRES,640*1.75
allsel,all
f,   32      ,fy,     -42864.79822*1.75
f,   32      ,mz,     -68597.3535*1.75
f,   33      ,fy,     -28165.36533*1.75
f,   33      ,mz,     82207.93951*1.75
f,   21      ,fy,     -969.8364428*1.75
f,   21      ,mz,     6805.293006*1.75
nsub,50

solve

/post1
etable,axial_force,smisc,1
*get,enum,elem,,count
*GET,NUMMIN,ELEM,,NUM,MIN
*DIM,AR,ARRAY,ENUM,2
*DO,I,1,433
 AR(I,1)=NUMMIN
*GET,AR(I,2),ETAB,1,ELEM,NUMMIN
NUMMIN=ELNEXT(NUMMIN)
*ENDDO
```

```
/INPUT,ansuitmp
*CREATE,ansuitmp
*CFOPEN,AXIAL_FORCE,TXT
*VWRITE,AR(1,1),AR(1,2)
(F15.5,3X,F15.5,3X,F15.5,3X,F15.5)
*CFCLOS
*END
```

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