ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

INVESTIGATION OF THE EFFECT OF HUMAN-INDUCED VIBRATIONS ON PEDESTRIAN CABLE BRIDGES

(A CASE STUDY OF SELECTED PEDESTRIAN CABLE BRIDGE)

A THESIS IN DEGREE OF MASTER OF SCIENCES IN STRUCTURAL ENGINEERING

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A THESIS
SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE

The undersigned have examined the thesis entitled '**Investigation of the effect of human-induced vibrations on pedestrian cable bridges (a case study of selected pedestrian cable bridge)'** presented by **Dawit Gebre Sahle**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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DECLARATION

I certify that this research work entitled "Investigation of the effect of Human-Induced Vibrations on Pedestrian Cable Bridges" is my own work. The work has not been presented elsewhere for assessment and award of any degree or diploma. Where material has been used from other sources it has been duly acknowledged.

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Date of submission:

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ABSTRACT

Many rural communities around the world become isolated from their basic needs during the rainy season. So, low cost pedestrian cable bridges are being built to provide hundreds of thousands of people with basic access. However, cable pedestrian bridges have low stiffness, mass and damping, causing them to be prone to vibration problems. Pedestrian loading can cause a dynamic effect that creates public alarm to the point where bridge users perceive it to be unsafe. Many footbridges have natural frequencies that coincide with the dominant frequencies of the pedestrian-induced load. Therefore, they have a potential to suffer excessive vibrations under dynamic loads induced by pedestrians.

The main focus of this thesis is an investigation of the effect of human-induced vibrations on pedestrian cable bridges and how the vertical forces that pedestrians impart to cable suspended footbridge can be modeled to be used in the dynamic design of footbridges. The work was mainly divided into four subtasks. First literature studies on effect of Human-Induced Vibrations on suspended footbridge have been performed. Second design criteria and load models proposed by three widely used standards have been introduced, then, numerical models using finite element software has been performed in persons walking and jogging condition to determine how changing certain design parameters including walkway and handrail cables diameter and percentage of sag and number of walking and jogging pedestrian affects modal frequencies and vertical dynamic response. Finally, the research results have been presented and discussed.

The study result showed that in most cases the modal frequencies of pedestrian suspended bridges do not meet the recommended ranges. Also, it reveals that the vertical accelerations of the structure depend on the number of walking and jogging pedestrians across the bridge. In addition, shorter bridge span length has higher modal frequencies and dynamic responses.

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1. INTRODUCTION

1.1 Background

The improvement of infrastructure in general, the road sector in particular, in a country determines the economic well-being and the overall growth of a nation. Construction of bridge structures, which is the major activity in the construction of roads that requires serious attention from modeling, analysis and design activity up to finalizing its construction. On the construction of roads, bridges are the major structures which are exposed to dynamic loading. Therefore, careful attentions should be employed in the modeling and assessment of bridge behavior under such loadings.

A pedestrian bridge, also called a footbridge is simply a bridge, whether over land or water, that is designed for foot traffic. Often, pedestrian bridges are constructed to give pedestrians a safe way to cross from one side of road to the other. Therefore, pedestrian bridges play a great role in minimizing traffic accidents and facilitating transportation.

There are different types of pedestrian bridges such as timber, steel and concrete footbridges. The combination of steel and concrete in a single composite structural element enhances the individual advantages of both materials. By utilizing the high tensile strength of steel together with the compressive strength of concrete, the resulting elements have one and a half times or even double the strength and stiffness in comparison with a non-composite element. Pedestrian bridges are built lighter and with longer spans. Design of Pedestrian bridges normally follows the same principles as for other bridges. However, because they are normally significantly lighter than vehicular bridges, these lightweight bridges are often susceptible for human induced vibrations and therefore dynamics effects are often given more attention in analysis and design. But it is still important to ensure pedestrian comfort.

Pedestrian bridge technologies vary vastly in design, cost and function. Crossings can be as simple as a fallen tree or as complex as a multi-million-dollar work of art. From a structural standpoint, pedestrian bridges have taken a number of forms, each with the function of providing safe transport over an otherwise impassable crossing. Arched bridges, simple beam bridges, truss bridges, suspended and suspension Cable Bridge and cable-stayed bridges constitute main types of pedestrian bridges.

The cable suspended bridge contains load carrying cables for the walkway and also often for the handrails. Because of its natural shape the main use is as a pedestrian bridge. The reason for the widely use of suspended bridges in aid projects around the world is that the structure is relatively simple and have shown to be convenient for construction in rural areas since it can cross large span in combination with being material efficient and therefore quite cheap. A suspended bridge can most often be built without the use of heavy machinery and it requires very little maintenance.

The maximum span length for the suspended cable bridge designs is 120 meters. Due to dynamic effects of lateral wind loadings for spans exceeding 120 meters, lateral stabilizing measures (wind guys) must be implemented. For locations with exceptionally high wind speeds, a qualified engineer should be consulted to determine the necessity of lateral stabilization [1].

The Suspension footbridge has taller towers at the supports which gives larger sag and a more effective transfer of vertical loads. The cable and the deck are connected with vertical hangers. The deck can be made more or less stiff. Suspension bridges have the advantage of having a horizontal walkway or traffic path, but it can even take a convex shape if the designer finds this more preferable. The difference between a cable-suspension footbridge and a cable-suspended footbridge type is shown in Figure 1.1.

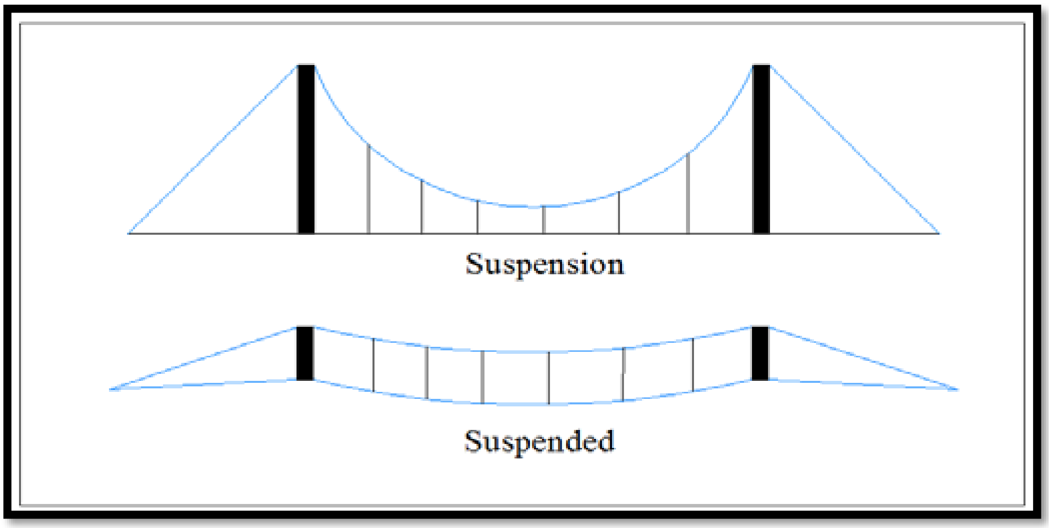


Figure 1.1 Suspension and Suspended Footbridge Comparison [1].

The modern cable-suspended footbridges constructed by Bridges to Prosperity do not vary greatly from many of the historical bridges. The simple design, constructed using manually-powered tools and only locally available materials are used.

Bridges to Prosperity (B2P) is a United States based non-profit organization that has recognized the need for rural pedestrian bridges. Their work building and training a specific cable suspended footbridge technology has connected rural communities with access and opportunities in over a dozen countries around the world. The cable suspended footbridge design used by Bridges to Prosperity (B2P) was first developed by the Swiss organization Helvetas (2001). Helvetas took footbridge building practices from improvised construction to a standardized bridge design manual while creating the world's largest trail bridge program in Nepal (Nepal, 2008). The suspended design relies on each cable for load distribution and lacks the tall towers equated with suspension bridges. An example of the Helvetas-type suspended bridge is shown in Figure 1.2.

Figure 1.2 Typical Suspended Footbridge [1].

Helvetas successfully accomplished their goal of standardizing the design such that a visual geotechnical evaluation and rudimentary topographic. Although the modulated design was appropriate for deep gorge applications as found in Nepal, there is a desire to break-down the design process to allow for easier design modifications more suitable for non-gorge crossings.

Bridges to Prosperity (B2P) started in 2001 by using the Helvetas design manual as it was the most comprehensive design reference available. Several design alterations and modifications have taken B2P away from the original designs as many B2P crossings have topographic situations not addressed in the Helvetas manual. All of these design addendums and calculation assumptions have been posted on their internet site. This document seeks to provide a more complete best-practice document to serve as a resource for potential bridge-builders around the world through B2P’s online database [1].

The research finding of this thesis will useful in understanding the complex behavior of vibration sensitive cable suspended footbridge under human-induced dynamic loads. They will be helpful in developing design guidance and techniques to improve the dynamic performance of such bridges and similar structure and hence to ensure their safety and serviceability.

1.2 Objective of the Research

The main aim of the thesis is to generate fundamental knowledge and contribute to a better understanding of dynamic performance of Pedestrian Cable Bridges under human-induced vibration and to investigate the effects of structural parameters and number of pedestrians on the vertical modal frequencies and vertical dynamic response of such bridges.

There are many different types of Pedestrian Cable Bridges, but the footbridges used for the present study will be based on the standards from Bridges to Prosperity because this type of footbridge is being built in countries all around the world and vibration problems are known to be an issue.

The specific research objectives are as follows:

- Select proper design guide (codes) to analysis cable suspended pedestrian bridges due to human induced vibration.

- Carry out numerical study and model (finite element) analysis using MIDAS/Civil Bridge software on the dynamic characteristic of cable suspended pedestrian bridges which designed and constructed by Bridges to Prosperity.

- Perform a comparative study over the research result and design guidelines standard.

1.3 Statement of the Problem

To construct bridges of medium and large size with concrete or steel requires considerable amount of money. And with the existing economy it is difficult for a country like Ethiopia. Also due to the topographic nature and the absence of narrow river embankment there are difficulties in constructing bridges with concrete and steel and it is unnecessary to construct with those materials for pedestrian use only.

Due to factors mentioned above, lightweight and low-cost bridge systems need to overcome these problems. And during the design of such bridges human induced vibration is the main concern. Shortcomings have been highlighted in existing codes of practice in relation to the human induced vibration response of footbridges but nevertheless, there is still no clear regulatory guidance on dynamic analysis and design of cable pedestrian bridges especially cable suspended pedestrian bridges which are designed and constructed by “Bridges to Prosperity” all around the world. The intention of this thesis is therefore, to investigate the effect of Human-Induced Vibrations on Pedestrian Cable bridges.

1.4 Applications and limitations

1.4.1 Applications

The research finding of this thesis will useful in understanding the complex dynamic behavior of vibration sensitive cable suspended footbridge under human- induced loads. They will be helpful in developing design guidance and techniques to improve the dynamic performance of such bridges and similar structure. Hence, it can be used to verify their safety and serviceability.

1.4.2 Limitations

The scope of this research is limited to the Investigation of the effect of Human-Induced Vibrations on cable-suspended pedestrian bridges. Only vibrations in the vertical direction which are caused by pedestrians are studied. This was because the largest vibrations caused by pedestrians are in the vertical direction. These studies do not consider other bridge type like concrete or cable stayed bridges.

1.5 Methodology

1.5.1 Introduction

This section explains more extensively on how the research is conducted. The study of investigation of the effect of human-induced vibrations on pedestrian cable bridges under goes different phases.

This study is mainly involved in the modeling and analysis of different span length and cable size pedestrian cable suspended bridges, which are designed and constructed by bridge to prosperity.

In this study, the main sources of literature review are from journals, articles and conference papers. Since the subject of human-induced vibrations on pedestrian cable bridges is new concept in Ethiopia, there are no published documents from the local authorities. Therefore, all the articles are referred mainly from the Journals of Civil Engineering, Structural Engineering and Bridge Engineering published by different society of Civil Engineers.

This research composed of several tasks in finite element modeling and time-history analysis for human induced vibration. Three dimensions finite element model is developed with the MIDAS/Civil finite element software. All the geometric data are included in the model. Cable suspended footbridges were modeled in the software to determine vertical natural frequencies of the footbridge and vertical dynamic response.

1.5.2 Selection of cable suspended pedestrian Bridges for Case Study

Pedestrian Cable suspended Bridges selected in this study is designed and constructed by Bridges to Prosperity at Kolfe Keranio, Addis Ababa in 2004. The selection of this bridges is based on its easily exposure to human induced vibration and being common types of bridge, which designed and constructed by Bridges to Prosperity in Ethiopia.

The selected bridge is rock anchored four cable suspended pedestrian bridge with steel cross beams and deck panels, modeled after Nepali trail bridge standard. All work performed by local community members, hand excavated, rock anchors hand drilled, masonry towers with concrete core, cable laid in place by hand and tightened with 3-ton chain block, deck and cross beams prefabricated in Addis Ababa and installed by hand. This bridge was the first structure completed in a new bridge building program started in Ethiopia by Bridges to Prosperity, based on the Helvetas Nepal trail bridge building system. The saddles are made of epoxy coated 10mm steel plate, the suspended cables of 26mm prestressed 7 x 19 galvanized wire rope. For the deck panels painted steel angles 40 x 40 x 3mm in '3 T' arrangement were used and the cross beams are painted steel angles 40 x 40 x 5mm in 'T' arrangement. The width and span of selected bridge is 1.06m and 42m respectively.

In this study, several numerical models were conducted in MIDAS/Civil. For all analysis the section and material properties were taken from existing selected cable bridge but certain design parameter including span length, walkway and handrail cable diameter, cable sag and number of pedestrians were changed to determine how those design parameters affect vertical natural frequency and vertical dynamic response of selected pedestrian cable suspended bridge. The information obtained is used in computer analysis as an input data.

1.5.3 Software Verification

Physical modeled cable suspension footbridge by Jennifer Kearney and Jeffrey A. Laman with 40m and 80m span length has been taken to verify the reliability of the software [5].

The physically model was published on International Journal of Bridge Engineering (IJBE), Vol. 4, No. 3, (2013), pp. 21-35 under title of "**Suspension footbridge dynamic response to pedestrian loading and corresponding human comfort**".

Like suspended footbridge physical modeled pedestrian suspension footbridges have low stiffness, low mass and low damping making them prone to significant vertical accelerations under walking and jogging pedestrian loads. In addition, for both footbridge cable is the main load resisting component.

Numerical models have been analyzed in MIDAS/Civil to determine the modal frequency and compare with physical modal result.

To correctly represent the physical model the main cables and suspenders cables were modeled with cable elements. Tower model with pipes. Nailers (wood members attached to cross-beams) and the safety fence were modeled as a distributed load and joint masses, respectively, rather than as structural elements. Boundary conditions were idealized as follows: support tower columns are pin connected; suspension cables are pin connected at the anchors.

The comparison between physical modal and Numerical model result presented as follow:

Table 1.1 40 m span comparison results.

Vertical Mode	Physical Model Freq (Hz)	Midas Civil Model Freq (Hz)	Percent Difference (%)
1.00	0.45	0.52	**15.56**
2.00	0.87	0.81	**7.41**
3.00	1.09	1.10	**0.92**

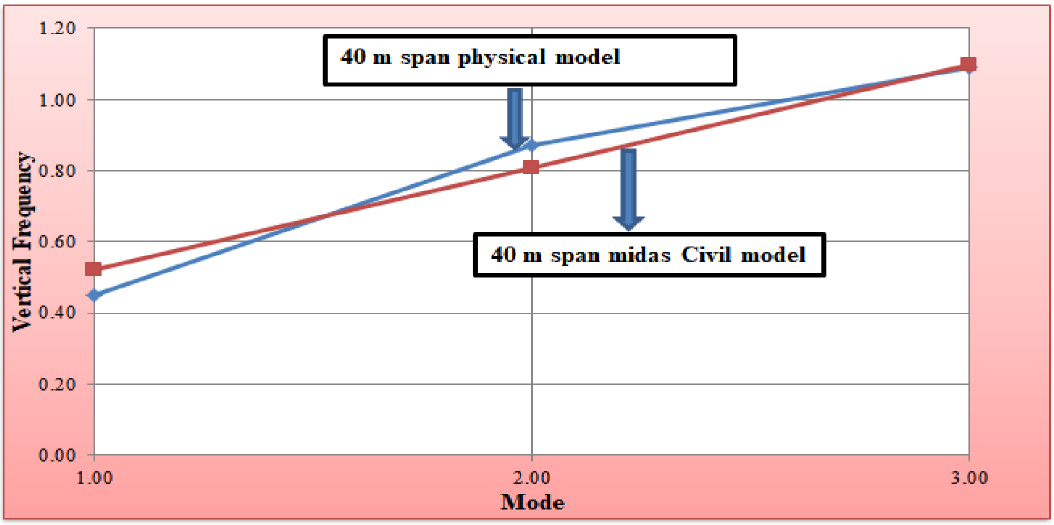


Figure 1.3 40m span physical model and MIDAS/Civil model results.

Table 1.2 80 m span comparison results.

Vertical Mode	Physical Model Freq (Hz)	midas Civil Model Freq (Hz)	Percent Difference (%)
1.00	0.41	0.42	**2.44**
2.00	0.53	0.52	**1.89**
3.00	0.63	0.71	**12.70**

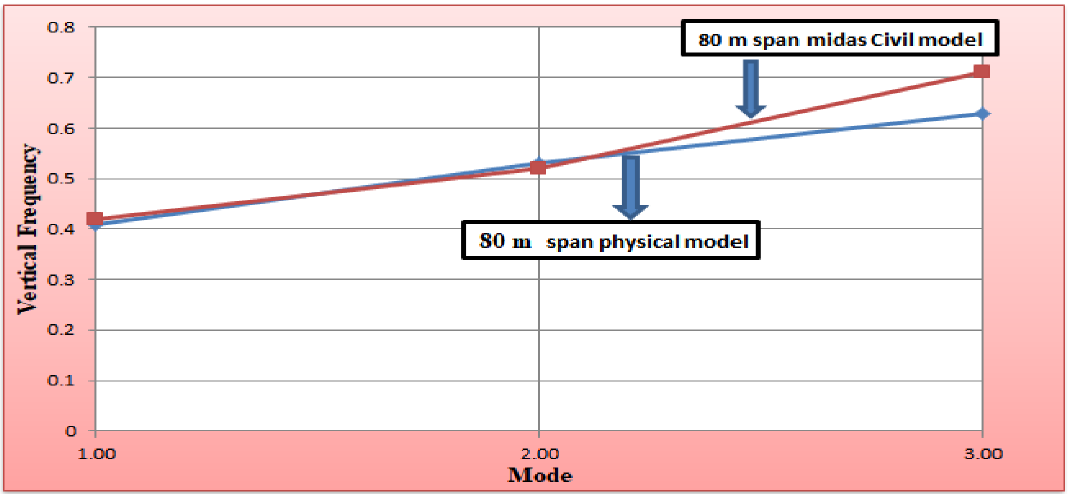


Figure 1.4 80m span physical model and MIDAS/Civil model results.

The small difference in the results of modal frequencies between midas civil model and physical model is due to construction imperfections in the physical model that are not present in the numerical model and suspender wire in the physical models is not perfectly straight. Therefore, it is appropriate to utilize the program as analysis software.

The detail physical and software model analysis including material and section properties are presented at the appendix A.

1.5.4 Finite Element Modeling and Analysis

MIDAS/Civil is used in this study to model the pedestrian cable suspended bridge. This widely used Commercial Finite element package is superior in modeling and analysis of suspended footbridges compared to its competent software's like SAP2000. This software is used for the present study due to its ease of modeling with suspended bridge wizard function and its ability to perform Eigen value analysis considering the geometric stiffness induced due to initial dead load cable tension [7].

The numerical model consists of cable and frame elements. The main cables and suspenders are cable elements because these members only carry tension forces. Abutment and deck are modeled frame element. The suspenders are modeled as undeformed cable elements that connect the main cable to the crossbeam. The main cables are modeled based on the maximum vertical sag in the deformed shape. The abutment, crossbeams and decking panels are modeled as 3D frame elements.

MIDAS/Civil analyses were conducted to determine the natural frequency and vertical dynamic response of the structure. First, a dead load analysis was run because all pedestrian loading occurs after the self-weight is applied to the structure. Then, time-history analyses were run to determine the vertical dynamic response.

Generally, the methodology of the research activities from desk study and bridge model development to data analysis and discussion involved in this study are summarized and presented in six phases as shown in figure 1.5.

- **Reading and Observation**

- Find the problem through reading and observation of the interest area. then, identify statement of problem
- clearly identify the objective, aim and scope of the project
- Plan methodology for the study

- **Desk Study**

- Clearly understand project background
- Clearly understand the basic concept of human induced vibration
- Identify the design guidelines to apply the proper load function
- Identify the finite element software that is proper in this study

- **Selection of Cable Bridges for Case Study**

- Identify the suitable bridge for case study
- Collect the bridge data information
- Select certain changing design parameter and identify total number of cable bridge to be modeled in software

- **Data Analysis**

- Perform Modal analysis to determine natural frequency
- Determine pedestrian time forcing function
- time-history analyses carry out by apply time forcing function along the length of bridge maintaining constant pedestrian velocity, then check for acceleration time response and determine peak or maximum acceleration

- **Results and Discussions**
- Discuss the modeling issue involve in study
- Discuss the results of the analysis
- Compare the result with design guide recommended comfort limit

- **Conclusion and Recommendation**

- Draw an overall conclusion from the study results
- Give some recommendations for future research on this area

Figure 1.5 Summary of Methodology for the Study

2. LITERATURE REVIEW

2.1 History of Cable Bridge

The model of suspending a rope as main load carrying element has been used for a very long time. An example of such a bridge can be seen in Figure 2.1.

Figure 2.1 A replica of a traditional Inca rope bridge [16].

Pedestrian Cable suspended bridges has been known and in use in ancient times. The wide use of natural rope bridge by early man led to today's modern suspension cable bridge. This type of construction is widely used and is indigenous in south East Asia, South America and Equatorial Africa, and was probably in use in these regions long before the earliest known contemporary record, which related to suspension bridge over the Indus River, around 400 A.D [8].

Probably nature provided the first river crossing bridge, a tree with a branch stretching across to the other side of the stream, accidentally fallen trees connecting the two banks of a stream, the natural arch formed by erosion of soil beneath and many other imaginable provisions.

Primitive suspension bridges are also most likely the first bridge in tropical environments where plants like vines creepers and bamboos were abundant. The simplest and the most primitive of such bridges are the earliest suspension bridges, which consists of a single cable only. The traveler sits in a loop and pulls himself along. Creepers of various kinds are used by the native of Equatorial Africa for their suspension bridges, which are often little more than assemblies of ropes between tree tops, though primitive anchorages by stakes in the ground occurred sometimes [8].

The development of these primitive suspension bridges of natural ropes is then replaced with iron chains first in China, around 1632 and western people became interested in the introduction of wrought Iron first, as in China, in the form of chains. In England wrought iron chains were first forged on large scale for use as anchor chains for ships, and suspension bridges using such chains tend to arise near the early shipyards.

It is believed that the first chain bridge in England was erected in 1741. It was of primitive nature just 2 ft wide foot-bridges, and it collapsed in 1802. Most of these early chain bridges suffered oscillations in high winds and some collapsed as a result. The early bridges were said to have one behavior in common, they all swayed and sagged so much that nervous travelers had to be blind folded, tied to a stretcher and carried over. The gradual development of Cable Bridge led to the construction of progressively economical and more slender structures. Cable Suspended pedestrian bridges are economical and simple in construction, which can be constructed with locally trained workmanship [9].

Due to these advantages cable suspended pedestrian bridges are widely used now a day in developing and under developed countries like Zimbabwe, Cambodia, Nepal and Ethiopia etc. by different local and international NGO's in cooperation with local people.

2.2 Modern Pedestrian suspended bridges

Pedestrian cable suspended bridge is a modern version of the traditional Chain bridge. This type of bridge has downward sagging walkway which is suspended below their anchorage cables. The cables are directly anchored to the main anchorage foundation using small pillars for handrail cable support.

A bridge with the main foundation on the same elevation is called a level bridge. The main foundations might not have the same elevation and the bridge is therefore called an inclined bridge. The hand rail cables are always secured with thimbles and bulldog grips to adjustable anchorages. The main foundations are usually designed as gravity foundations on soil or rock. Anchorage rods maybe provided to stabilize the foundation on rock.

The main components of suspended cable bridges are walkway, handrail, main cables and anchorage foundation, shown in Figure 2.2. The handrail and the main cables are the load bearing elements connected throughout the bridge with hanger rods at constant interval. The hanger rods are fixed at the top to the handrail cable and at the bottom to the cross-beams which are bolted to the main cables. The cross-beams support the walkway deck Chain-link wire mesh netting fences the walkway. It is fixed at the top to the handrail cables and at the bottom to a fixation cable. The superstructure is completely unstiffened and thus allows some reasonable degree of lateral, vertical and torsional vibration. Therefore, stabilizing gauges are required and must be provided by means of wind guy arrangement. Bridge up to 50m spans may be designed without wind guy arrangements. The suspended bridge is an economical design whenever the required free board can be achieved along with the geological site conditions allowing its construction [6].

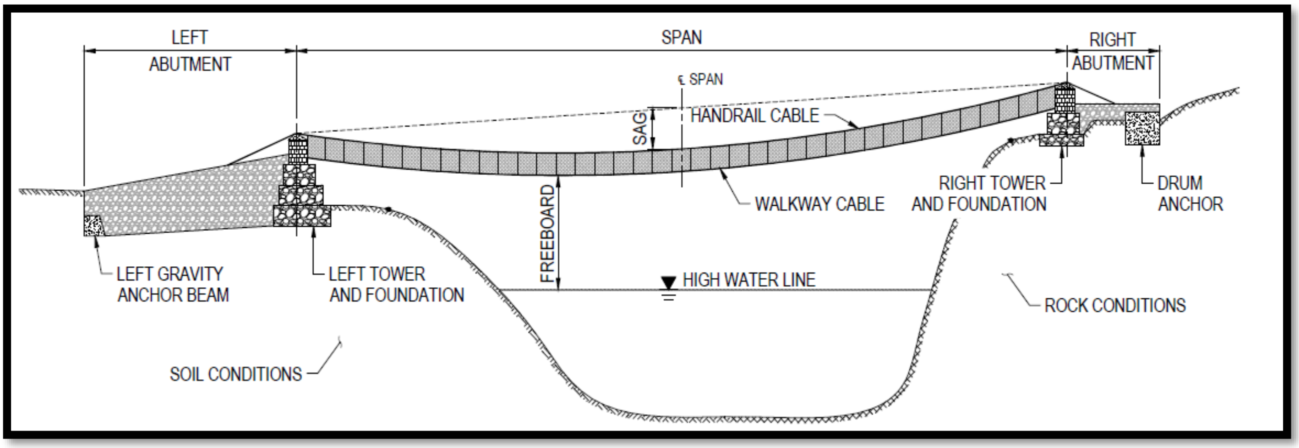


Figure 2.2 Cable suspended Pedestrian bridge typical Elevation View [6].

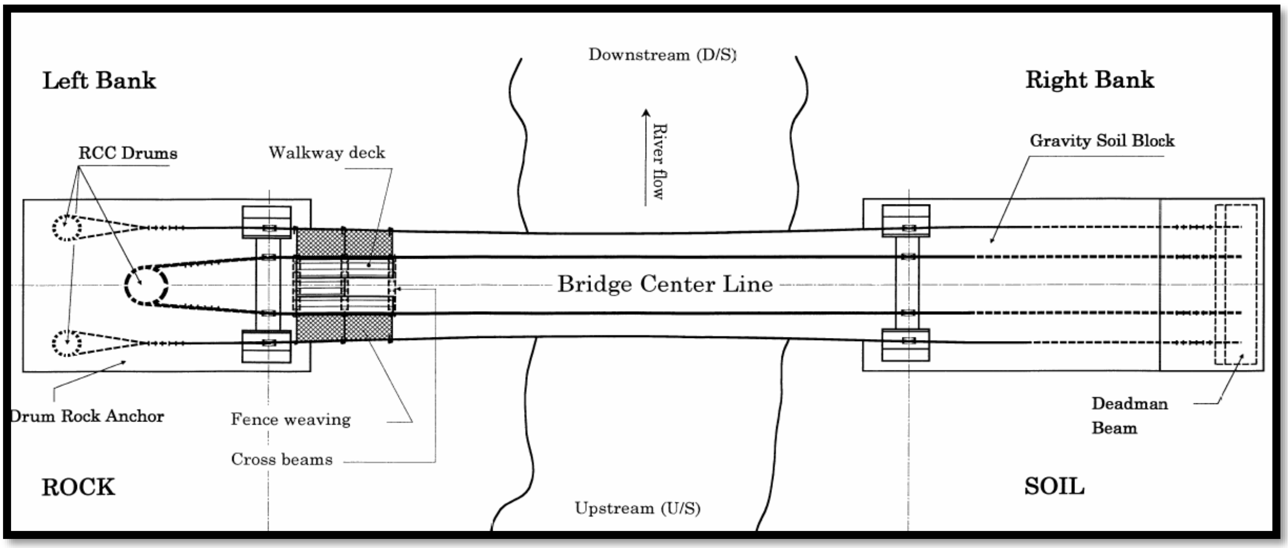


Figure 2.3 Cable suspended Pedestrian bridge typical plan View [6].

Cables are main structural component which are used as walk way support and hand rail. Cables are made flexible and strong structural members that carry axial loads in tension. They do not carry compression. The hand rail and walk way supporting cables are connected by vertical cable hangers of the same height at equal interval. The voids between successive hangers are filled by wire mesh. Meshing between hangers is made to protect people from falling sidewise.

The walk way system of the pedestrian suspended cable bridge is a service giving part of the bridge in which people walk on it and cross the river. The component of the walkway may be timber or steel fixed to the lower chord.

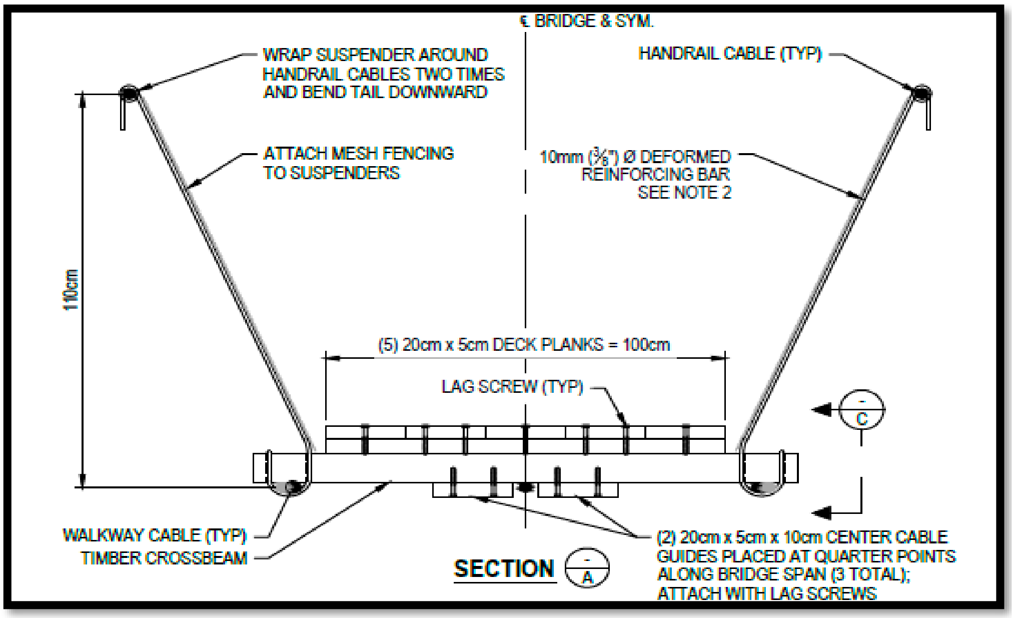


Figure 2.4 Cable suspended Pedestrian bridge walk way systems [6].

Foundation and Anchor block is a structural component of suspended bridge that anchors the cable outwards and downwards at the end of the bridge. Beside the configuration of the cables, Cable Bridge can also be distinguished by the manner the cable system is anchored at the ends.

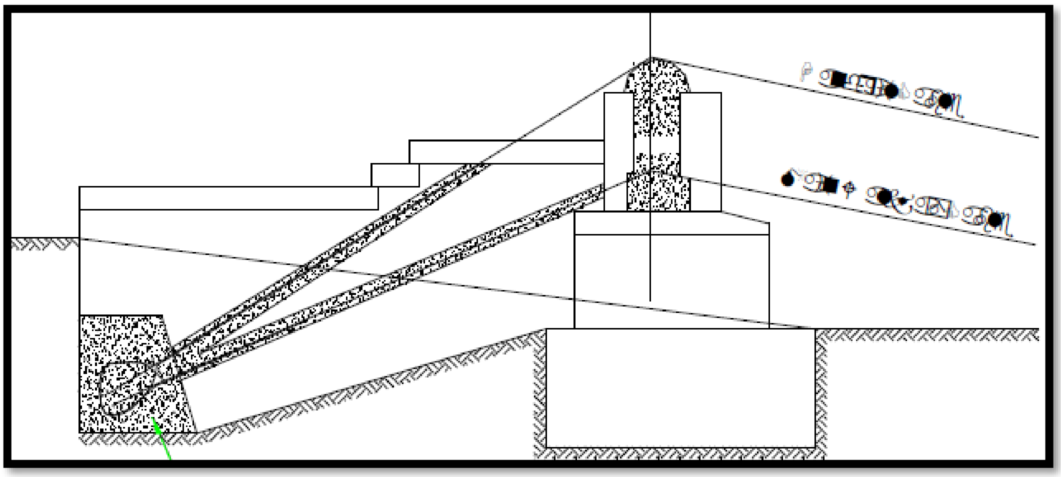


Figure 2.5 Cable suspended Pedestrian bridge Gravity block anchor [6].

Bridge foundation transfers all loads from the bridge superstructure to the foundation soil safely. During foundation design a number of failure mode are analyzed by different models. Because subsoil conditions vary in a great deal it is necessary to determine the local subsoil parameters by means of geotechnical investigation. Such as, pits, laboratory investigation of samples etc. before foundation design is carried out [6].

The choice of construction materials to be used for cable suspended pedestrian bridge affects the final project cost and construction methodology. The choice of material is governed by local availability, the cost and ease of the transportation to the site, the degree of workmanship to be employed, the degree of supervision, quality control availability, durability and maintenance of cable bridges.

The strengths and properties of every cable type depend on the properties of the individual wires comprising the rope as well as the rope construction. The cable type used in these type of cable bridges is the helical (spiral strand) type and the nominal size of to be used are 13mm, 26mm, 32mm and 48mm for different span range. The wire rope is a flexible, multi-wired, stranded precision product. It is composed of wires, strands and core. In general, any numbers of multi wired strands are helically arranged around the core [6].

Cables are long, slender, flexible structural members that are designed to carry axial loads in tension. Cables to be used in cable supported bridge made up of the steel wire characterized by considerably large tensile strength than that of ordinary structural steel. The steel wire is of cylindrical shape with diameter of 3 to 7mm. The steel material for the wire is manufactured with a chemical composition characterized by a higher carbon content than allowed for structural steel. In the chemical composition, the high carbon content of cable steel, about four times that of structural steel. This high content of carbon makes the cable Steel unsuited for welding.

The main advantage of using a cable as a load carrying element lies in the most efficient load transfer by pure tension. A cable carrying transverse load, the geometrical configuration is decisive due to the fact that the axial force which is horizontal force at mid span is inversely proportional to the sag. From this it follows that a straight cable is unable to carry any transverse load as zero sag will imply infinitely large cable force.

Cable that is transversely loaded will have to be supported vertically and horizontally. The large horizontal reactions required at the supporting point of the main cables will make it impossible to support them on vertical columns only. It will be necessary to continue the main cables as back stays to anchor blocks positioned in some distance from the masonry support [6].

2.3 Human-induced dynamic loads

Human induced vibration can occur in many common engineering systems and if uncontrolled or not dissipated can lead to catastrophic results. Vibrations of a bridge structure under Human induced loads are an important consideration in the design of cable suspended pedestrian bridges. The possibility of elastic failure of a structure with dynamic effects are neglected and in long time repetition of dynamic stresses may lead to cumulative fatigue failures even it would be considered to be safe from static considerations.

As footbridges are mainly designed for the conveyance of pedestrians and cyclists, the dominant design loads are those induced by pedestrians. Since the pedestrian loads are produced by different kinds of human activities such as walking, running, jumping and bouncing as well as other human movements, these loads are in fact human-induced dynamic loads and it was noted very early that these dynamic excitations could cause excessive vibration, and in extreme cases even a collapse of the structure [10].

The Human-induced dynamic response of cable suspended pedestrian bridges are dependent on a number of factors which include the main dynamic parameters of the bridge structure, the frequency content of the loads transmitted by the human activities and the types of suspension systems used. Among those several factors, this study investigates effect of human-induced vibrations on pedestrian cable bridges.

For modern footbridges, these human-induced dynamic excitations could cause serious serviceability problems rather than problems of safety, especially for the slender footbridges with natural frequencies lower than 5.0 Hz, independent of the structural form.

2.3.1 Characteristics and measurements of pedestrian loads

During walking on a structure, pedestrians induce dynamic time varying forces on the surface of the structure. These forces have components in all three directions, vertical, lateral and longitudinal and they depend on parameters such as pacing frequency, walking speed and step length. Dynamic forces induced by humans are therefore highly complex in nature [11].

This study has paid more attention to the vertical component of the dynamic force than the horizontal component. This is because until the opening of the Millennium Bridge, almost all documented problems with pedestrian-induces vibrations were associated with vertical forces and vibrations.

The typical pacing frequency for walking is around 2 steps per second, which gives a vertical forcing frequency of 2 Hz. Slow walking is in the region of 1,4 - 1,7 Hz and fast walking in the range of 2,2 - 2,4 Hz. This means that the total range of vertical forcing frequency is 1,4 - 2,4 Hz with a rough mean of 2 Hz [12].

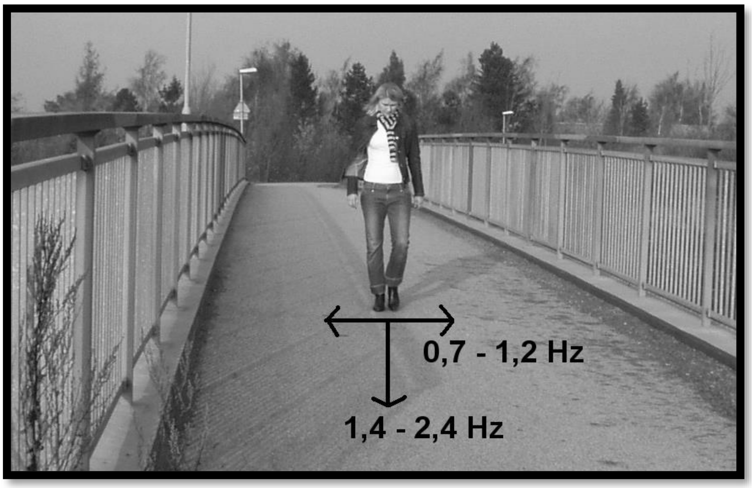


Figure 2.6 Vertical and horizontal forcing frequencies [13].

Many footbridges have natural vertical and lateral frequencies within the limits mentioned above (1.4 – 2.4 Hz vertical and 0.7 – 1.2 Hz horizontal). They have therefore the potential to suffer excessive vibrations under pedestrian actions. The necessity to consider horizontal as well as vertical pedestrian excitation is therefore obvious [12].

Several measurements have been conducted to quantify vertical loads imposed by pedestrians on structures. Most measurements indicate that the shape of the vertical force produced by one person taking one step is of the kind shown in Figure below [10].

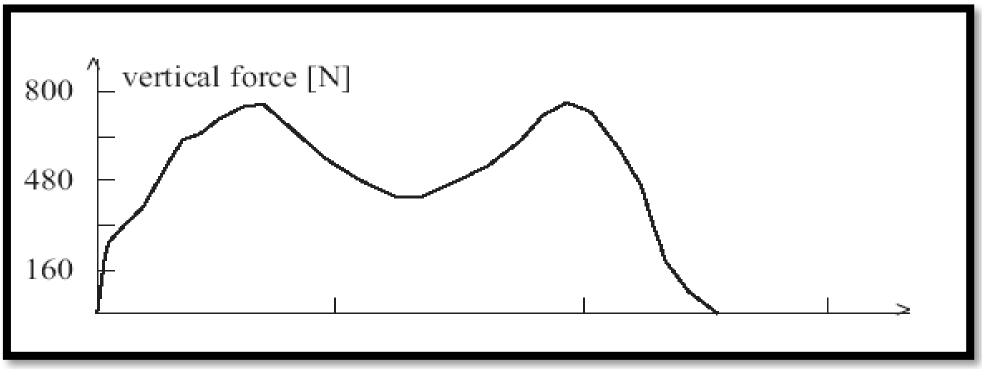


Figure 2.7 Vertical force produced by one person taking one step [10].

Measurements of continuous walking have also been made. The measured time histories were near periodic with an average period equal to the average step frequency. General shapes for continuous forces in both vertical and horizontal directions have been constructed assuming a perfect periodicity of the force.

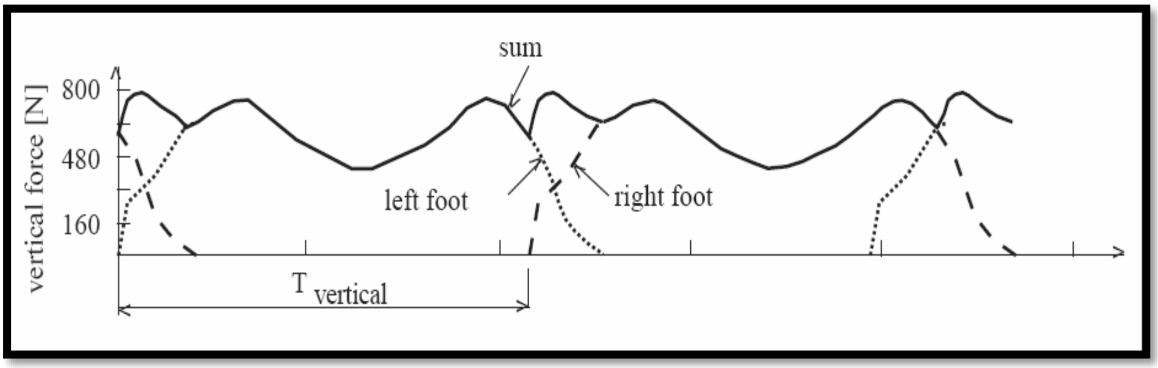


Figure 2.8 Periodic walking time histories in vertical directions [10].

During walking, the vertical component shows a characteristic double hump, which is the result of the impact of the heel on the ground (first one) and the push off (second one). The maximums increase with increasing step frequency.

In this research work the basic objective of modeling and analysis of the bridges for human induced load is to investigate the effect of human-induced vibrations on pedestrian cable bridges. And provide critical number of pedestrians which satisfies design requirement according to different guidelines or codes.

2.3.2 Modeling of Pedestrian Loads

It is necessary to model the human-induced forces analytically in order to apply them into the dynamic analysis of structures. However, this is a complicated task, since the footfall forces induced by human activities are affected by many factors such as pacing rate, subject's weight, footwear and surface condition of the dynamic loads [15], and they are different from person to person, and change not only in time but also in space, as well as with human-structure interaction. Considering the common features of human-induced forces, the force models based on some justifiable assumptions do exist and are used in contemporary design.

It is believed that dynamic force induced by footfalls is a summation of the forces produced by continuous paces and may be simulated by pulse trains created by a single footstep force. This assumes that the force from each footstep is approximately the same and that the time the feet overlap is kept constant for a given pacing rate, i.e. the force has a periodic nature. For this reason, the dynamic force induced by a single person is always represented by means of Fourier series which decomposes the periodic force into distinct harmonic components.

2.3.3 Effect of Group of People

A footbridge is rarely submitted to forces due to one pedestrian only. Groups of pedestrians are much more common. Each person has its own characteristics (weight, speed, frequency, initial phase shift, location on the bridge) which make the system much more complicated. Moreover, dependent of the number of pedestrians on a bridge, people tend to walk more or less synchronous with each other, during which the natural pace of pedestrian changes. This behavior is nearly impossible to model correctly [12].

According to the statistics, the load can be increased with the square of the number of pedestrians on the bridge$\sqrt{N}$. This means that a number of $\sqrt{N}$ are walking synchronously. However, this is only the case if initially none of the motions on the bridge are synchronous. In reality a (small) part of the crowd will be synchronized.

2.3.4 Sensitivity of pedestrians to vibrations of footbridges

Both vertical and horizontal vibrations can be perceived as a disturbing effect during the stay on the bridge and can therefore considerably influence the serviceability limit state of the bridge. Perception of vibrations is a rather complicated topic, as it has many influencing factors where human psychology plays an important role. Each person senses a vibration differently, but this is also dependent of the moment when the vibrations are perceived, the eventual sounds from the structure or even the height above the ground. Pedestrians can also get used to vibrations over the time and acceptance regarding to vibrations can rise.

Experiments have shown that pedestrians are more sensitive to vibration when standing still than when walking. As can be seen, the comfort criteria are being expressed in accelerations of the vibrations. The acceleration, the speed and the displacement of the vibrations are closely related with each other through the dynamics formula. One will therefore find the comfort criteria according to one of these factors. It becomes clear that a standing person is more sensible to vibrations than a walking person. However, this does not imply that standing persons should not feel any vibrations of passing pedestrians. This could lead to an uneconomical design of the bridge.

2.3.5 Pedestrian Loading and Structure Interaction

The dynamic response of footbridges changes when pedestrians are present on the structure. Moving pedestrians increase the mass and damping of flexible footbridges with light timber floors. This is due to the fact that the mass of people is significant compared to the mass of the structure. Walking crowds can increase the damping of the structure in the vertical direction; however, there is limited data to quantify this effect, and data for lateral dynamics of footbridges with moving people is very scarce. In addition, jumping and bouncing can change dynamic properties. Jumping forces are about two times less on flexible footbridges than on rigid structures [12]. The present study does not model pedestrian and structure interaction.

2.4 Natural vibration modes

A mode of vibration is a characteristic pattern or shape in which a bridge vibrates. Footbridges can have many vibration modes which can be determined by a modal analysis. The actual vibration of a footbridge is generally a combination of all the vibration modes. Some examples of vibration mode are shown in Figure 2.9.

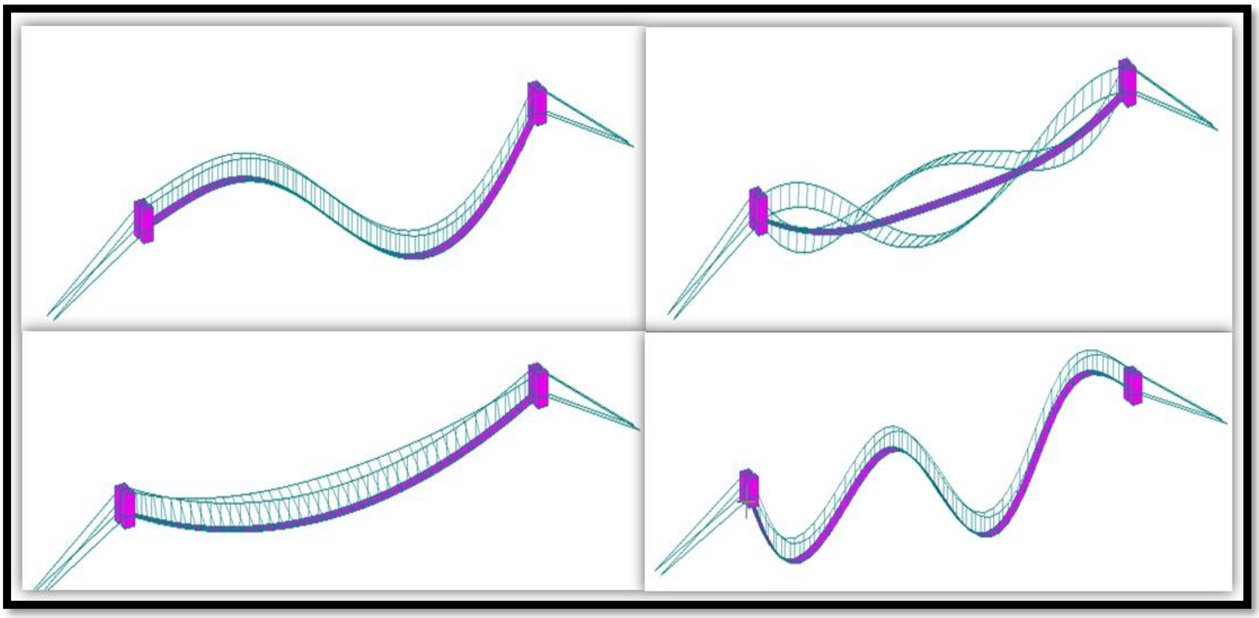


Figure 2.9 Examples of vibration modes.

2.5 Serviceability Limits

While strength limits are very important for structural design, serviceability limits are as well, especially for modern suspended footbridges. Footbridges are being built with longer spans and greater slenderness due to the reduction in weight of bridge elements. These types of bridges have low stiffness, low mass, and low damping. Suspended footbridges have low modal frequencies and are therefore susceptible to pedestrian loading that occurs at low frequencies. Under typical pedestrian loading, suspended footbridges are at risk of reaching resonance or exceeding human tolerance levels for comfort [11].

While most footbridges are designed to withstand strength criteria, some footbridges have not been designed to satisfy serviceability limits. Pedestrians must use footbridges for the structure to fulfill its purpose; however, in the process of walking across a footbridge, pedestrians create vibrations that cause the structure to move or twist in all directions. If the bridge has excessive movements, the pedestrians become uncomfortable, resulting in a serviceability failure.

Suspended bridge analysis has changed over the years, but serviceability analysis of suspended bridges continues to be a problem. Suspended footbridges can have a large dynamic response to pedestrian loading because of their low modal frequencies. In addition, numerical models can be used to study footbridge dynamics. Pedestrian loading must be applied to determine the response of the footbridge, and this response must be compared to serviceability limits to determine if the footbridge meets human comfort criteria.

2.6 Synchronization of Pedestrians with Bridge Vibrations

While pedestrians have certain tolerance levels, they also can subconsciously add to the dynamic response of the bridge through synchronization. High densities of people can add to synchronous excitation when they walk together with a frequency that matches the low frequency of the footbridge. When the footbridge starts to resonate, pedestrians have a tendency to change their walking frequency to match the vibration of the bridge. This escalates the vibration and adds to the discomfort of the users [11].

Sometimes pedestrians are limited in their movement on footbridges. When people walk in small groups, they tend to all walk at the same velocity. Therefore, each person walks with a different frequency because their step length varies. However, when footbridges are exposed to a crowd of people with a density between 0.6 and 1.0 pedestrians/m^2, free walking is limited, and pedestrians are forced to adjust their step length and velocity to the group. This is typically when Synchronization occurs, which can lead to structure serviceability problems [11].

Vertical synchronization of pedestrians with footbridge vibrations is less common than lateral synchronization and more difficult to measure. Therefore, there are several ranges of predictions for the probability of pedestrians synchronizing to vertical vibrations. One study suggested a probability of synchronization of 22.5 percent for a bridge with a frequency of 2

Hz. However, other studies predicted higher percentages. While there are many equations that attempt to characterize pedestrian synchronization, more research is needed to determine the relationship between the number of pedestrians, walking speed, walking frequency, and probability of synchronization [11]. Vertical synchronization will not be modeled for the present study.

2.7 Review of the Codes

New lightweight and high-strength structural materials, longer spans and greater slenderness of footbridges have been caused several problems with vibration serviceability; this section will discuss how these problems are dealt with in current standards and codes of practice.

The main focus in this section will be on the serviceability criteria and the load Models proposed by three widely used standards.

2.7.1 Eurocode

The Eurocode is a set of building codes developed by the European Committee for Standardization.

Eurocode 0 (EN 1990:2002 “Basis of Structural Design”) [4], states that the comfort criteria should be defined in terms of maximum acceptable acceleration. The amplitude of the vibrations are directly related to the acceleration. Even though the acceptable acceleration criteria can be defined by the national annexes, some recommended maximum values are given. These values are given in Table 3.1.

Table 2.1 Recommended maximum Acceleration, EN 1990:2002 Annex A2.

Vertical vibrations	0.7 m/s^2
Horizontal vibrations (normal use)	0.2 m/s^2
Horizontal vibrations (crowd conditions)	0.4 m/s^2

Table 2.2 Recommended Fundamental Frequencies, EN 1990:2002 Annex A2.

Vertical vibrations	< 5 Hz
Horizontal and torsional vibrations	< 2.5 Hz

Eurocode 1 (EN 1991-2:2003 “Actions on Structures”): Paragraph 5.7 mentions three points of attention:

- Depending on the dynamic characteristics of the structure, the relevant natural frequencies (corresponding to vertical, horizontal, torsional vibrations) of the main structure of the bridge deck should be determined from an appropriate structural model.

- Forces exerted by pedestrians with a frequency identical to one of the natural frequencies of the bridge can result into resonance and need to be taken into account for limit state verifications in relation with vibrations.

- Appropriate dynamic models of pedestrian loads and comfort criteria should be defined.

Eurocode 3 (EN 1993-2:2006 “Design of Steel Structures”): Paragraph 7.9 mentions, for footbridges and cycle bridges with excessive vibrations could cause discomfort to users, measures should be taken to minimize such vibrations by designing the bridge with appropriate natural frequency or by providing suitable damping devices.

2.7.2 American Association of State Highway and Transportation Officials (AASHTO)

The American Association of State Highway and Transportation Officials (1997) provide limits for fundamental frequencies in the Specification for Pedestrian Bridge Design. The fundamental frequency in the vertical plane of a pedestrian bridge without live load must be greater than 3 Hz, and the fundamental frequency in the lateral direction must be greater than 1.3 Hz [4]. If the fundamental frequency cannot satisfy these limitations, or if the second harmonic is a concern, an evaluation of the dynamic performance shall be made. This evaluation shall consider.

- The frequency and magnitude of pedestrian Footfall loadings

- The phasing of loading from multiple Pedestrians on the bridge at the same time including the “lock-in” phenomena

- Appropriate estimation of structural damping

- Frequency dependent limits on acceleration and/or velocity

Therefore, codes have limits on frequencies that fall within a typical range but do not exactly agree on the frequency range; also, the codes propose design procedures to evaluate the footbridge performance against serviceability limits.

2.7.3 British National Annex for Eurocode 1 of EN 1991-2

The British National Annex dealing with dynamic models for pedestrian actions on Footbridges is based on the research done by Barker and MacKenzie [2]. The aim of the UK design rules mentioned in the National Annex is to provide sufficient guidance to take into account the effects of vibration of complicated structures and those in sensitive locations without imposing undue conservatism that might constrain designers in achieving and economic solution.

The UK National Annex does not mention anything about the assessment of the natural frequencies and the structural damping. However, Barker and MacKenzie advise the following.

The designer is advised to explore the sensibility of the contribution of non-structural elements to investigate potential variation in structural response. Typical values for the structural damping are given.

Table 2.3 Recommended values of the damping ratio for fundamental modes.

Material of construction	damping ratio
Steel	0.03
Steel and concrete composite	0.04
Concrete	0.05
Timber	0.06 to 0.12
Aluminum alloy	0.02
Glass or Fiber Reinforced Plastic	0.04 to 0.08

The National Annex assumes an upper limit for the vibration serviceability: if the vertical natural frequency of the unloaded bridge is exceeding 8Hz and if the horizontal frequency of the loaded bridge is exceeding 1.5 Hz, then the vibration serviceability is deemed to be satisfied. If that is not the case, the designer is required to assess the likely dynamic response of the structure.

Two distinct analyses are required:

-The determination of the maximum vertical deck acceleration and its comparison with the comfort criteria, and

-An analysis to determine the likelihood of large synchronized lateral responses.

Bridges are classified in bridge classes according usage, from rural locations (Bridge class A) to primary access routes (Bridge class D) [2].

Table 2.4 Bridge classification according to UK National Annex to EN1991-2 [2].

Bridge Class	**Bridge usage**	**Group size (walking)**	**Group size (jogging)**	**Crowd density ρ (pers./m²)**
A	Rural locations used and in sparsely populated areas	N=2	N= -	0
B	Sub-urban location likely to experience slight variations in pedestrian loading intensity on an occasional basis	N=4	N=1	0.4
C	Urban routes subject to significant variation in daily usage (e.g. structures serving access to offices or schools)	N=8	N=2	0.8
D	Primary access to major public assembly facilities such as sports stadiums or major public transportation services	N=16	N=4	1.5

(N= Number of pedestrians in the group)

- **Dynamic actions representing the passage of single pedestrians and pedestrian groups (vertical)**

The design maximum vertical accelerations that result from single pedestrians or pedestrians' groups should be calculated by assuming that these are represented by the application of a vertical pulsating force F(N), moving across the span of the bridge at a constant speed v_t, as follows:

$$\boldsymbol{F = F_o K(f_v)\sqrt{1+\gamma(N-1)}\,\sin(2\pi f_v t)} \quad \text{……………………………………………} (2.1)$$

Where:

$F(t)$= Applied point load, representing a group moving at the speed V_t

N= Number of pedestrians in the group depending of the bridge Class

F_o =Reference amplitude of the applied force

f_v= Natural frequency of the vertical mode under consideration

k(fv)=Factor to deal with (a) the effect of a more realistic pedestrian population, (b) harmonic responses and (c) relative weighting of pedestrian sensitivity to response t = Elapsed time

γ= Reduction factor to allow for the unsynchronized combination of actions in a pedestrian group and is a function of damping and effective span length

Table 2.5 Parameters to be used, according to UK National Annex to EN1991-2 [2].

Load Parameter	Walking	Jogging
Reference load F_o (N)	280	910
Pedestrian crossing speed, V_t (m/s)	1.7	3

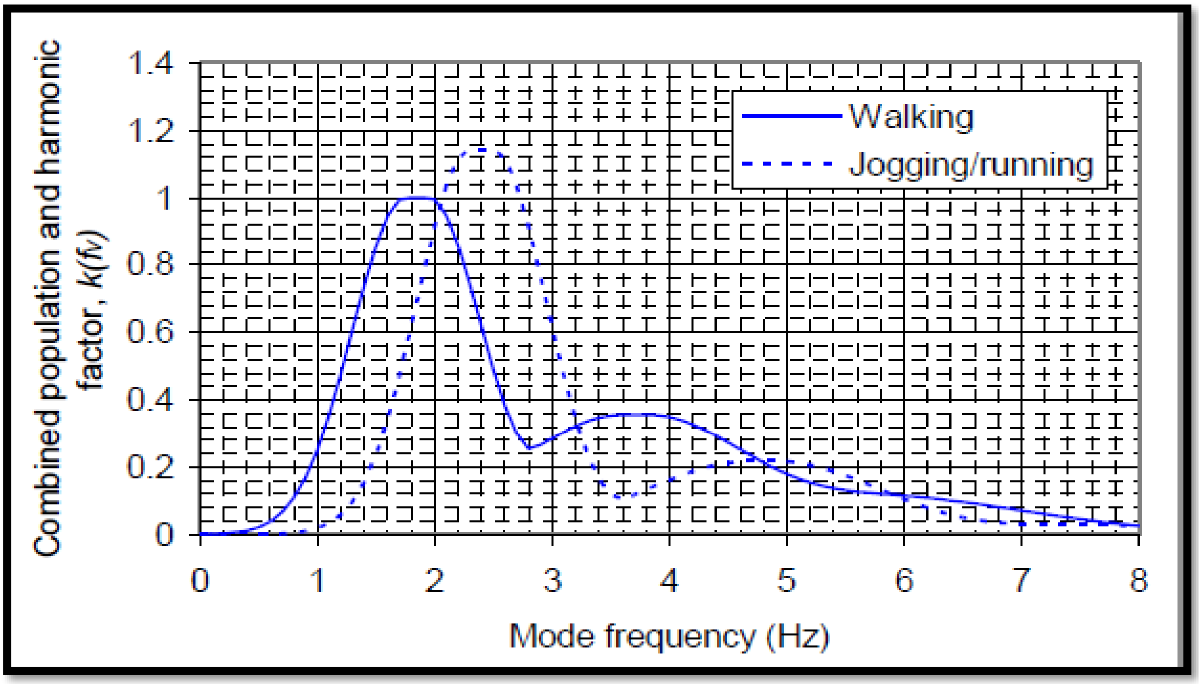


Figure 2.10 Relation between k(fv) and frequencies fv [2]

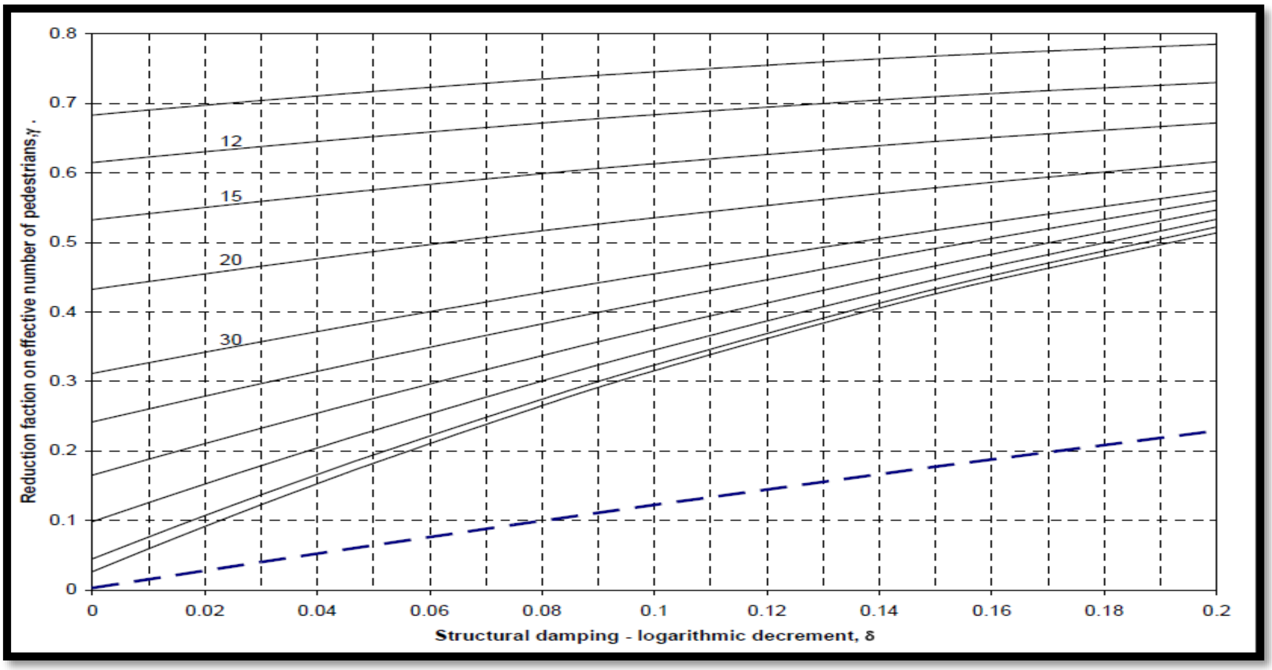


Figure 2.11 Reduction factors, γ [2]

The UK National Annex recommends a vertical design acceleration limit:

$a_{limit} = 1.0 k_1 k_2 k_3 k_4$ m/s^2 and 0.5 m/s^2 $\leq a_{limit} \leq$ 2.0 m/s^2

where:

k_1 = site usage factor, according to Table 3.6

k_2 = route redundancy factor, according to Table 3.7

k_3 = height of structure factor, according to Table 3.8

k_4 = exposure factor = 1.0 unless determined otherwise for individual project

Table 2.6 Recommended values for the site usage factor k1[2].

Bridge function	**k_1**
Primary route for hospitals or other high sensitivity routes	0.6
Primary route for school	0.8
Primary routes for sports stadia or other high usage routes	0.8
Major urban centres	1
Suburban crossings	1.3
Rural environments	1.6

Table 2.7 Recommended values for the route redundancy factor k2 [2].

Route redundancy	k_2
Sole means of access	0.7
Primary route	1.0
Alternative routes readily available	1.3

Table 2.8 Recommended values for the height of structure factor k3 [2].

Bridge height	k_3
Greater than 8 m	0.7
4mto8m	1.0
Less than 4 m	1.1

k_4 may be assigned a value of between 0.8 and 1.2 to reflect other conditions that may affect the users' perception towards vibration.

2.7.4 Codes Comparison

The two codes Eurocode and British National Annex for Eurocode 1 contain requirements for the serviceability limit state for vibrations.

However British National Annex for Eurocode 1 is only one which proposes pedestrian load models. The way to determine the responses of the bridge are left over to the designer.

The Eurocode expresses limits in maximum accelerations, the AASHTO determine the limit for fundamental frequency. The UK National Annex is the one proposes a speed at which the force is moving over the bridge this corresponds much more to the reality, as pedestrians do not load a bridge dynamically when standing still.

The UK National Annex introduces the Bridge Class for group of pedestrians, load models change for the different bridge classes. The load models presented in the UK National Annex only deal with vertical vibrations. A method is given to determine if the bridge is laterally stable, but this does not depend on force models.

Table 2.9 Vertical Natural frequency criteria.

Standard	**Vertical Natural frequencies**
Eurocode	$fv_{limit} \geq 5Hz$
AASHTO	$fv_{limit} \geq 3Hz$
British National Annex	$fv_{limit} \geq 8Hz$

Table 2.10 Vertical Acceleration criteria.

Standard	**Vertical acceleration**
Eurocode	$a_{limit} \leq 0.7$ m/s2
AASHTO	-
British National Annex	$0.5 \text{ m/s}^2 \leq a_{limit} \leq 2.0$ m/s2

3 MODELLING AND ANALYSIS

3.1 Introduction

Using computers engineers can analyze and design the structure with less error and time particularly for complex structures. Bridge structures can be analyzed through a variety of models in 2D and 3D.

In general, there are several basic methods for developing models of a bridge and all of those models give reasonable solutions for the first mode of vibration but differ significantly in their ability to predict subsequent modes. In this study, MIDAS/Civil was used to model and analysis pedestrian cable suspended bridges.

In this study, several numerical models were evaluated in parametric study to determine how walkway and handrail cable diameter and cable sag and number of pedestrians were affect 42m, 60m, 80m and 100m span pedestrian cable suspended bridges. Based on bridge to prosperity bridge builder manual the two extreme cable sag values considered are 5 and 7 percent of span length and the two extreme walkway and handrail cable diameter considered are 26mm and 32mm.The number of pedestrians taken for walking loads are 2, 4 and 8. For jogging load case 1 and 2 are the number of pedestrians considered for analysis. The numbers of pedestrians are selected from British National Annex for Eurocode 1 of EN 1991-2 manual based on three bridge locations which are rural, suburban and urban area.

The basic case study pedestrian cable suspended bridge which is located at Kolfe-Keranyo, Addis Ababa is presented in figure 3.1.

Figure 3.1 Basic case study pedestrian cable suspended bridge

3.2 Static Analysis

Before finite element modeling is carried out to investigate the effect of human induced vibration on pedestrian cable bridge, it is important to determine the capacity of suspended cable by static loads. So, in this section a sample static analysis of cables carried out.

The stress-strain behavior of cables is, due to its cross-section with much void, not linear. When cables are loaded, the void between the strands decreases with a change in the cross-sectional area as result. This will lead to both permanent and temporary deformations. Permanent deformations often arise in the beginning of the lifetime of a cable. The reason for this is when the cable is subjected to a load; a strain hardening takes place when the void between the strands closes. This will lead to an increased axial stiffness and permanent elongations. Besides the permanent deformations there will always be elastic deformations in the cable as well. Since cables are subjected to large deformations, it’s necessary to formulate the equilibrium conditions in the deformed shape of the cable.

3.2.1 Cable Analysis

A cable hanging between two supports and carrying a uniformly distributed load along its length (as opposed to the true horizontal dimension) forms a catenary. However, the cable can be analyzed as a parabolic curve for simplicity since the difference between catenary and parabolic profiles is negligible in the range of sag values used for suspended cable bridges. Suspended cable forces decrease with increased sag percentages (i.e. when cables are lower), this restriction is imposed to limit the inclination of the walkway surface for serviceability. Conversely, because cable forces increase with decreased sag percentages (i.e. when cables are more straight across the river), the resulting forces from cables with less sag requires larger foundations and the eccentricity of the cable forces on the abutment towers increases [1].

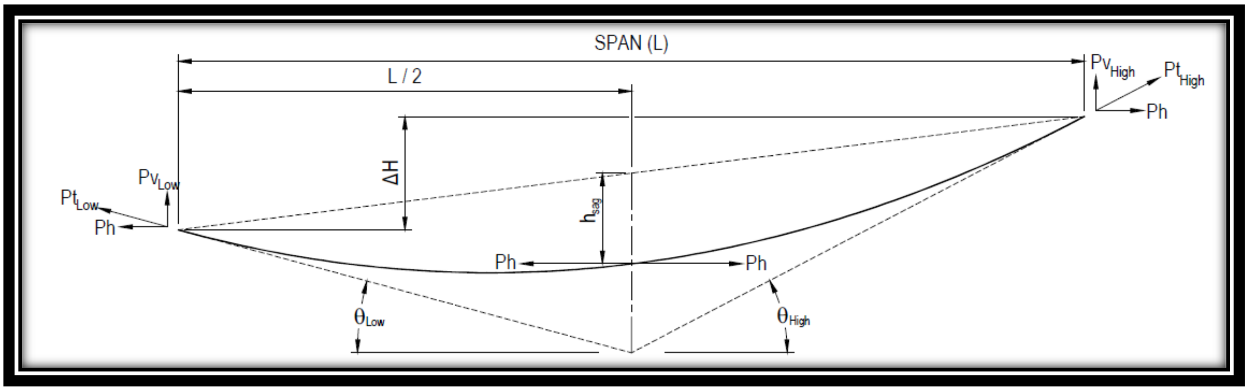


Figure 3.2 Cable Geometry and Forces for a Cable Subjected to Distributed Load [1].

By using simple mathematics, the following equation can be drive:

Horizontal Tension:

$$P_h = \frac{W_c * L^2}{8 * h_{sag}} \qquad (3.1)$$

Where:

P_h = horizontal cable tension, kN
W_c = distributed load, kN/m
L = bridge span length, m
h_{sag} = cable sag, m

Angle to Horizontal (High Side):

$$\Theta_{High} = \tan^{-1} \left(\frac{(4 * h_{sag} + \Delta H)}{L} \right) \qquad (3.2)$$

Vertical Tension (High Side):

$$PV_{High} = P_h * \tan (\Theta_{High}) \qquad (3.3)$$

Total Tension (High Side):

$$Pt_{High} = \frac{P_h}{\cos (\Theta_{High})} \qquad (3.4)$$

Angle to Horizontal (Low Side):

$$\Theta_{Low} = \tan^{-1} \left(\frac{4 * h_{sag} - \Delta H}{L} \right) \qquad (3.5)$$

Vertical Tension (Low Side):

$$PV_{Low} = Ph * \tan(\Theta_{Low}) \qquad (3.6)$$

Total Tension (Low Side):

$$Pt_{Low} = \frac{P_h}{\cos (\Theta_{Low})} \qquad (3.7)$$

In this study, Cable sag (ΔH) =0. Therefore, $Pv_{high} = Pv_{low}$ *and* $Pt_{high} = Pt_{low}$

Cable design shall satisfy:

$$P_s \leq \frac{P_u}{FS} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.8)}$$

Where:

Ps = maximum axial tension in cable, KN
Pu = ultimate breaking strength of cable, KN
FS = factor of safety = 3.0

Cable size and quantity selection example for span of 42m and 100m Cable suspended footbridges are presented in Appendix (D and E).

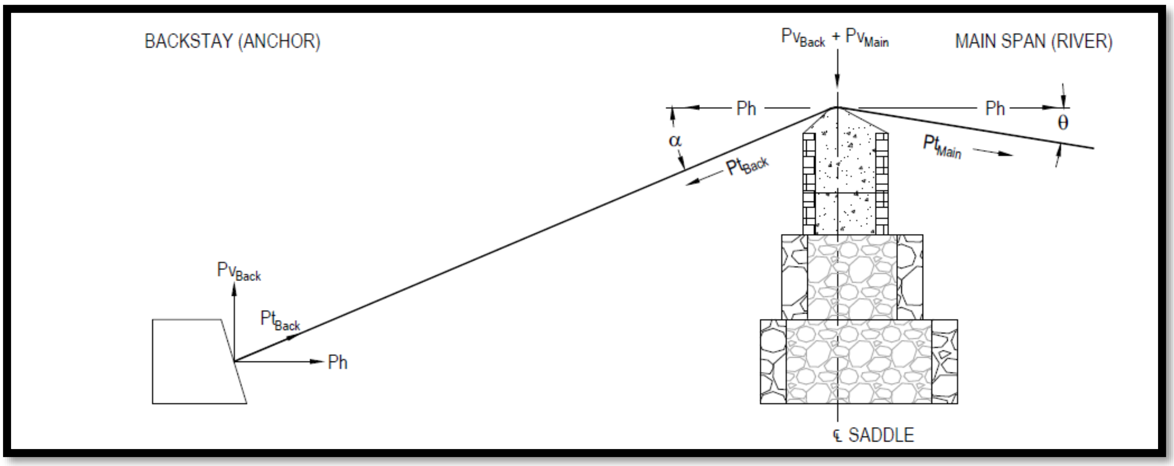


Figure 3.3 Backstay Cable Geometry and Forces [1]

Total Backstay Tension:

$$Pt_{Back} = \frac{Ph}{\cos \quad (\propto)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.9)}$$

Vertical Backstay Tension:

$$PV_{Back} = Pt_{Back} * \sin \quad (\propto) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.10)}$$

Total Main Span Tension:

$$Pt_{Main} = Pt_{High} \; or \; Pt_{Low} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.11)}$$

Vertical Main Span Tension:

$$PV_{Main} = Pt_{Main} * \sin \quad (\theta) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.12)}$$

Total Vertical Reaction at Tower:

$$R_{Tower} = PV_{Back} + PV_{Main} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.13)}$$

3.3 Finite Element Modeling

Finite element analysis is an effective method of determining the dynamic performance of structures for three reasons which are saving in design time, cost effective in construction and increase the safety of the structure. A finite element analysis is a numerical simulation of the behavior of a real-world structure which is intended to provide information that can be used by a designer or design team to ensure a structural design is fit for purpose when it enters in-service operation.

This research work presents a finite element method of analysis which basically consists of dividing the bridge decks and beam elements into a number of parts, called elements. The modeling of the bridges consists of key points/nodes, lines, areas and volumes with increasing complexity. These are connected to each other at their nodes as each node may have six degree of freedoms which are defined as independent displacements (three translation and three rotations) used to describe the movement of each node.

The bridge system is modeled using bridge structure analysis software, MIDAS/Civil. The models are based on member dimensions, design material and section properties from selected footbridge. But different span length, cables diameter, cables sag and number of pedestrians were taken to correctly investigate the effect of human-induced vibrations on pedestrian cable bridges. The software incorporates bridge design wizard in order to input the bridge properties. The numerical model consists of cable and frame elements. The main cables and suspenders are cable elements because these members only carry tension forces. Abutment and deck are modeled frame element. The suspenders are modeled as undeformed cable elements that connect the main cable to the crossbeam. The main cables are modeled based on the maximum vertical sag in the deformed shape. The abutment, crossbeams and decking panels are modeled as 3D frame elements.

In this study, suspended footbridges the mailers, which are angle steel attached to the double angle crossbeams. For ease of nailing the decking boards are not modeled in MIDAS/Civil explicitly. Instead, the mailers are represented as a distributed dead load centered on the crossbeams that acts along the length of the standard mailer.

In constructing a finite element mesh for a dynamic analysis there are a range of aspects of the problem that have to be considered. The stiffness distribution is important in static analysis whereas for dynamic analysis, the mass distribution is the most important.

In this study, the models are discretized into a minimum of one-meter edge length. The total number of elements increases as the span length increase. Minimum of 309 elements used for 40m span length and maximum of 714 elements used for 100m span length. Figure shows the typical finite element discretization for the bridge models.

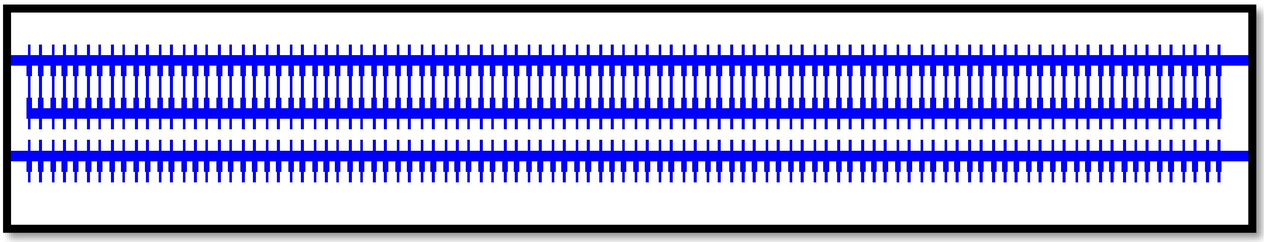


Figure 3.4 Typical finite element discretization**.**

3.4 Materials and section Properties

3.4.1 Materials Properties

In MIDAS/Civil analysis elements were selected to correctly represent the behavior of the Pedestrian Cable Bridges. The structural material used in this research taken from selected pedestrian cable suspended bridge.

Table 3.1 Material Definitions for Suspended footbridges.

Element	Material	Modulus of Elasticity (KN/m2)	Density (KN/m3)
Abutments	Stone Masonry	2.70E+07	21.00
Crossbeams	S235 (Steel)	2.10E+08	76.98
Suspenders	S235 (Steel)	2.10E+08	76.98
Deck	S235 (Steel)	2.10E+08	76.98
Cable	S235 (Steel)	2.10E+08	76.98

3.4.2 Section Properties

All element sizes are the same for all model analysis. Two cable diameter sizes (26mm & 32mm) and two cable sags (5 percent and 7 percent) are used for dynamic response comparison purpose since cable element is the main load resisting component for cable suspended footbridge.

Table 3.2 Section Definitions for Suspended footbridges.

Element	Type	Dimension
Abutments	Solid Rectangle	1000x700 (mm)
Crossbeams	Angle	L40×40×5
Suspenders	Solid Round	10mm
Deck	Angle	L40×40×3
Cable	Solid Round	Varies (26mm & 32mm), Number =4

3.5 Dynamic Analysis according to UK National Annex

In this study, The UK National Annex proposes moving loads (vertical only) for pedestrians which are moving along the most unfavorable line across the bridge for both walking and jogging pedestrians are considered. According to the annex the dynamic Load Factors which are applied to the loads vary based on the type of pedestrian (walker, jogger), the number of pedestrians and the frequency of the considered mode. The velocity of the dynamic load dependent on the type of pedestrians.

According to the UK National Annex, the selected pedestrian cable suspended bridge can best be categorized in bridge class B, as it is built in Suburban location likely to experience slight variations in pedestrian loading intensity on an occasional basis. But, for vertical dynamic response study and comparison purpose assumed the selected bridge located rural and urban area in addition to suburban area. So, in this study, based on bridge location, different numbers of pedestrians considered for both pedestrian walking and jogging load cases. The code recommends the following values for given formula to be considered in the analyses:

$$\boldsymbol{F = F_o K(f_v)\sqrt{1 + \gamma(N - 1)}\sin(2\pi f_v t)} \text{ .... } (3.2)$$

Table 3.3 Recommended values for models.

Pedestrian Type	No. of pedestrian			Reference Load (KN)	Crossing velocity (m/sec)	Reduction factor, γ	Stride length(m)
	Rural area	Suburban area	urban area				
Walking	2	4	8	280	1.7	0.8	1
Jogging	0	1	2	910	3	0.8	1.75

To study and compare the vertical dynamic results the natural and forcing frequency of 1.7Hz (worse case due to resonance) for both walking and jogging case is considered.

A moving dynamic load means that the amplitude of the load is fluctuating in time with a certain frequency and is moving over the bridge with a certain speed. In this case, the load is moving from node to node, as this is the most practical. A vertical point load, with amplitude of the reference load F_0 multiplied by the correct dynamic Load factor, is placed in the same direction on each node of the path.

The modal damping ratio is defined as 0.01 or 1 percent for all model case. As discussed in chapter three according to The UK National Annex recommends a vertical design acceleration limit:

$a_{limit}=1.0k_1k_2k_3k_4\ m/s^2$ and $0.5\ m/s^2 \leq a_{limit} \leq 2.0\ m/s^2$

k_1 =1.3 for Suburban crossings (site usage factor)

k_2 =1.0 for Primary route (redundancy factor)

k_3 = 1.0 for 4to8m bridge height (height of structure factor)

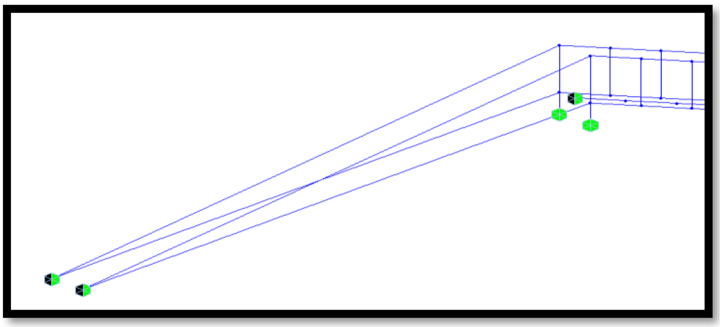
k_4 =1.0, exposure factor

$$\mathbf{a_{limit}=1.0k_1k_2k_3k_4\ m/s^2=1.0*1.3*1.0*1.0*1.0=1.3\ m/s^2}$$

3.6 Pedestrian Loading and Boundary Conditions

Pedestrian cable bridges are subject to many types of loading but this study limited to dead load for the self-weight of the structure and dynamic pedestrian loads. A vertical dynamic moving live load is modeled through a time-history analysis to determine the model response to pedestrian traversing the bridge. As discussed in previous chapter three the UK National Annex proposes moving along the most unfavorable line across the bridge in vertical direction for are both walking and jogging pedestrians are considered in this study. This corresponds much more to the reality. Several other attributes were added in an attempt to increase the reliability and adaptability of the model. This could be achieved through providing an appropriate finite element discretization of structural elements and taking care in the assignment of boundary conditions.

One of the major problems associated with a dynamic analysis is the modeling of boundary condition. These boundary conditions are the limitations on movement of the structure at places such as anchor locations. They will have mass and stiffness and can respond dynamically. The boundary conditions at the anchors and the ends of the deck consist of pin connections, and the base of the abutment consist of rigid connections and each of suspender cables are place every 1m and connected by rigid link in addition, the end moments between decking boards are released. Figure 3.5 shows the boundary conditions for bridge models.



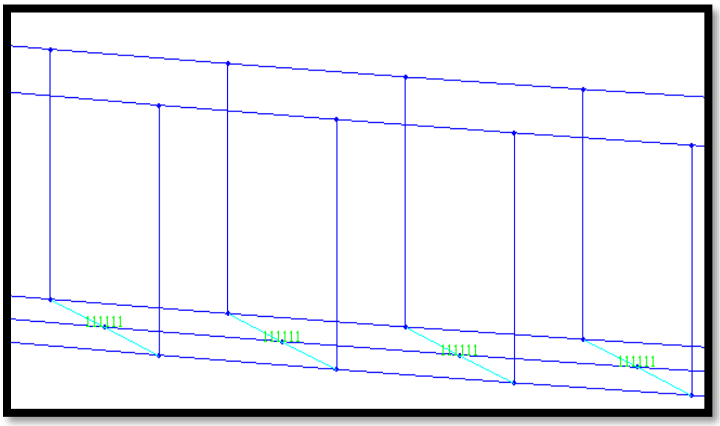


Figure 3.5 Model Bridges Boundary Conditions.

Several studies have been performed in order to quantify pedestrian walking forces. These studies have paid more attention to the vertical component of the dynamic force than the horizontal component. This is because, almost all documented problems with pedestrian-induces vibrations were associated with vertical forces and vibrations.

The load models presented in this study only deal with vertical vibrations. Therefore, for walking and jogging load case the time functions used to define the dynamic loading are based on British National Annex for Eurocode 1 of EN 1991-2.

The typical pacing frequency for Slow walking is in the region of 1,4 - 1,7 Hz and fast walking in the range of 2,2 - 2,4 Hz. This means that the total range of vertical forcing frequency is 1,4 - 2,4 Hz. Time functions are applied along the length of the bridge as dynamic nodal loads with a time gap to achieve worst case response due to resonance. Therefore, the natural frequency which equal to forcing frequency of 1.7 Hz is used for the model loading for this study.

The time function for the next step begins 0.58 seconds after the previous step began for all cases. The forces vs. time function for the vertical pedestrian force in all cases are showed in figure below (N represents number of pedestrians).

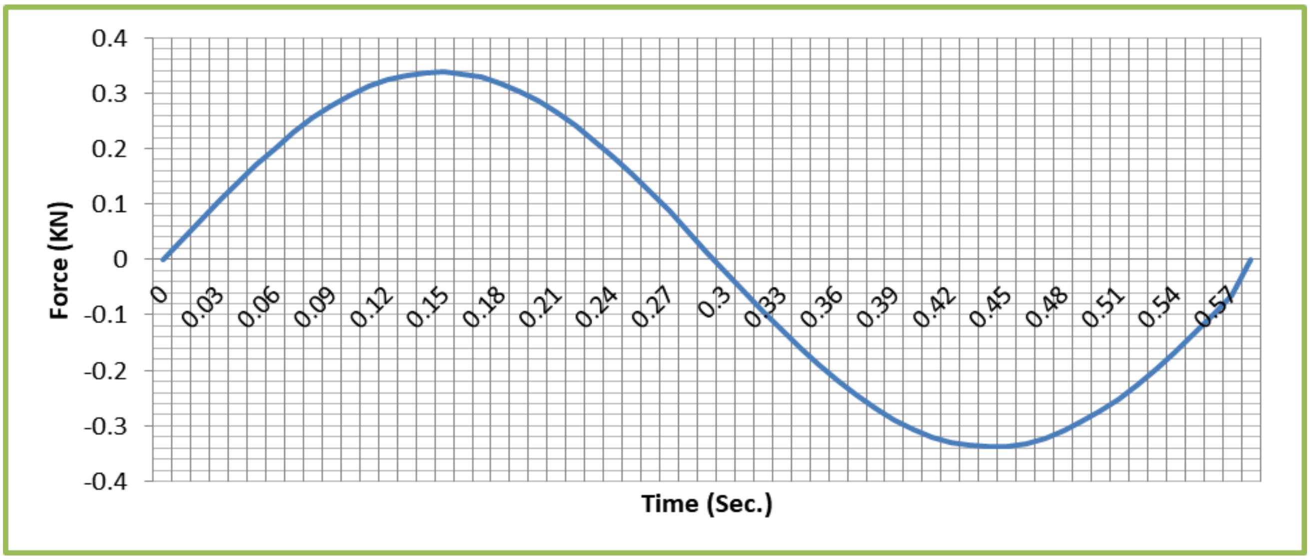


Figure 3.6 Vertical Pedestrian Walking Force Time Function for N=2.

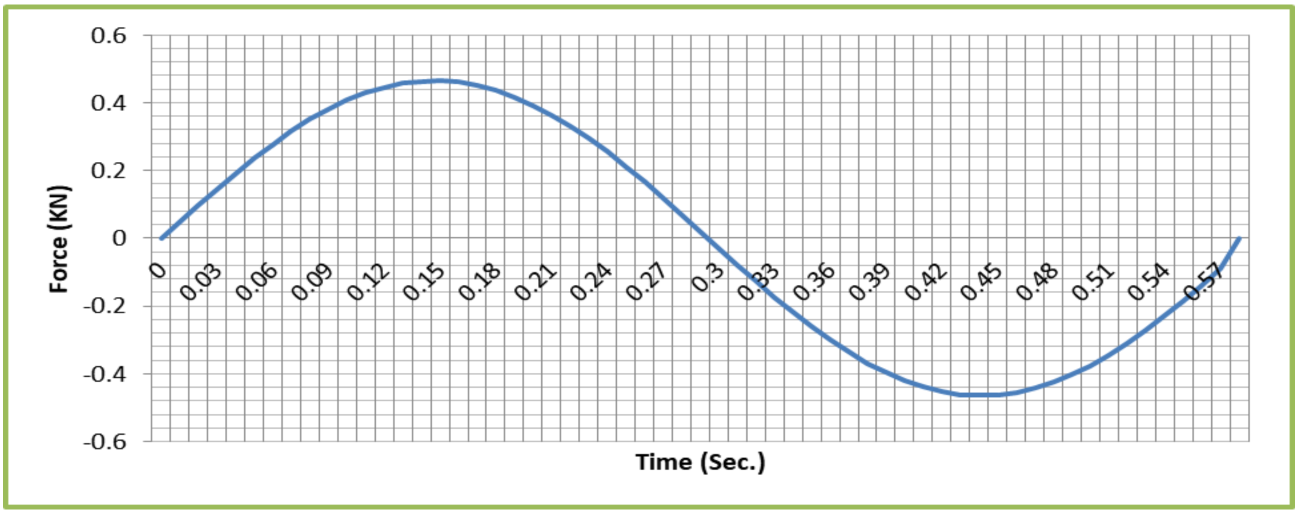


Figure 3.7 Vertical Pedestrian Walking Force Time Function for N=4.

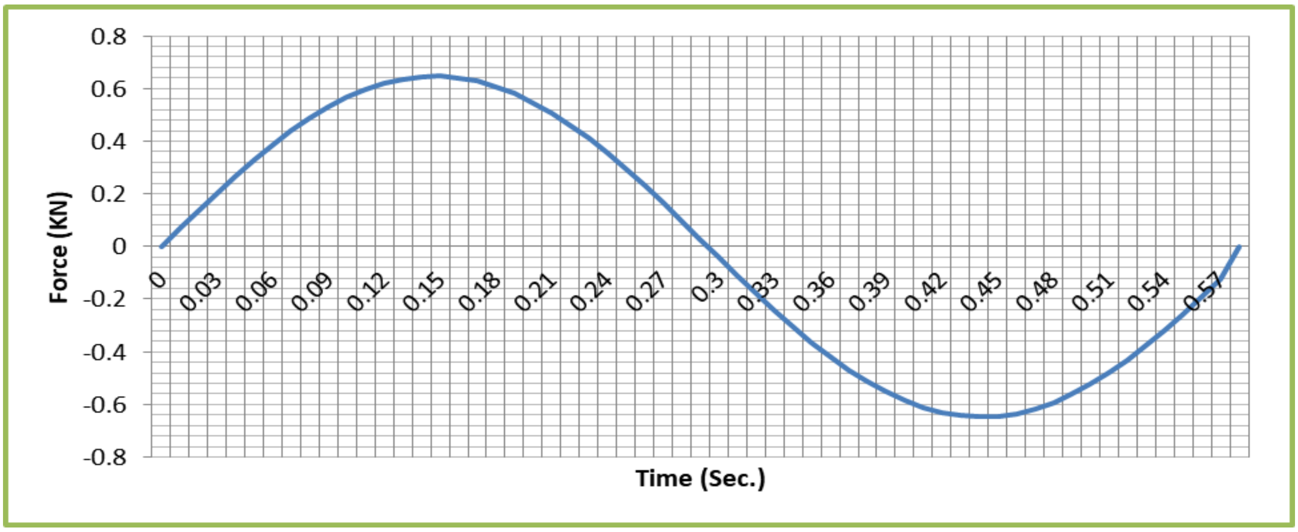


Figure 3.8 Vertical Pedestrian Walking Force Time Function for N=8.

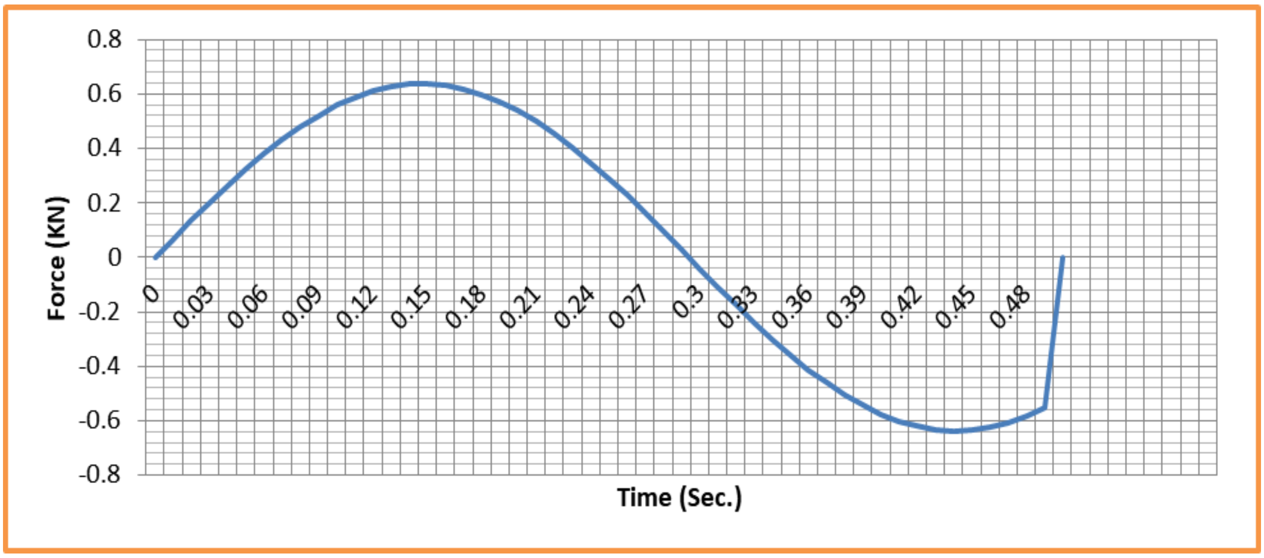


Figure 3.9 Vertical Pedestrian Jogging Force Time Function for N=1.

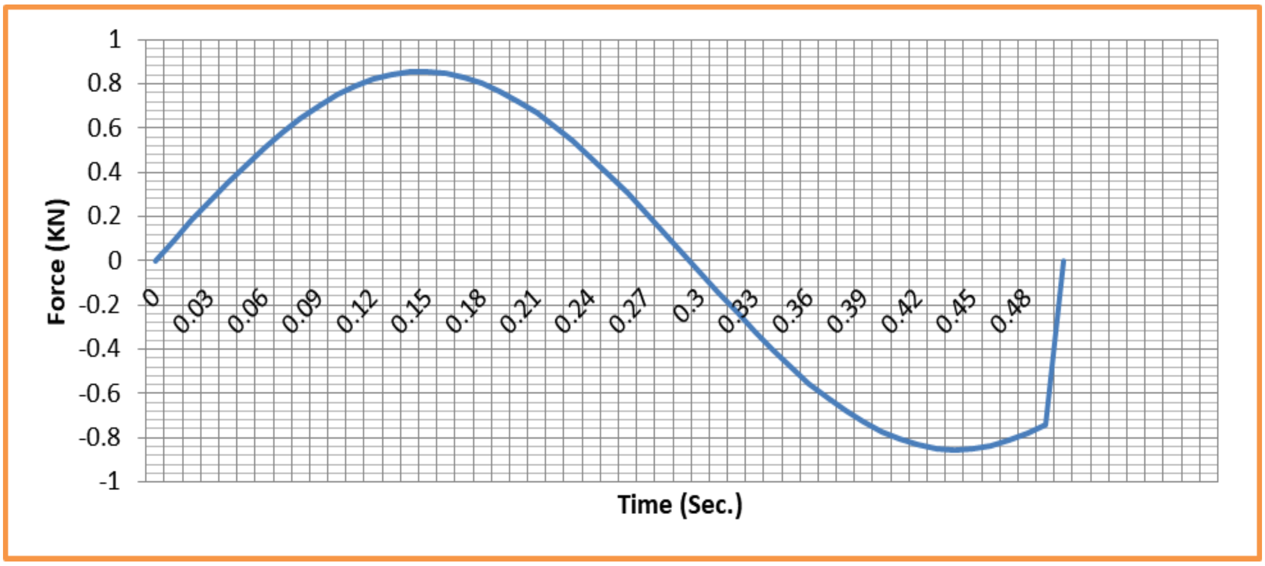


Figure 3.10 Vertical Pedestrian Jogging Force Time Function for N=2.

Having the geometric and material properties specified above and loading condition. The MIDAS/Civil software structural analysis software generates the bridge models that have a close feature with the actual bridge behavior. The load function was applied as nodal load at the center of the bridge deck. The finite element models of typical pedestrian cable suspended bridges are shown in figure 3.11 and 3.12.

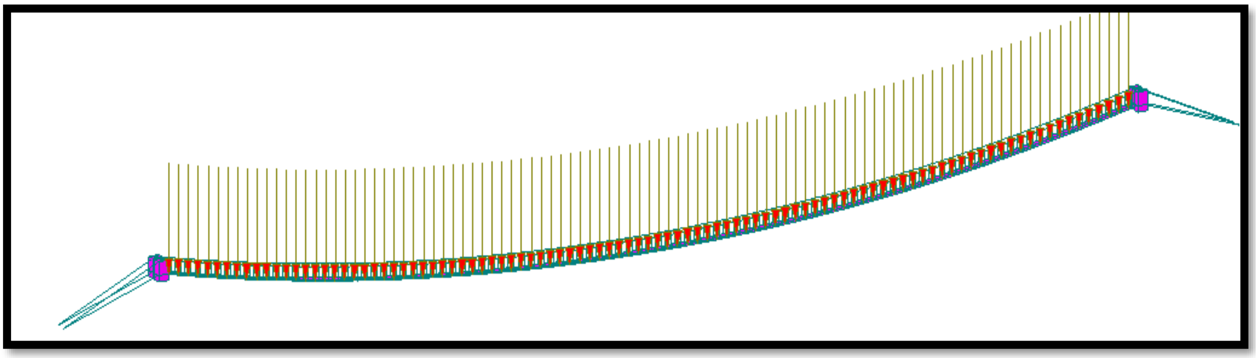


Figure 3.11 Typical pedestrian cable suspended bridge for Walking pedestrian.

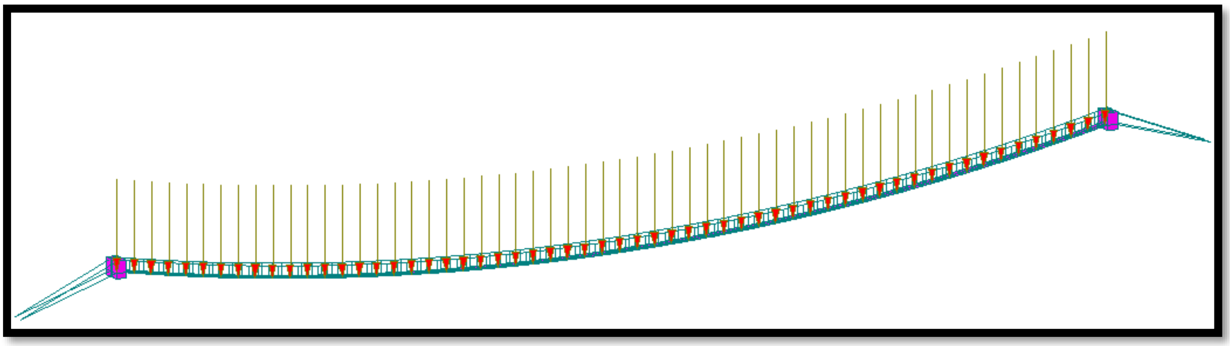


Figure 3.12 Typical pedestrian cable suspended bridge for Jogging pedestrian.

3.7 Assessment of the Natural Frequencies

Natural frequency, also known as Eigen frequency is the frequency at which a system tends to oscillate in the absence of any driving or damping force. If the oscillating system is driven by an external force at the frequency which the amplitude of its motion is greatest (close to a natural frequency of the system) this frequency is called resonant frequency.

A modal analysis was conducted for each model to determine the mode shapes and corresponding frequencies. The modal analysis was set as an Eigen vectors analysis. The modal analysis starts from the end of the nonlinear dead load analysis to evaluate the mode shapes of the structure under self-weight. The software defined dead load as nonlinear static to account for the nonlinearity of the cable elements. The case considers self-weight of the members, distributed loads from the nailers, and the lumped mass of the fence.

The frequency extraction procedure in MIDAS/Civil uses eigenvalue techniques to calculate the natural frequencies and the corresponding mode shapes of the structure. MIDAS/Civil includes initial stress and load stiffness effects due to preloads when geometric nonlinearity is accounted for in the base state.

3.8 Modal Time History Analysis and Parameter Studies

The approach of this study is to model the footbridge by MIDAS/Civil software and determine dynamic response parameter while one or two parameters are varied and all the others are kept constant. The load model proposed by UK National Annex has been used in this study.

Time history analysis is a step-by- step analysis of the dynamic response of a structure to a specified loading that may vary with time. It is used to determine the response of a structure under dynamic loading.

To determine the vertical accelerations of the footbridge under vertical pedestrian loading, the recommended modal damping ratio of 0.01 or 1 percent is used.

In this study, a parameter study has been performed. The objective is to investigate the influence of several parameters including cable diameter, cable sag and number of pedestrians on the vertical dynamic response of cable suspended footbridge. The purpose of increase the diameter of main cable is to stiffen the structure and to increase the modal frequencies and decrease vertical dynamic response like the accelerations of the structure resulting from a pedestrian walking and jogging across the bridge.

Vertical dynamic response was conducted through analyzing eighty numerical models which are presented in table 3.4.

Table 3.4 Bridge Models for sensitivity analysis.

Cable Footbridge Type	Span (m)	Cable Sag (%)	Load Case	Walkway and Handrail Cable Diameter (mm)	Number of Pedestrian
Suspended	42	5	Walking	26.00	N=2
					N=4
					N=8
				32.00	N=2
					N=4
					N=8
			Jogging	26.00	N=1
					N=2
				32.00	N=1
					N=2
		7	Walking	26.00	N=2
					N=4
					N=8
				32.00	N=2
					N=4
					N=8
			Jogging	26.00	N=1
					N=2
				32.00	N=1
					N=2

Suspended	60	5	Walking	26.00	N=2
					N=4
					N=8
				32.00	N=2
					N=4
					N=8
			Jogging	26.00	N=1
					N=2
				32.00	N=1
					N=2
		7	Walking	26.00	N=2
					N=4
					N=8
				32.00	N=2
					N=4
					N=8
			Jogging	26.00	N=1
					N=2
				32.00	N=1
					N=2
Suspended	80	5	Walking	26.00	N=2
					N=4
					N=8
				32.00	N=2
					N=4
					N=8
			Jogging	26.00	N=1
					N=2
				32.00	N=1
					N=2
		7	Walking	26.00	N=2
					N=4
					N=8
				32.00	N=2
					N=4
					N=8
			Jogging	26.00	N=1
					N=2
				32.00	N=1
					N=2

Suspended	100	5	Walking	26.00	N=2
					N=4
					N=8
				32.00	N=2
					N=4
					N=8
			Jogging	26.00	N=1
					N=2
				32.00	N=1
					N=2
		7	Walking	26.00	N=2
					N=4
					N=8
				32.00	N=2
					N=4
					N=8
			Jogging	26.00	N=1
					N=2
				32.00	N=1
					N=2

4. RESULTS AND DISCUSSIONS

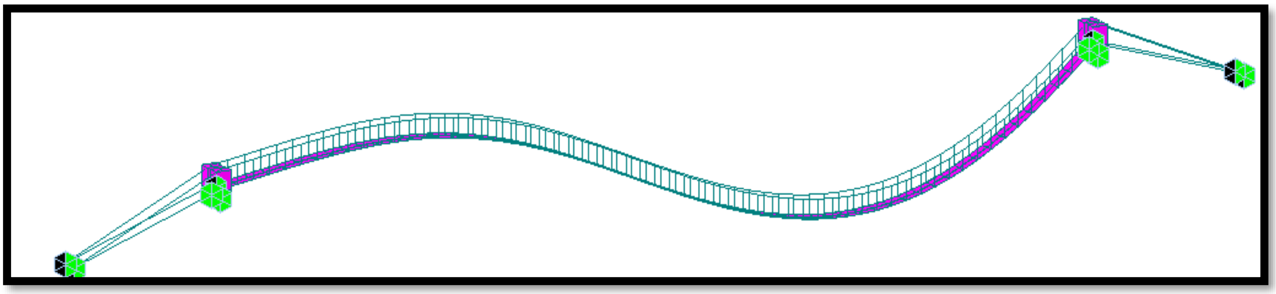
4.1 Introduction

MIDAS/Civil finite element software used to determine the dynamic vertical response of cable suspended footbridges, and it includes studying a total of eighty models. In this study a suitable model for modal time history analysis was developed to assess effect of human-induced vibrations on pedestrian cable bridges. The assessment was done by determining the mode shapes, natural frequency and vertical acceleration for different cable size, cable sag and number of pedestrians of selected bridge. In order to assess the contribution of higher modes in human induced vibration, modal analysis was used as analysis method for the dynamic problem. The number of modes in the analysis depends on the number of discretization points. The results are compared to the limitation with the standard which discussed in chapter three and check whether the modal frequencies and vertical Acceleration fall within the recommended range.

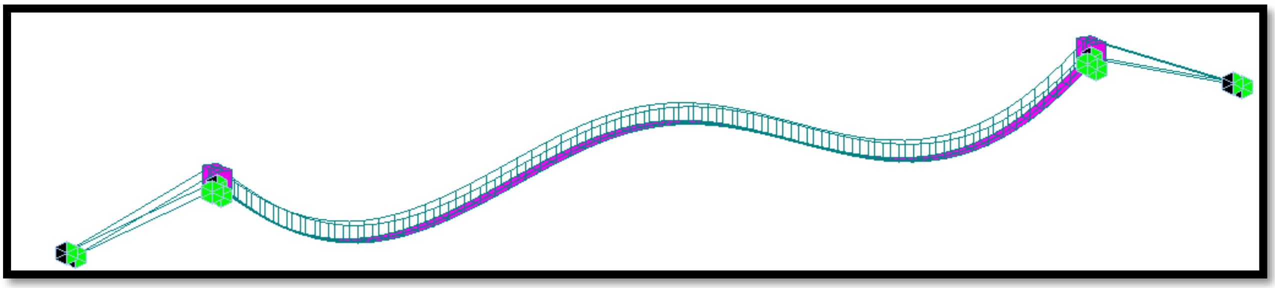
The basic modal properties including vertical natural frequency for the subject bridges are presented in this study. It is noted that the fundamental mode for all cases are vertical mode because the effect of human induced vibration is highly observed in vertical direction. The time history analysis results discussion over the case study bridges are briefly presented in these sections.

4.2 Modal shapes and Natural Frequency

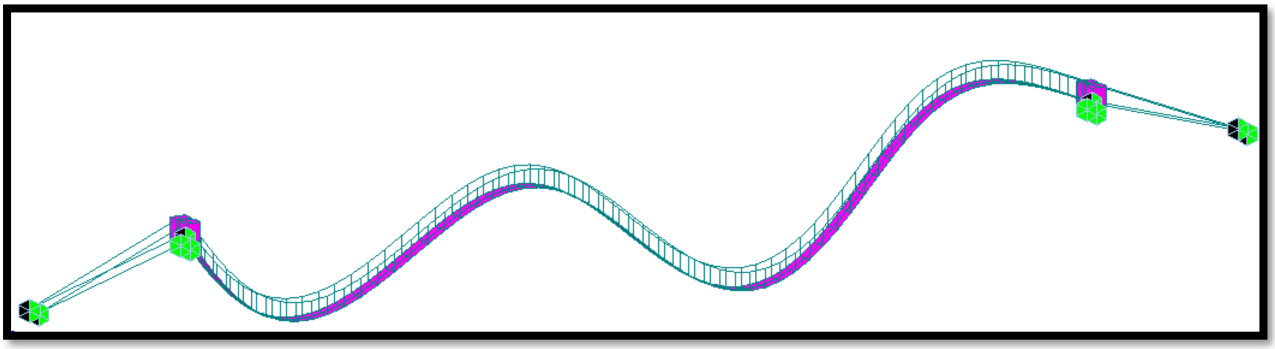
Modal analyses have been conducted to evaluate modal shape and Natural Frequency. The results are presented in Table 5.1 through Table 5.4. And the first three vertical mode shapes are presented since the vertical vibration is the main concern of this study. Three representative vertical modal shapes also showed in the figure 4.1.



A. Typical first vertical mode shape.



B. Typical second vertical mode shape.



C. Typical third vertical mode shape.

Figure 4.1 Pedestrian cable suspended bridge typical vertical mode shape.

Table 4.1 Vertical Natural Frequencies for walking and jogging with 5 Percent Cable Sag and 26mm cable diameter.

Span (m)	VF1	VF2	VF3
42	0.74	1.06	1.50
60	0.65	0.93	1.32
80	0.55	0.80	1.12
100	0.50	0.71	1.00

Table 4.2 Vertical Natural Frequencies for walking and jogging with 5 Percent Cable Sag and 32mm cable diameter**.**

Span (m)	VF1	VF2	VF3
42	0.69	0.99	1.40
60	0.60	0.87	1.23
80	0.52	0.74	1.05
100	0.46	0.66	0.94

Table 4.3 Vertical Natural Frequencies for walking and jogging with 7 Percent Cable Sag and 26mm cable diameter.

Span (m)	VF1	VF2	VF3
42	0.6	0.88	1.24
60	0.41	0.59	0.83
80	0.40	0.61	0.86
100	0.38	0.55	0.77

Table 4.4 Vertical Natural Frequencies for walking and jogging with 7 Percent Cable Sag and 32mm cable diameter.

Span (m)	VF1	VF2	VF3
42	0.56	0.82	1.15
60	0.38	0.56	0.79
80	0.37	0.57	0.81
100	0.35	0.51	0.72

The first three vertical natural frequencies decreased as the span length of footbridge increased for all case. However, this percentage of change varies as the span length increases.

For 5 percent cable sag and 26mm main cable diameter the first vertical modal frequencies decrease by 13.80 Percent when the span length increased from 42 m to 60 m but it decreased by 18.18 percent when the span length increased from 60 m to 80 m. On the other hand, the frequency decreased by 10 percent for span length from 80m to 100m. As observed from figure 4.2 and 4.3, for 7 percent cable sag and 26mm main cable diameter the modal frequencies decrease by different percent as the span length increase.

For 5 percent cable sag and 32mm main cable diameter the first vertical modal frequencies decrease by 13.70 Percent when the span length increased from 42 m to 60 m but it decreased by 17.56 percent when the span length increased from 60 m to 80 m. On the other hand, the frequency decreased by 12.12 percent for span length from 80m to 100m. For 7 percent cable sag and 32mm main cable diameter the modal frequencies decrease by different percent as the span length increase.

Generally, for 5 percent cable sag the span length has a greater effect on the modal frequencies for span lengths between 60m and 80m. Demonstrating that the span length has a greater effect on the modal frequencies for shorter span lengths. But the modal vertical frequencies do not depend on the number of pedestrians.

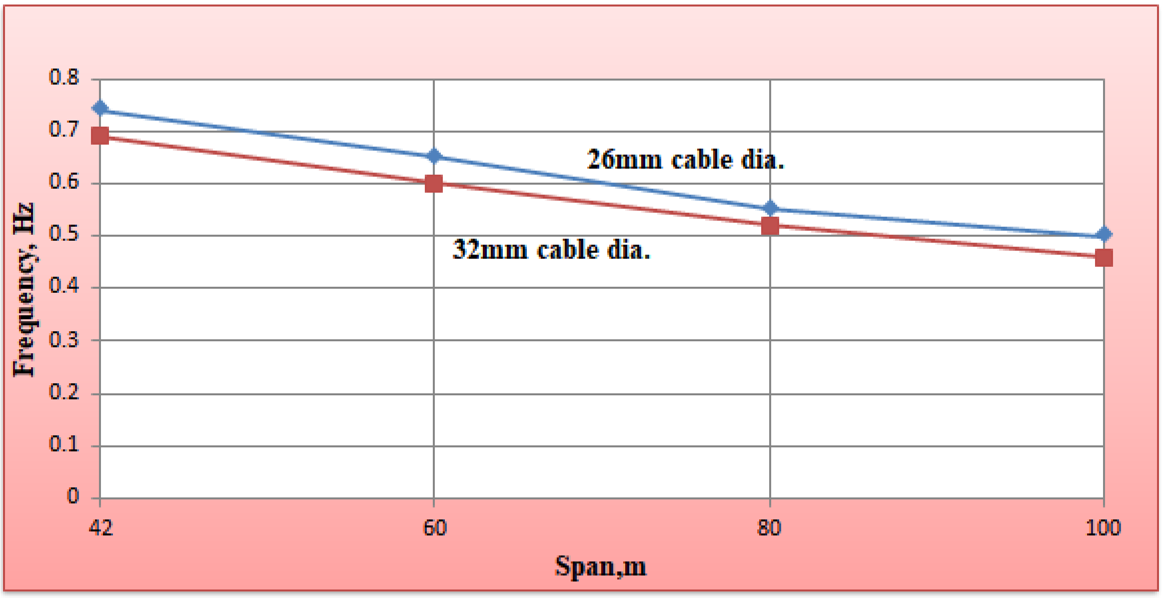


Figure 4.2 Vertical Modal Frequencies For 5 percent cable sag.

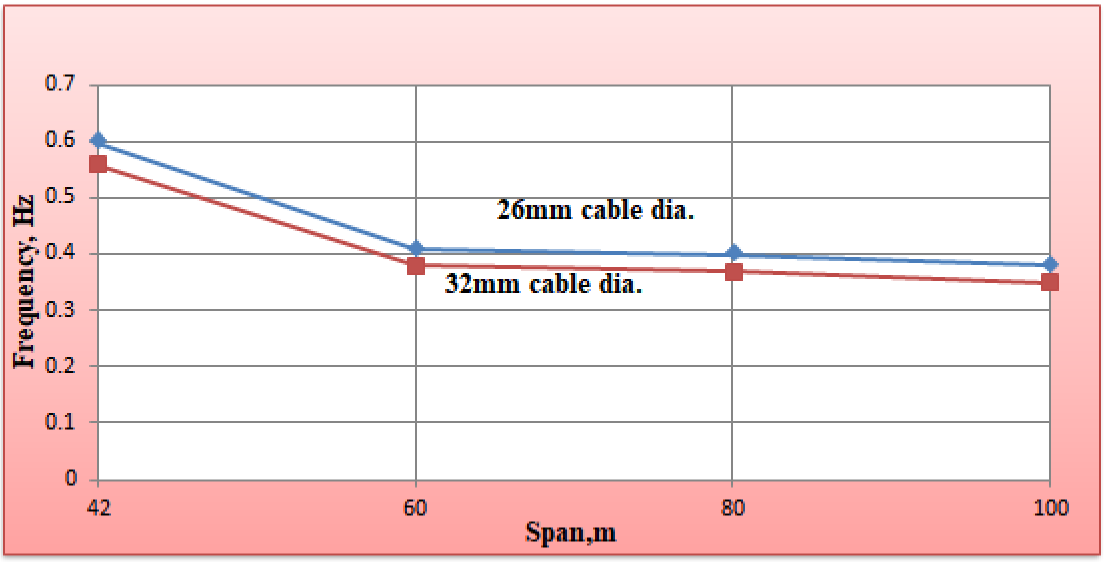


Figure 4.3 Vertical Modal Frequencies For 7 percent cable sag.

The limited values for natural frequency for selected footbridge according to the three codes are as follow;

- Minimum vertical natural frequency, Eurocode = 5Hz
- Minimum vertical natural frequency, AAHTO = 3Hz
- Minimum vertical natural frequency, The UK National Annex = 8Hz

For 7 percent cable sag with 26mm and 32mm cable diameter bridge the vertical natural frequencies below the UK National Annex and Eurocode minimum vertical natural frequency comfort limit. On other hand, for 5 percent cable sag with 26mm and 32mm cable diameter the third natural frequency almost equal to forcing natural frequency. So, it is expected that the vertical acceleration has been larger for this case.

As observed in figure 4.4, for all modal analysis the first three vertical natural frequencies are below the code specification. So, the verification of the comfort criteria should be assessed.

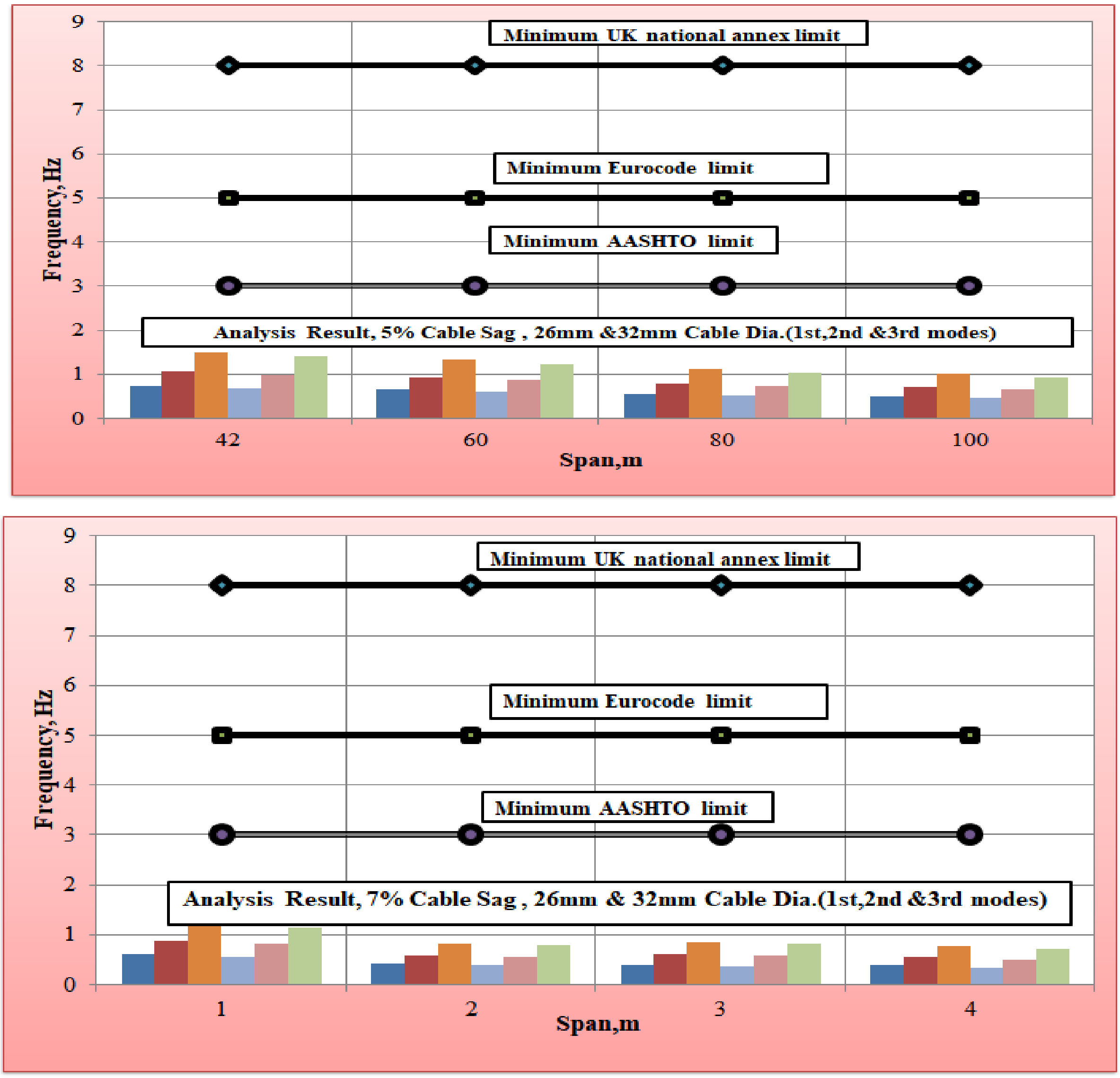


Figure 4.4 Vertical Modal Frequencies comparison with comfort limit.

4.3 Vertical Dynamic Responses

The vertical dynamic response included the displacements, velocities and accelerations**.** In this study, model analyses have been conducted and compared with comfort limit to determine vertical acceleration of selected pedestrian cable suspended bridge by varying different design parameters including span length, walkway and handrail cable diameter and cable sag and number of pedestrians.

In this section, only the result of vertical accelerations has been analyzed and discussed because it is the most critical dynamic parameter for cable footbridge.

4.3.1 Pedestrian cable suspended bridge located at rural area

The vertical acceleration of pedestrian cable suspended bridges are depend on the number of pedestrians who cross the bridge. According to UK National Annex guide line (code) the number of pedestrians directly related to bridge location.

When the bridge located at rural area the code specifies two number of pedestrians for walking case and zero number of pedestrians for jogging case. Vertical accelerations due to human induced loads are not experienced in jogging. Because, there is no pedestrian in this case.

For pedestrian walking case the vertical acceleration tends to decrease as span length increased. So, span lengths of 42m have the greatest vertical acceleration for all case. 42m span length with 5 percent cable sag has a higher vertical mode with third vertical modal frequency of 1.5 Hz which is almost frequency of walking person. So, it is expected that the vertical accelerations are larger because the forcing frequency match modal frequency of structure causing resonance.

For all span length and load case but similar cable diameters and 7 percent cable sag the bridge experienced smaller vertical acceleration than that of 5 percent cable sag. The bridge becomes stiffer when the walkway and handrail cable diameter increase. So, due to that vertical acceleration of 26mm cable diameter footbridge greater than that of 32mm cable diameter footbridge. The result of maximum vertical acceleration presented in table 4.5.

Table 4.5 Maximum Acceleration for cable suspended bridge located at rural area.

N=2, Suspended Bridge @ Rural area

	Maximum Vertical Acceleration for Walking (m/s^2)			
	5% Cable Sag		7% Cable Sag	
Span (m)	26mm cable Dia.	32mm cable Dia.	26mm cable Dia.	32mm cable Dia.
42	5.040	3.780	3.340	0.700
60	4.830	2.330	0.640	0.410
80	2.720	2.790	0.510	0.310
100	2.720	2.310	0.340	0.220

Where N=Number of pedestrians

Some shortcomings have been highlighted in existing codes of practice in relation to the dynamic response of footbridges regarding to the vertical displacement and vertical velocity limitation but for vertical acceleration the maximum limited value according to UK National Annex and Eurocode are 0.70 m/s^2 and 1.30 m/s^2 respectively.

As discussed, for most case the vertical acceleration decreases as the bridge span length increase. For 42m span bridge, the maximum vertical acceleration experienced by walking load case with 5 percent cable sag and 26mm cable diameter is 4 times greater than the comfort limitation specified by UK National Annex and Eurocode.

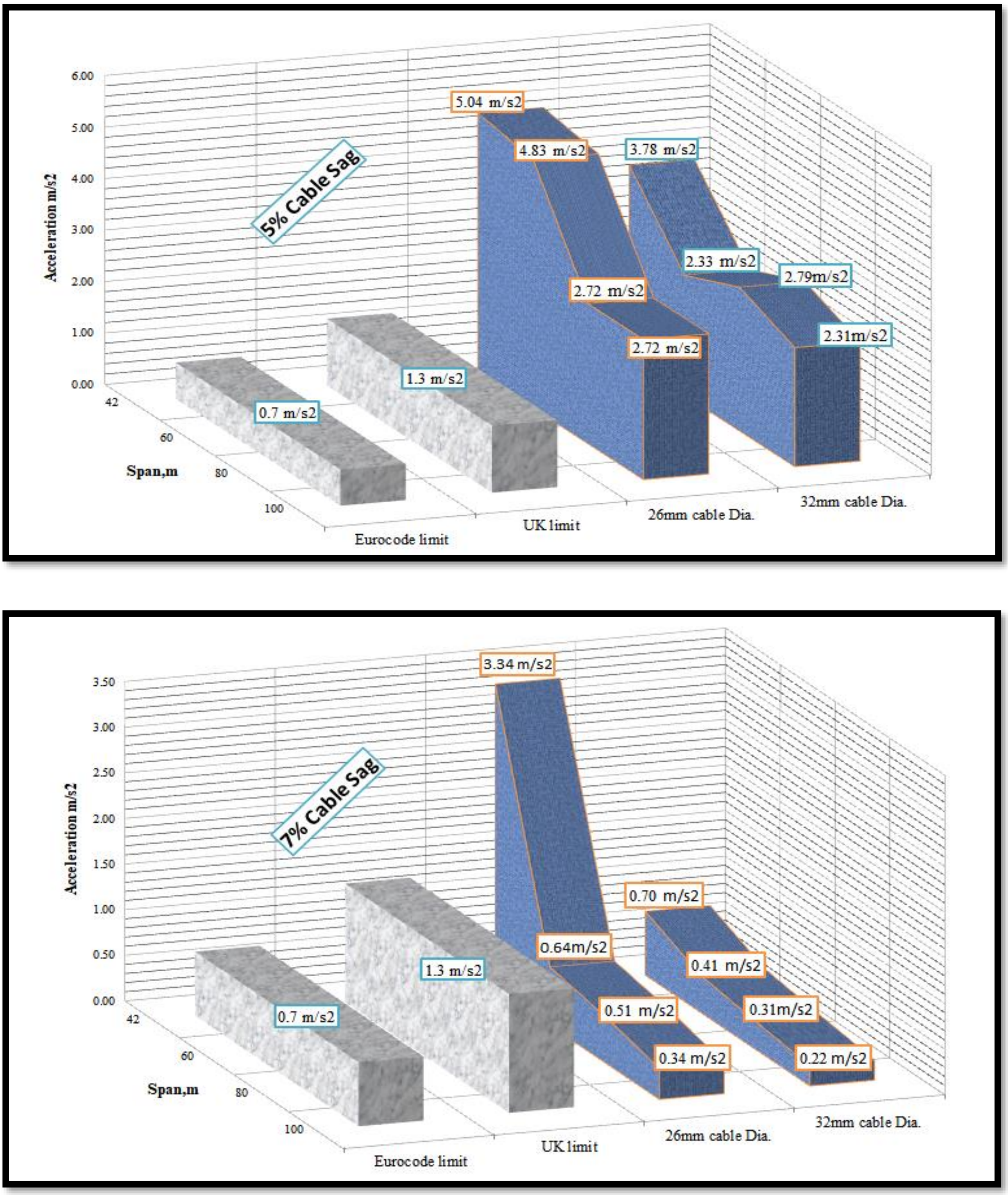


Figure 4.5 Vertical Acceleration comparison for walking pedestrians N=2.

Except for 42m span, for all other span the maximum vertical acceleration experienced by walking load cases with 7 percent cable sags are below the code specified comfort limit. But for the 42m span bridge with 7 percent cable sag and 26mm cable diameter the maximum vertical acceleration expected by jogging loads are 3 to 5 times greater than the code specified comfort limit.

Generally, from sixteen total modal analyses of pedestrian cable suspended bridges which located at rural area only seven of them satisfied the acceleration limit. So, most of them need remedial measure to keep the vertical acceleration with in the limit range.

4.3.2 Pedestrian cable suspended bridge located at suburban area

According to UK National Annex guide line (code) when the bridge located at suburban area the code specifies four number of pedestrians for walking case and one number of pedestrians for jogging case.

Table 4.6 Maximum Acceleration for cable suspended bridge located at suburban area.

N=4 **Suspended Bridge @ Suburban area** **N=1**

	Maximum Vertical Acceleration for Walking (m/s^2)			
	5% Cable Sag		7% Cable Sag	
Span (m)	26mm cable Dia.	32mm cable Dia.	26mm cable Dia.	32mm cable Dia.
42	7.010	5.140	4.600	1.020
60	6.520	3.123	0.908	0.571
80	3.900	3.580	0.730	0.420
100	4.090	3.130	0.553	0.320

	Maximum Vertical Acceleration for Jogging (m/s^2)			
	5% Cable Sag		7% Cable Sag	
Span (m)	26mm cable Dia.	32mm cable Dia.	26mm cable Dia.	32mm cable Dia.
42	10.580	4.408	10.110	1.450
60	7.530	7.880	1.277	0.784
80	3.910	7.450	1.043	0.602
100	4.280	5.800	0.755	0.463

Where N=Number of pedestrians

The vertical accelerations of subjected footbridges are dependent on the span length of the structure. For all cases the value of vertical acceleration is higher for shorter spans as observed from table 4.6. For walking load case and 5 percent cable sag with 26mm and 32mm cable diameter the maximum vertical acceleration decreases by 7 to 39 percent when the span length is increased from 42 m to 60 m. but for jogging load case with similar cable sag and cable diameter the maximum vertical acceleration decreases by 28 to 44 percent when the span length increased from 42 m to 60 m. It demonstrated that the effect of vertical accelerations has greater on pedestrian jogging case. The model with a 42 m span length and 26mm cable diameter has high vertical accelerations because of resonance.

As observed in figure 4.6 and 4.7, for 42 m span bridges, the average vertical acceleration experienced by the walking pedestrian with 5 percent cable sag and 26mm cable diameter are

5 to 10 times greater than the comfort limit, and the average vertical acceleration experienced by the jogging pedestrians are 8 to 15 times great than Eurocode and UK national annex comfort limit specification.

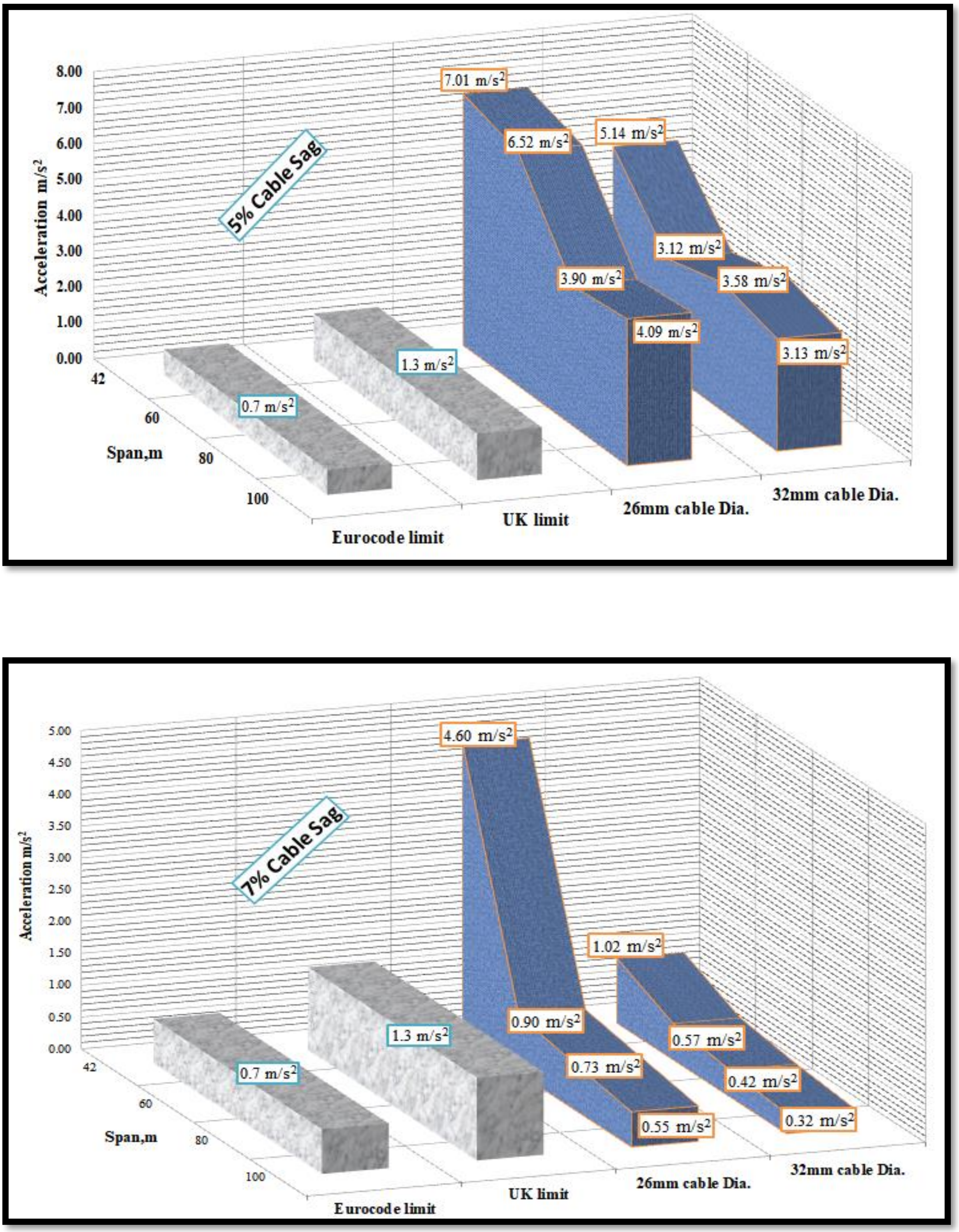
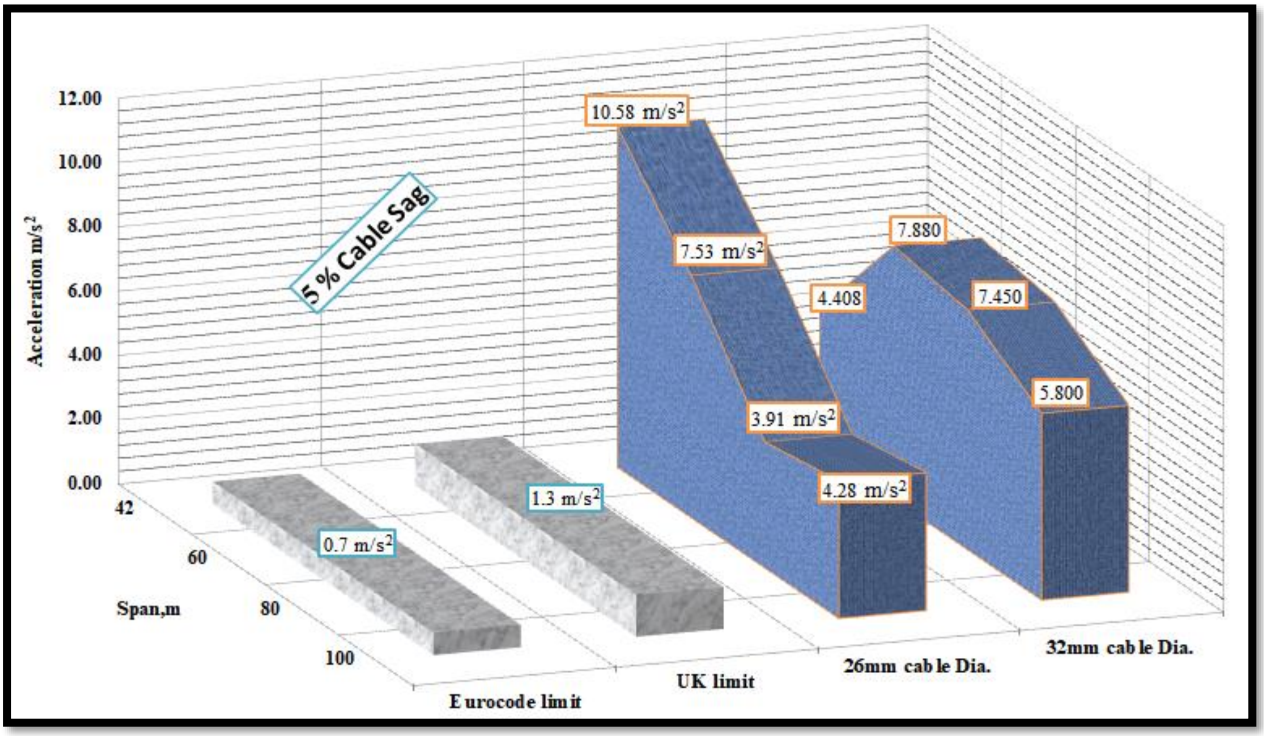


Figure 4.6 Vertical Acceleration comparison for walking pedestrians N=4.

The cable sag has great effect on the vertical accelerations. The vertical accelerations are typically lower for the 7 percent cable sag models as observed from table 4.6. The maximum vertical accelerations experienced by the pedestrian joggings are the greatest for 42 m span models with 5 percent sag because a higher vertical modal frequency for this model matches the jogging vertical frequency. The average vertical accelerations differ for 5 percent cable sag as compared to 7 percent cable sag by 68 percent for pedestrian walking with 26mm cable diameter. The average vertical accelerations differ by 83 percent for pedestrian walking with 32mm cable diameter. Similarly, for pedestrian jogging with 26mm and 32mm cable diameter the average vertical accelerations differ for 5 percent cable sag as compared to 7 percent cable sag by 50 percent and 87 percent respectively. The vertical accelerations are anticipated to depend on the cable sag because the sag is in the vertical direction.

Changing the walkway and handrail cable diameter from 26mm to 32mmm causes the vertical acceleration to decrease because the footbridge becomes stiffer when the main cable sizes increase. Table 4.6 presents the effect of cable size on the vertical acceleration result. For both walking and jogging pedestrians with 7 percent cable sag, 32mm cable diameter and span length of 60m, 80 and 100m the vertical accelerations are below the comfort limit. It demonstrated that cable size has a greater effect on the vertical acceleration for larger cable sag.

Generally, as observed in figure 4.6 and 4.7 the effect of cable size increase as the span length increase, and the longer bridge span experienced less vertical acceleration.



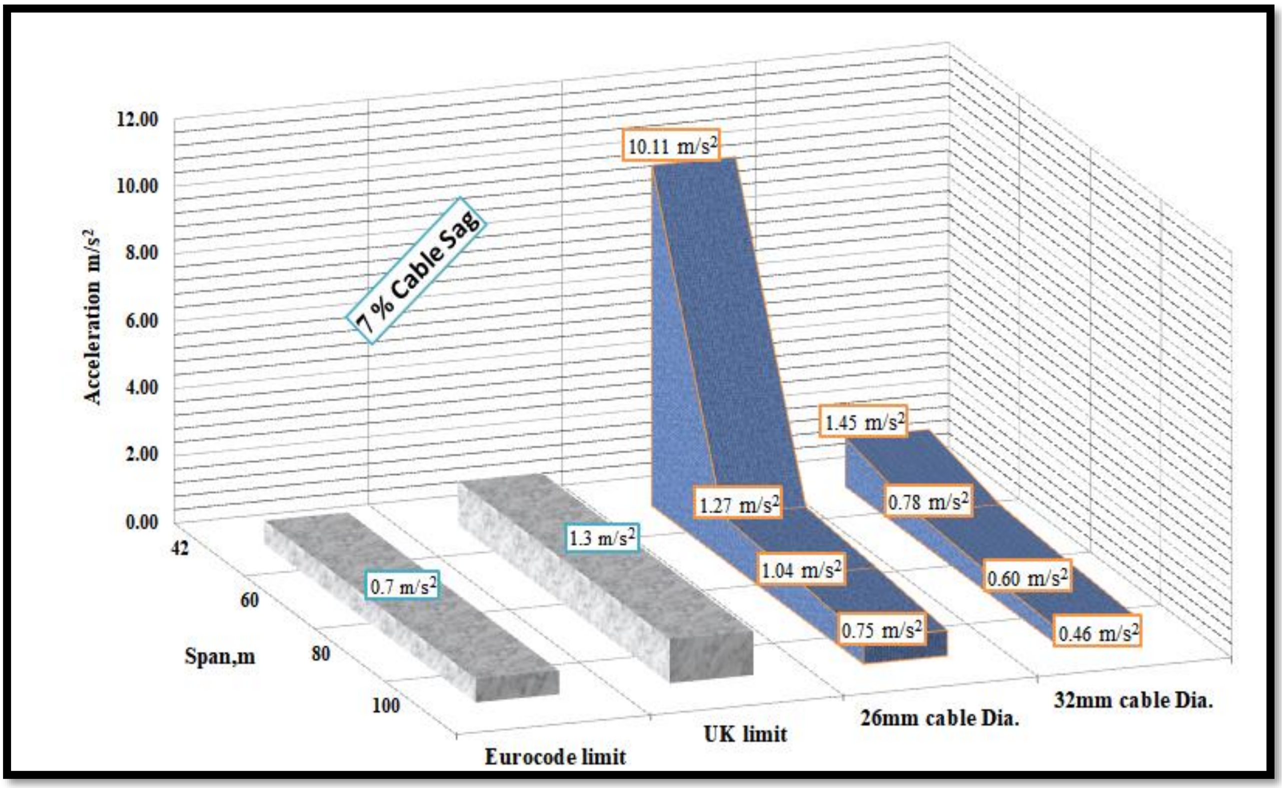


Figure 4.7 Vertical Acceleration comparison for jogging pedestrians N=1.

4.3.3 Pedestrian cable suspended bridge located at urban area

According to UK National Annex guide line (code) when the bridge located at urban area the code specify night number of pedestrians for walking case and two number of pedestrians for jogging case.

Table 4.7 Maximum Acceleration for cable suspended bridge located at urban area.

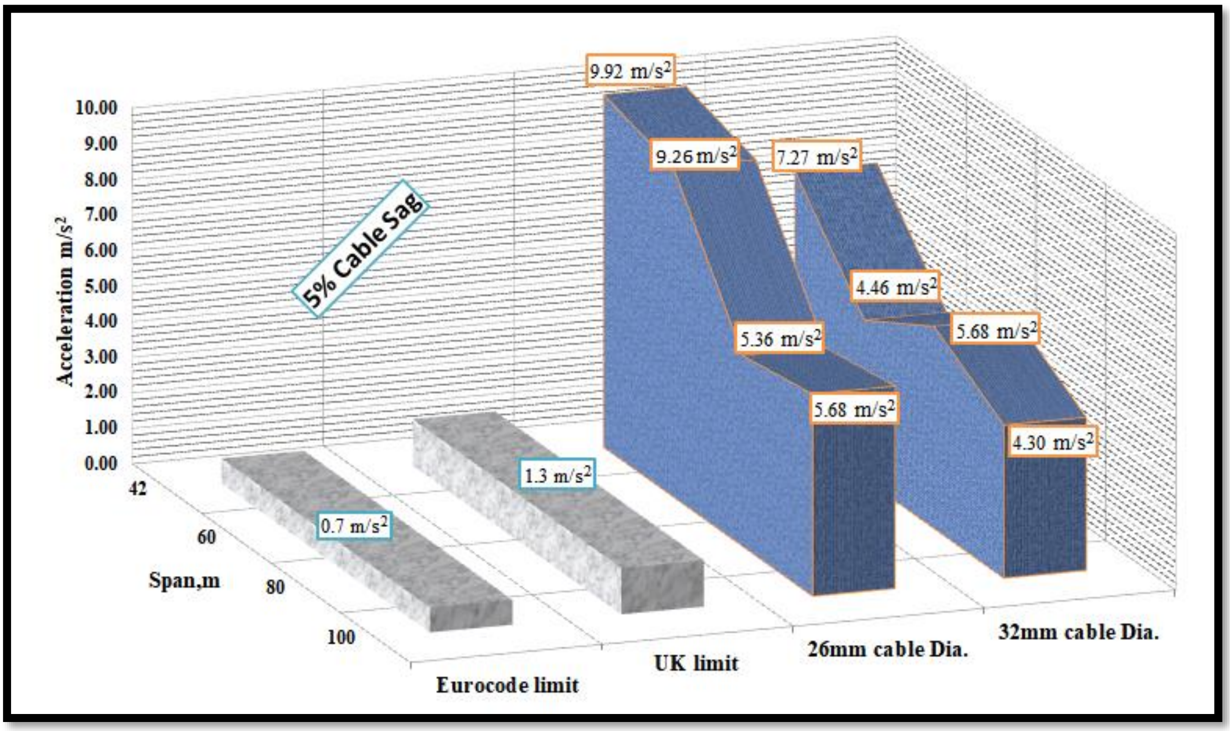
N=8 **Suspended Bridge @ Suburban area** **N=2**

	Maximum Vertical Acceleration for Walking (m/s^2)			
	5% Cable Sag		7% Cable Sag	
Span (m)	26mm cable Dia.	32mm cable Dia.	26mm cable Dia.	32mm cable Dia.
42	9.920	7.270	6.400	1.400
60	9.260	4.460	1.260	0.790
80	5.360	5.680	1.030	0.580
100	5.680	4.300	0.750	0.430

	Maximum Vertical Acceleration for Jogging (m/s^2)			
	5% Cable Sag		7% Cable Sag	
Span (m)	26mm cable Dia.	32mm cable Dia.	26mm cable Dia.	32mm cable Dia.
42	13.880	5.840	13.480	1.890
60	8.260	8.590	1.710	1.050
80	5.240	9.500	1.390	0.790
100	7.770	7.770	0.990	0.620

Where N=Number of pedestrian

The vertical accelerations of cable suspended footbridges are dependent on the span length of the structure. In all case the acceleration decrease as the span length increase. And 42m span ,5 percent cable sag and 26mm cable diameter footbridge model by jogging pedestrian experienced greater vertical acceleration.



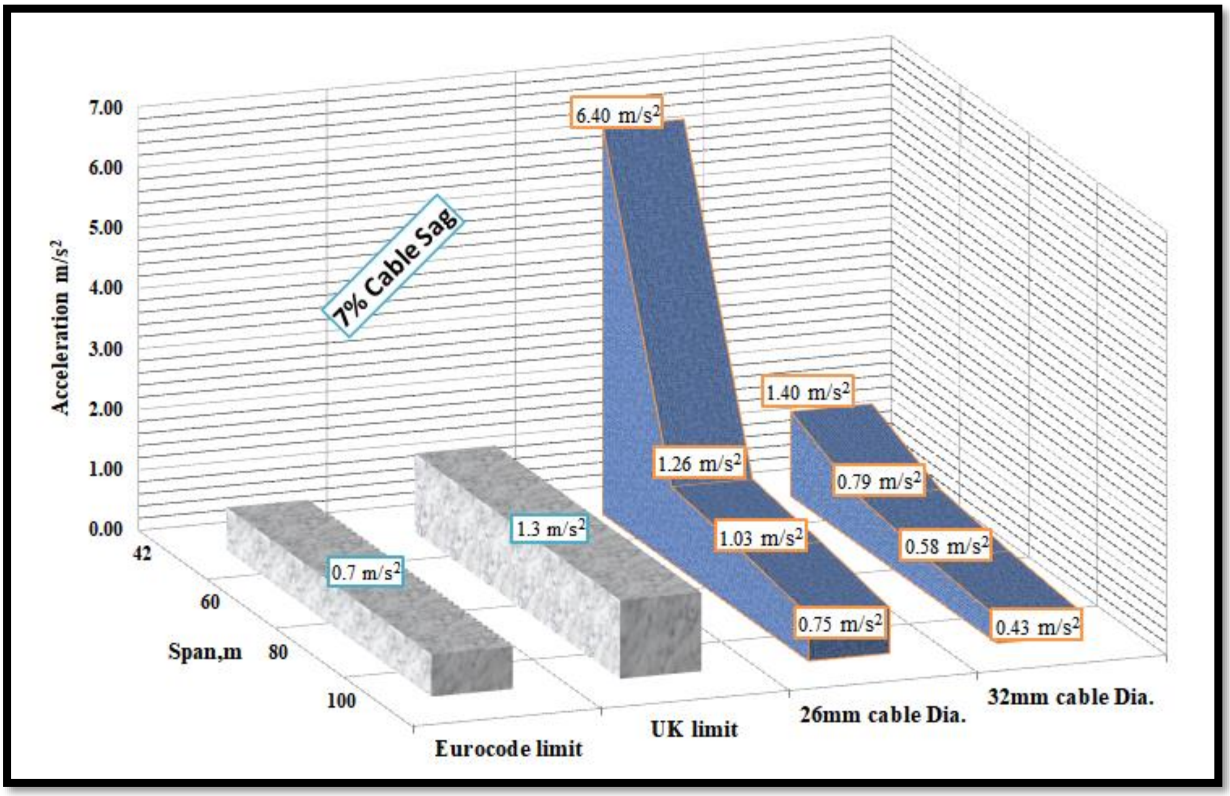


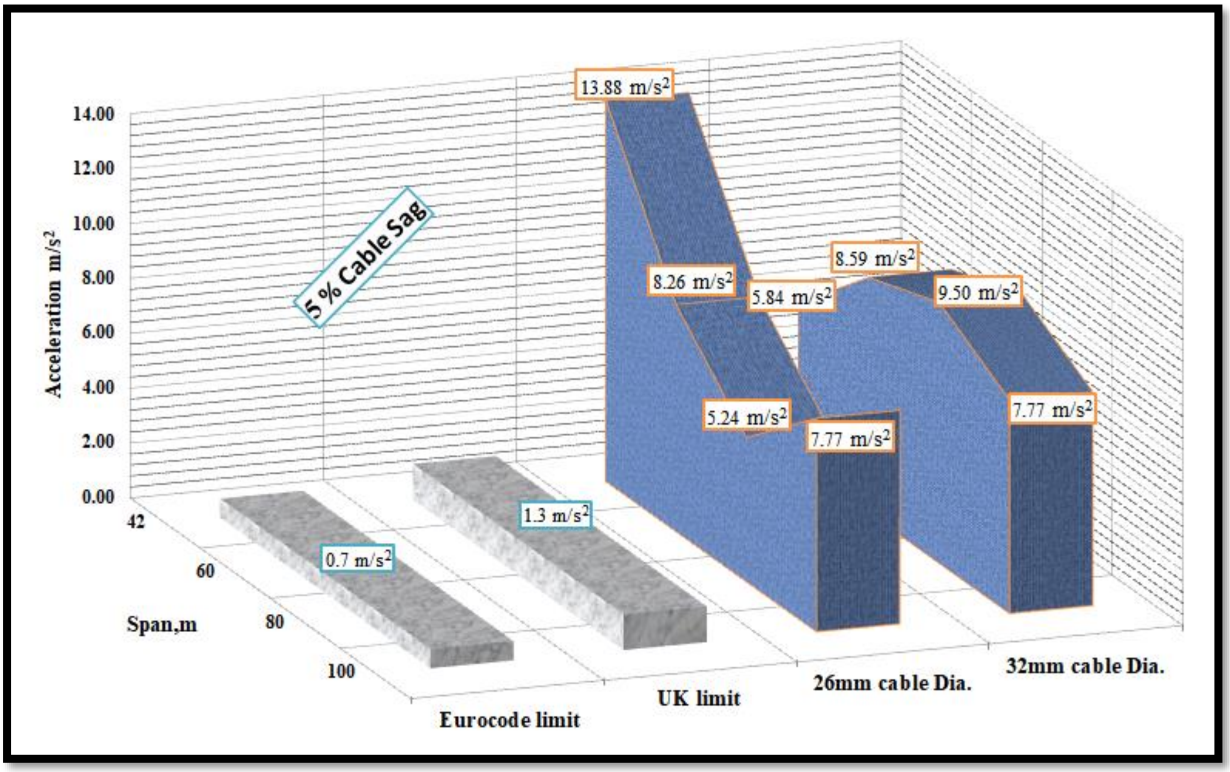
Figure 4.8 Vertical Acceleration comparison for walking pedestrians N=8.

For walking load case and 5 percent cable sag with 26mm and 32mm cable diameter the maximum vertical acceleration decreases by 7 to 38 percent when the span length increased from 42 m to 60 m. but it decreases by 20 to 57 percent when the span length increased from 60 m to 80 m. For 7 percent cable sag with similar cable diameter and pedestrian load the maximum vertical acceleration decreases by 80 to 43 percent when the span length increased from 42 m to 60 m but it decrease by 16 to 26 percent when the span length is increased from 60 m to 80 m.

As observed in figure 4.8 and 4.9, for 42 m span bridges, the average vertical acceleration experienced by the walking pedestrian with 5 percent cable sag and 26mm cable diameter are 7 to 14 times greater than the comfort limit, and the average vertical acceleration experienced by the jogging pedestrians are 10 to 20 times great than Eurocode and UK national annex comfort limit specification.

The cable sag has great effect on the vertical accelerations. The vertical accelerations are typically lower for the 7 percent cable sag models as observed from table 5.7. The maximum vertical accelerations experienced by the pedestrian joggings are the greatest for 42 m span models with 5 percent sag. The average vertical accelerations differ for 5 percent cable sag as compared to 7 percent cable sag by 68 percent for pedestrian walking with 26mm cable diameter. The average vertical accelerations differ by 85 percent for pedestrian walking with 32mm cable diameter. Similarly, for pedestrian jogging with 26mm and 32mm cable diameter the average vertical accelerations differ for 5 percent cable sag as compared to 7 percent cable sag by 51 percent and 86 percent respectively.

For all cases, the walkway and handrail cable diameter have great effect on vertical acceleration results. Table 4.7 presents the effect of cable size on the vertical acceleration result. For both walking and jogging pedestrians with 7 percent cable sag, 32mm cable diameter and span length of 80 and 100m the vertical accelerations are below the comfort limit. It demonstrated that cable size has a greater effect on the vertical acceleration for larger cable sag.



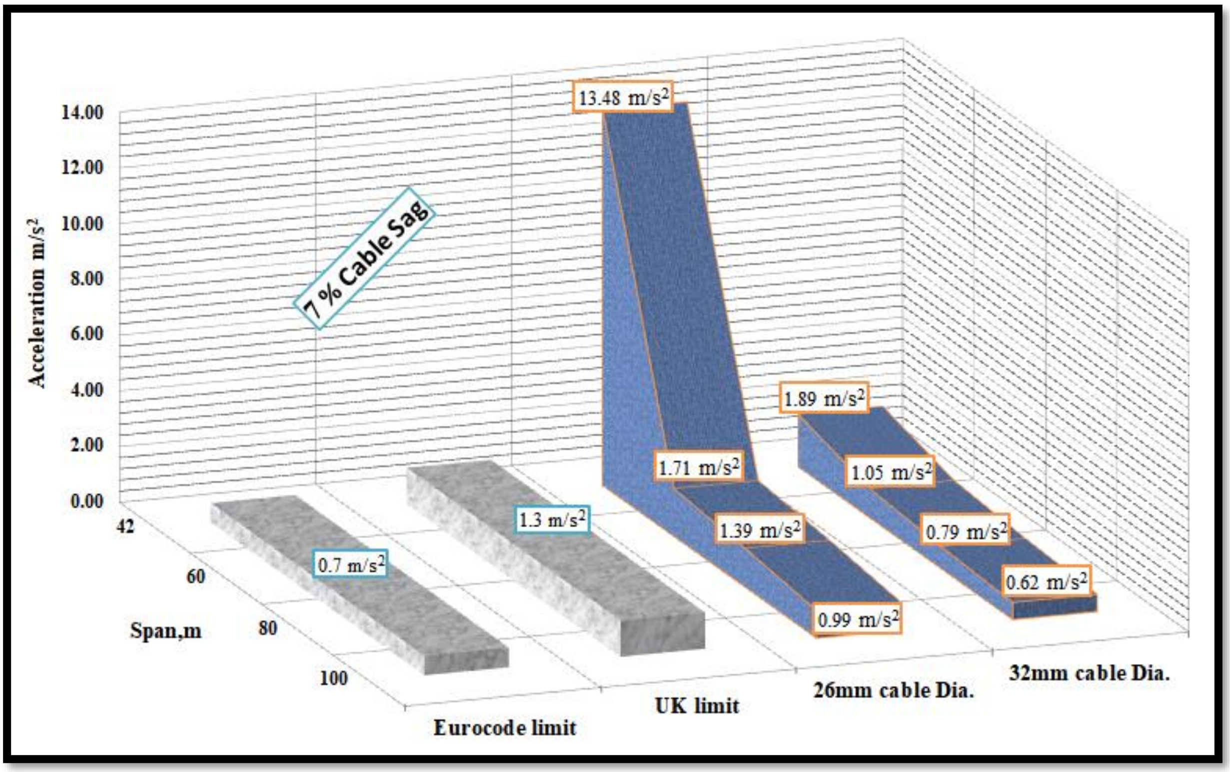


Figure 4.9 Vertical Acceleration comparison for jogging pedestrians N=2.

4.4 Summary

4.4.1 Effect of number of pedestrians on vertical acceleration

A total of eighty models were evaluated in vertical dynamic response study to determine the effect of span length, cable sag, cable diameter and number of pedestrians on cable suspended footbridge. The results were compared to determine if the modal frequencies and vertical acceleration fall within the human comfort limits. Most modal vertical frequencies and vertical acceleration do not meet the required criteria.

The modal vertical frequencies are not affected by loading function and number of pedestrian because modal frequencies depend on self-weight of structure not pedestrian loads. But vertical dynamic response especially vertical acceleration effected by loading functions and number of pedestrian.

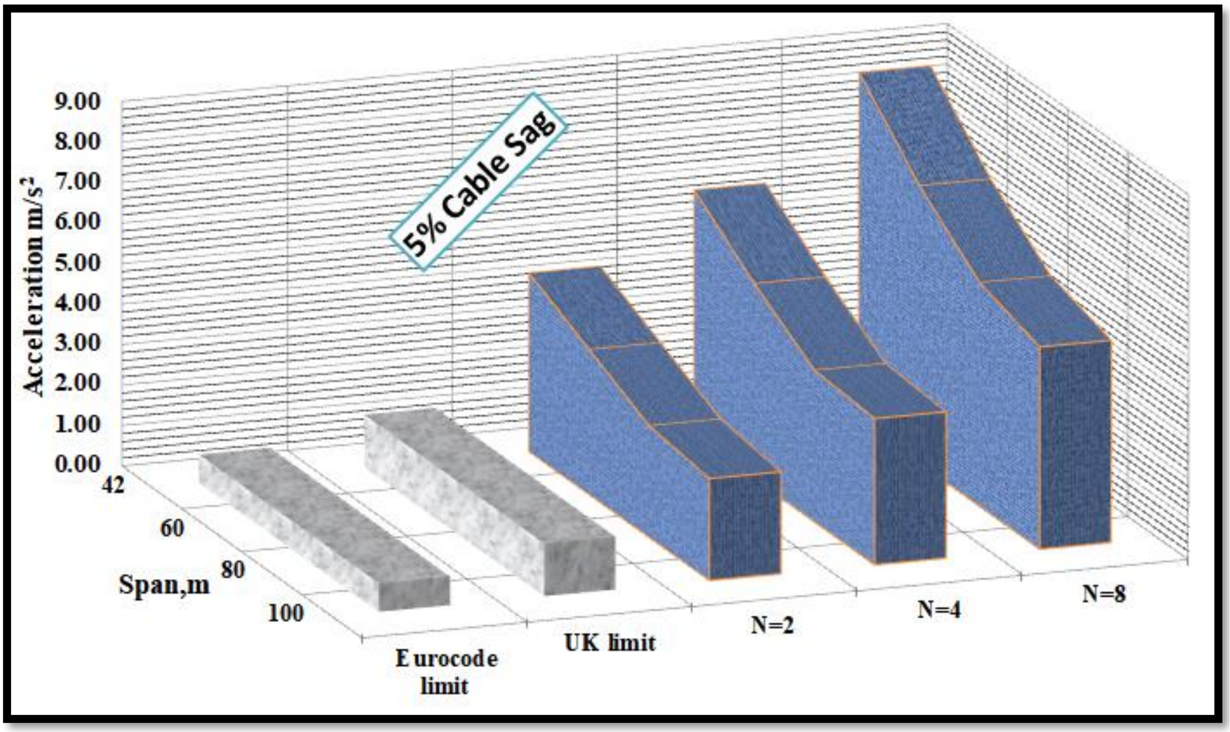
As observed from table 4.8 for both pedestrian walking and jogging load case the average maximum acceleration increase as the number of pedestrian increases.

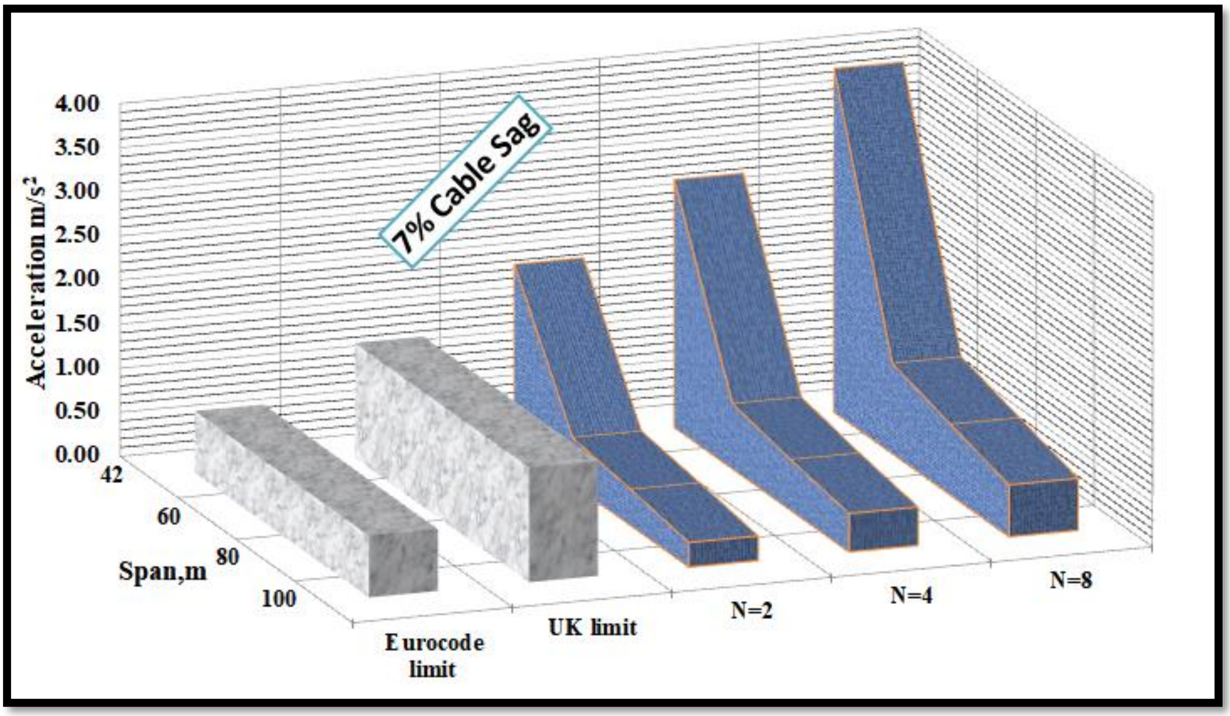
Except 42m span, in all other bridge span the vertical accelerations experienced by 2 and 4 number of walking persons are within the comfort limits when 7 percent cable sag is present. But, for 42m bridge span vertical acceleration is not within comfort limits.

Table 4.8 Number of Pedestrians versus Vertical acceleration.

	Average Maximum Vertical Acceleration for Walking (m/s^2)					
	No. of Pedestrians (5% Cable Sag)			**No. of Pedestrians (7% Cable Sag)**		
Span (m)	2.00	4.00	8.00	2.00	4.00	8.00
42	4.410	6.075	8.595	2.020	2.810	3.900
60	3.580	4.822	6.860	0.525	0.740	1.025
80	2.755	3.740	5.520	0.410	0.575	0.805
100	2.515	3.610	4.990	0.280	0.437	0.590

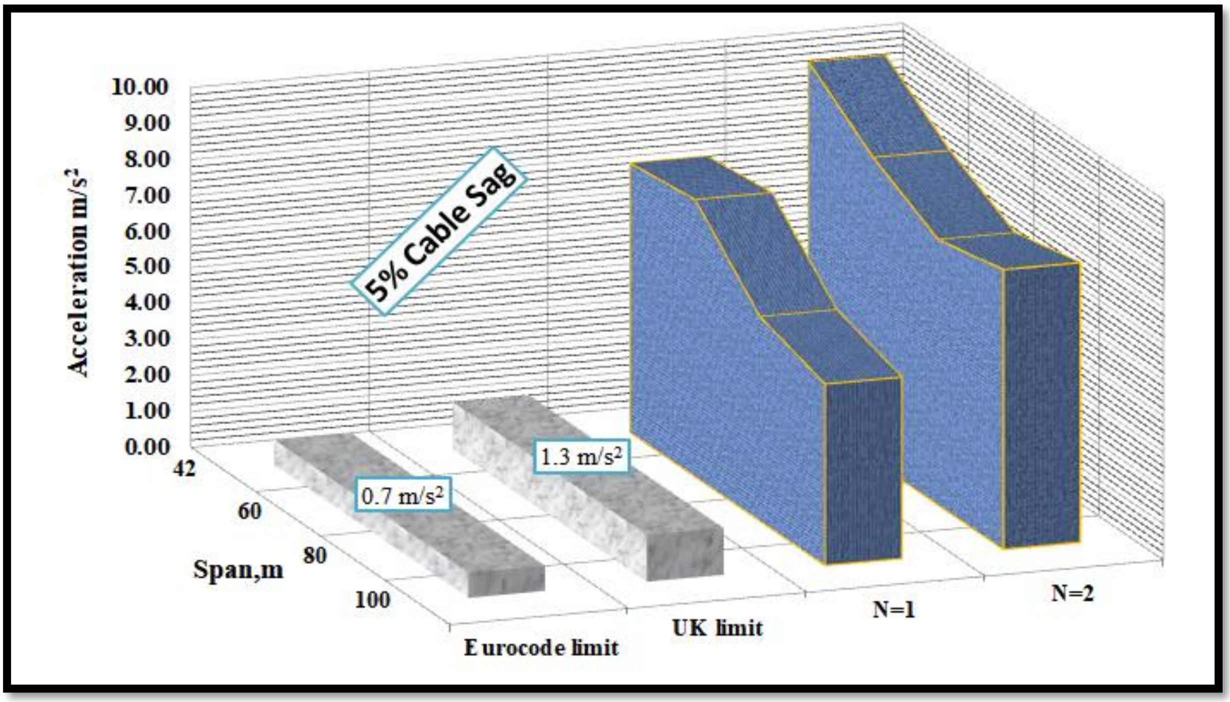
	Average Maximum Acceleration for Jogging (m/s^2)			
	No. of Pedestrians (5% Cable Sag)		**No. of Pedestrians (7% Cable Sag)**	
Span (m)	1.00	2.00	1.00	2.00
42	7.494	9.860	5.780	7.685
60	7.705	8.425	1.031	1.380
80	5.680	7.370	0.823	1.090
100	5.040	7.770	0.609	0.805

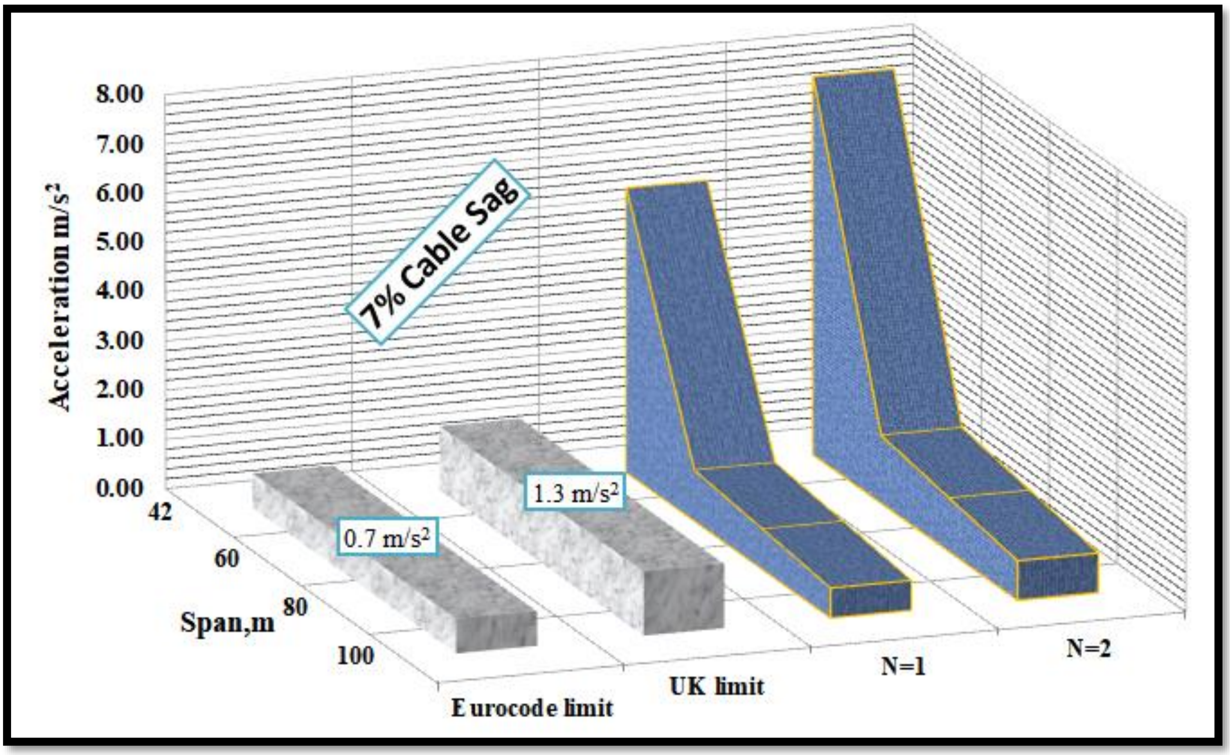




Where N=Number of pedestrians

Figure 4.10 Effect of number of pedestrian's for walking case.





Where N=Number of pedestrians

Figure 4.11 Effect of number of pedestrian's for jogging case.

4.4.2 Remedial Measure

New highly stressed materials, more slender structures, smaller cross-sectional areas and greater spans of suspended footbridges have led to decreasing structural stiffness, damping and mass. Decreasing stiffness leads to smaller natural frequencies resulting in more sensitivity to vibrations. Also, decreased mass and damping means smaller forces are required to excite the structure. So, suspended footbridges must be designed for dynamic as well as static loads and a dynamic analysis should be a part of the early design stages. Resonance can cause a magnification of the dynamic response which can lead to structural instability, overstressing or fatigue failure of materials. An effort should therefore be made to avoid resonance.

One possible solution to vibration problems due to pedestrian loading is to avoid natural frequencies which are in ranges coinciding with forcing frequencies. This can be achieved by increasing the stiffness of the footbridge thus moving all its natural frequencies out of the range that can be excited by pedestrians.

In this study, time history analysis has been carried out by changing the main cable diameter, cable sag and number of pedestrians.

As discussed in previous section of this chapter, when the cable diameter changed from 26mm to 32mm the structural stiffness also increased and the dynamic response significantly reduced, this effect clearly noticed in 7 percent cable sag for both walking and jogging load case.

Changing cable diameter has a greater effect on the dynamic response for higher cable sag and shorter span lengths. Therefore, increase stiffness by changing main cable diameter highly reduce vibration problem on suspended footbridge.

Another measure against vibration problems of footbridges is to increase the overall damping of the structure. There are several energy absorption mechanisms that contribute to the damping of a structure. Increasing the damping by modifying the structure, connections, supports and non-structural elements may be considered. To increase the damping, it is far more effective, and less expensive, to install a damping system. Damping systems increase the amount of energy that is dissipated by the structure [14].

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Bridges to Prosperity is a United States-based nonprofit organization that partners with local governments to connect their rural last mile via pedestrian bridges. One of the most popular footbridges which designed and constructed by Bridges to Prosperity is cable suspended footbridge. However, these bridges have a low mass, stiffness and damping. So, they are susceptible to serviceability failures under pedestrian loading.

This study has attempted to investigate the effect Human-Induced Vibrations on Pedestrian Cable Bridges. Cable suspended footbridge which is addressed in this particular study has been designed and constructed by bridge to prosperity.

The present study, investigated how bridge span length, walkway and handrail cable sag and cable diameter and number of pedestrians affect modal vertical frequency and dynamic response of cable suspended footbridge due to human induced loads. From this research, the following conclusions are made.

- For all modal cases vertical modal frequencies decrease as the bridge span length increase which mean shorter bridge span lengths have higher vertical modal frequencies.

- Smaller cable sags have higher vertical modal frequencies than higher cable sags. As the main cable diameter increase the bridge become stiffer. So, the vertical modal frequencies increased. Modal frequencies not affected by number of pedestrians.

- For most cases shorter span lengths have higher vertical acceleration. However, 60m span length for jogging load case with 5 percent cable sag and 32mm cable diameter experienced greater vertical acceleration than that of 40m span length.

- Greater vertical acceleration experienced by higher number of pedestrians. Because, the applied time varying load increase as the number of pedestrians increase.

- The vertical acceleration decrease as the cable sag increases. So, bridge model with 5 percent cable sag experienced greater vertical acceleration than that of 7 percent cable sag.

- Smaller cable diameters have higher vertical acceleration than higher cable diameter. And the vertical accelerations felt by walking and jogging load case have been within the comfort limits for bridge model with 7 percent cable sag and 32mm cable diameter.

- As the vertical modal frequency increased and closed to forcing pedestrian vertical frequency the vertical acceleration would be increase. So, if it is possible construct cable suspended footbridge using higher cable diameter and cable sag is prefer to reduce the effect of vibration.

- Jogging load case experienced greater vertical acceleration than walking load case. And greater vertical acceleration expected for shorter span length. So, Remedial measure like stiff the structural component or provide damping element necessary for bridge model with jogging load case and shorter span length.

- The result of this particular study indicated that cable sag is the most sensitive parameter because as this parameter change the vertical acceleration also dramatically change.

- For most cases 42m span length and 5 percent cable sag have higher vertical acceleration. So, in order to make the vertical acceleration within the recommended range, the span length of suspended cable pedestrian bridge should be greater than 42m and it is also advisable to use 7 percent cable sag.

5.2 Recommendations

Based on the finding on this thesis, the following recommendations for future work within this field are drawn.

- One should conduct this kind of research on other type of bridge like concrete, composite, suspension and cable stayed footbridges.

- One should conduct research on the parameters that are influencing the responses of UN national annex recommended moving harmonic loads. This could lead to a general guideline which describes how to model structures and loads to avoid errors.

- One can extend this research by considering lateral dynamic response.

- One can consider other Moving or non-moving load cases and other loading scenarios including groups of people (Crown) to investigate the effect of human induced vibration on suspended footbridges.

- Consider longer span lengths, other cable diameter and cable sag for cable suspended footbridges.

- Investigate the effect of adding mass in different locations along the bridge to improve the dynamic response and reduce the vibration problem.

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APPENDIX A: MIDAS/Civil Software Verification

A.1 Physical models

Jennifer Kearney and Jeffrey A. Laman analyzed two scaled, laboratory physical models and conducted simulations on forty numerical models to determine how particular design parameters affect modal frequencies and the dynamic response as compared to human comfort limits.

Two physical models, shown in Figures 1 and 2, Scaled models of a 40 m span bridge with 5 percent cable sag and a 80 m span bridge with 7.5 percent cable sag were constructed. The scaled physical models were designed at 1:18 scale of suspension footbridges. These two span lengths and sags are the extremes for the bridges evaluated. In addition, the physical models are based on the standard dimensions from the Bridges to Prosperity Design Manual.

The physical model design included setting the scale factor for the models relative to the full scale suspension footbridges and determining materials for the models. The materials were selected based on mass, which is the controlling parameter.

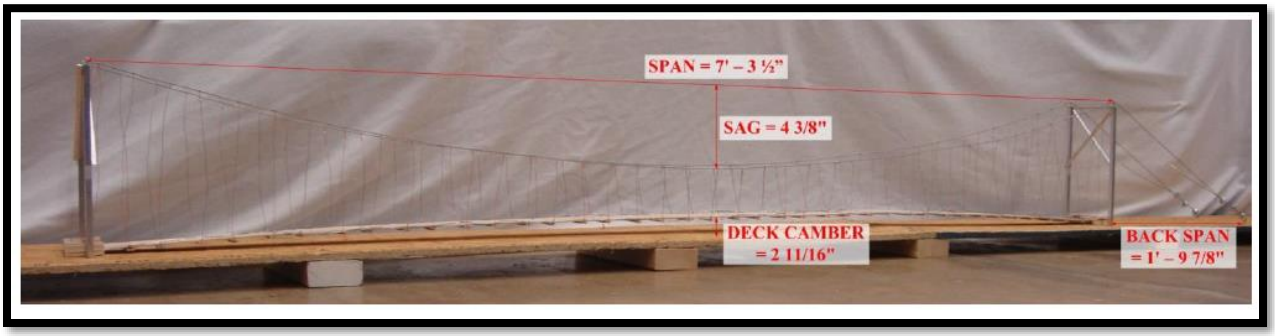


Figure 1 Scaled 40 m physical model with dimension [5].

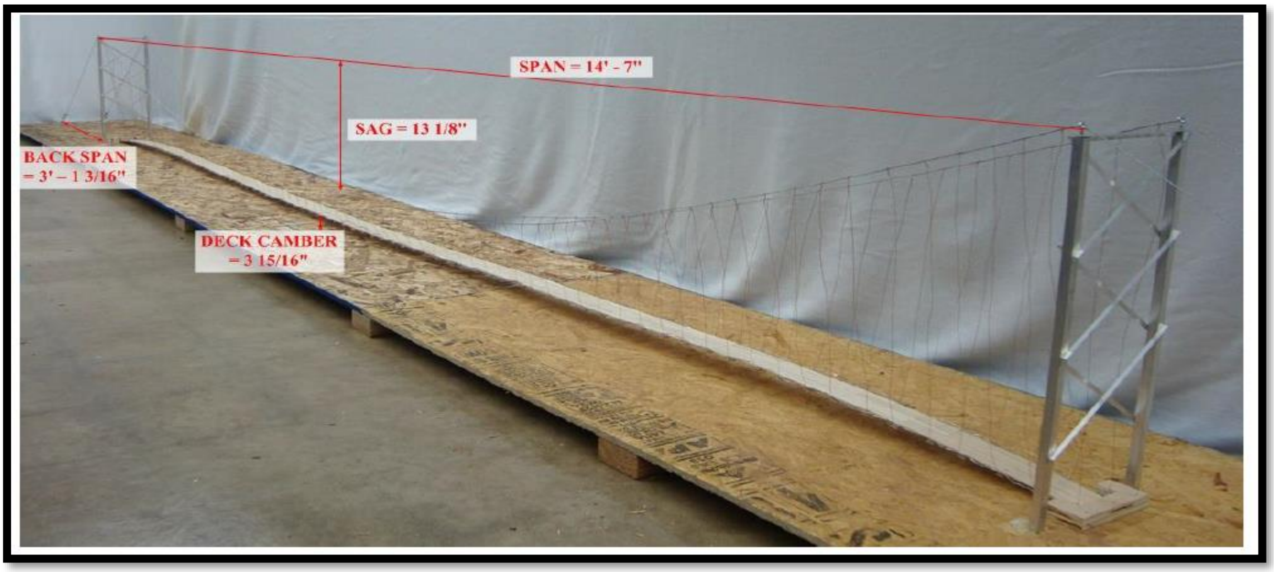


Figure 2 Scaled 80 m physical model with dimension [5].

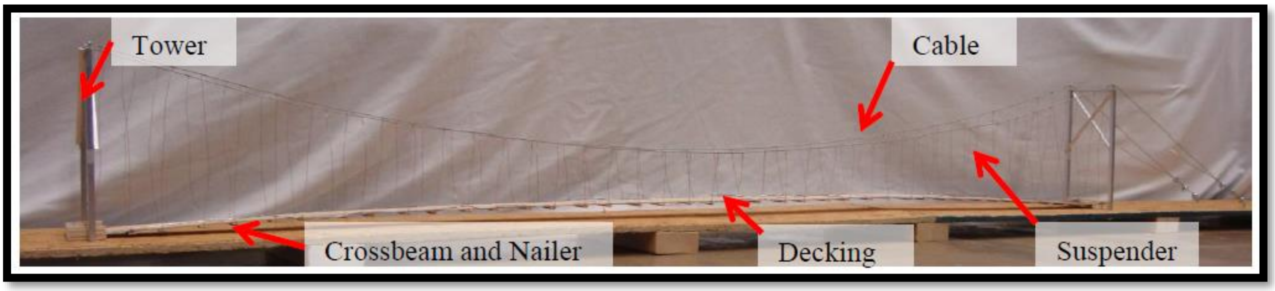


Figure 3 Suspension Bridge Model Elements [5].

Table 1.1 Model Materials.

Element	Actual Bridge	Model Structure
Cable	1⅛" diameter cable	1/16" diameter cable
Suspenders	9 mm diameter rebar	24 gage copper wire
Crossbeam	2-L 1¾" × 1¾" × ¼" bolted back-to-back	0.032" thick aluminum plate (7/16" x 2 5/8")
Towers	Hollow steel pipe with angle bracing	⅜" or ½" hollow, square aluminum tube with ¼" aluminum angles
Nailers	2"×8" wood board	3/32" thick basswood pieces (1/16" x 2 3/8")
Decking	2"×8" wood board	3/32" thick basswood pieces (1/16" x 2 3/8")
Fence	Fence, hand cable, and cable clamps	4 pieces of 24 gage steel wire

The response data from the model bridge test was collected through the use of a Casio EX-F1 high speed video camera. The camera is set to a rate of 300 frames per second. Then the videos collected with the high speed digital camera were imported into Tracker, Which is a video analysis program.

Therefore, two physical models of a 40 m bridge with 5 percent cable sag and an 80 m bridge with 7.5 percent cable sag were tested to calibrate the numerical simulations.

A.2 Numerical models

Numerical models have been analyzed in MIDAS/Civil to determine the modal frequency and compare with physical modal result.

To correctly represent the physical model the main cables and suspenders cables were modeled with cable elements. Tower model with pipes. Nailers (wood members attached to crossbeams) and the safety fence were modeled as a distributed load and joint masses, respectively, rather than as structural elements. Boundary conditions were idealized as follows: support tower columns are pin connected; suspension cables are pin connected at the anchors; deck ends are supported by 1.25KN/m longitudinal and lateral springs.

The nailers, which are wood boards attached to the double angle crossbeams for ease of nailing the decking boards, are not modeled in MIDAS/Civil explicitly. Instead, the nailers are represented as a distributed dead load centered on the crossbeams that acts along the length of the standard nailer. In addition, the fence and hand rail cable are represented in MIDAS/Civil as 0.028KN joint masses on the ends of every crossbeam.

Material Definitions and dimension for each element on MIDAS/Civil shown in table 1.8b and 1.8c. The tower geometry and steel pipe size taken from bridge to prosperity manual and differ depending on the span length.

Table 2.1: Material Definitions for Each Element.

Element	Material	Modulus of Elasticity (KN/m^2)	Density (KN/m^3)
Tower Pipe	Steel	2.00E+08	78.5
Tower Angles	Steel	2.00E+08	78.5
Crossbeams	Steel	2.00E+08	78.5
Suspenders	Steel	2.00E+08	91.1
Wood	Native Hardwood	2.00E+08	11.66
Cable	6.4mm Dia. Steel wire cable	1.20E+07	91.1

A modal analysis was conducted of each model to determine the mode shapes and corresponding frequencies. The modal analysis was set as an Eigen vectors analysis. The modal analysis starts from the end of the nonlinear dead load analysis to evaluate the mode shapes of the structure under self-weight.

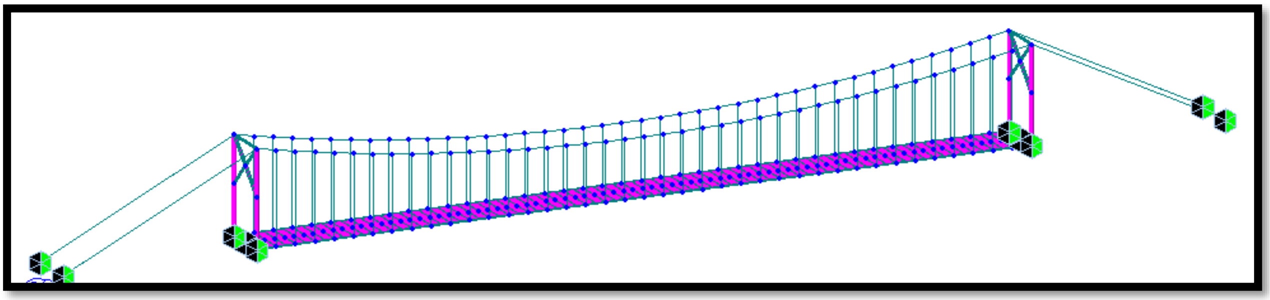


Figure 2.1 MIDAS/Civil Model for 40m span suspension footbridge.

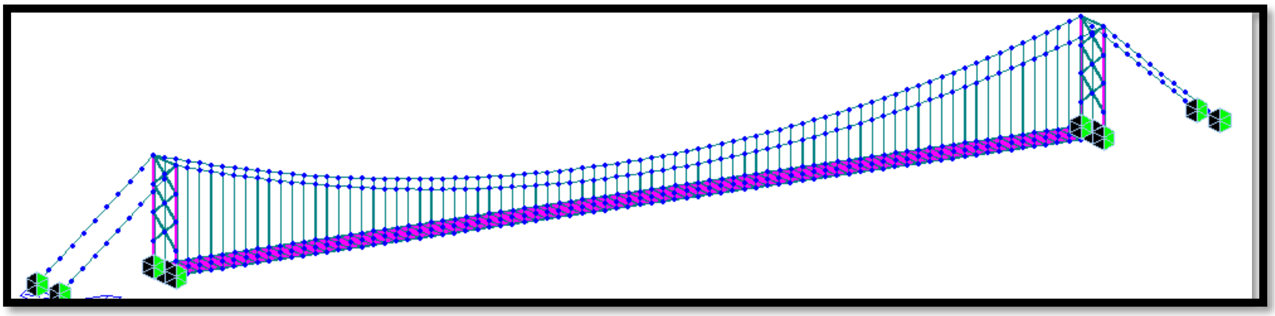
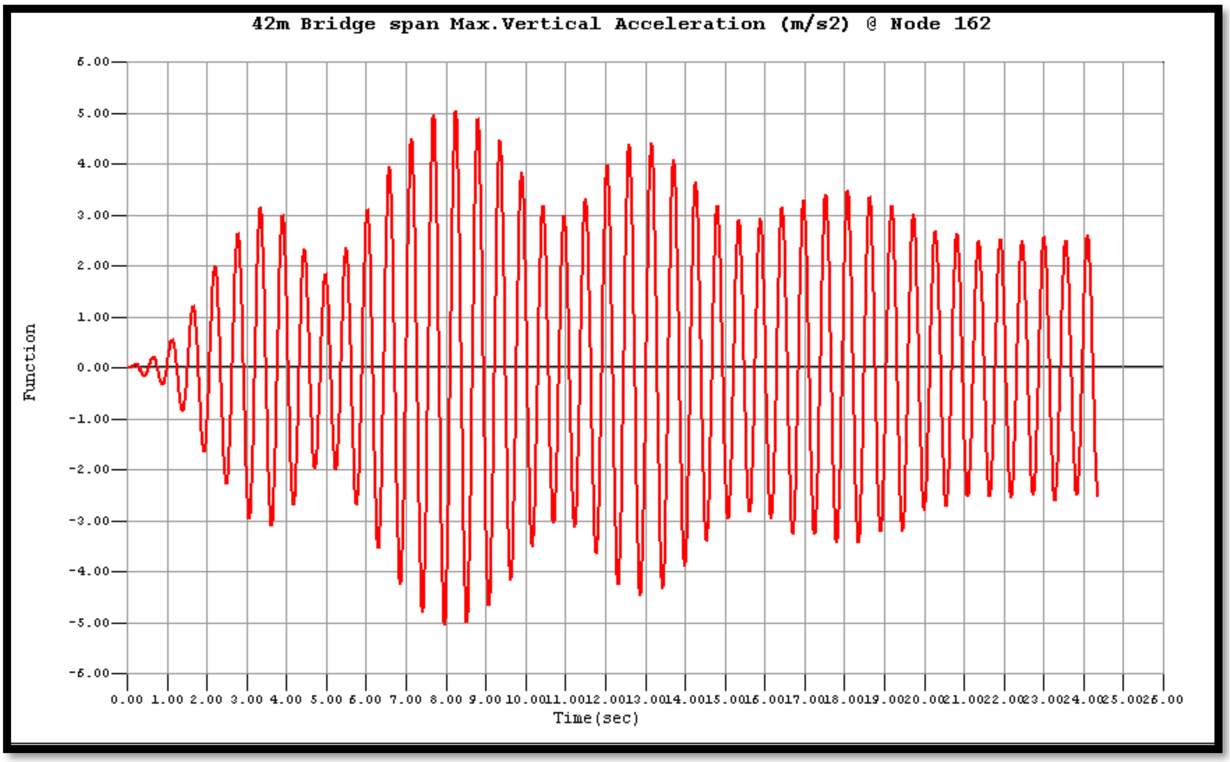


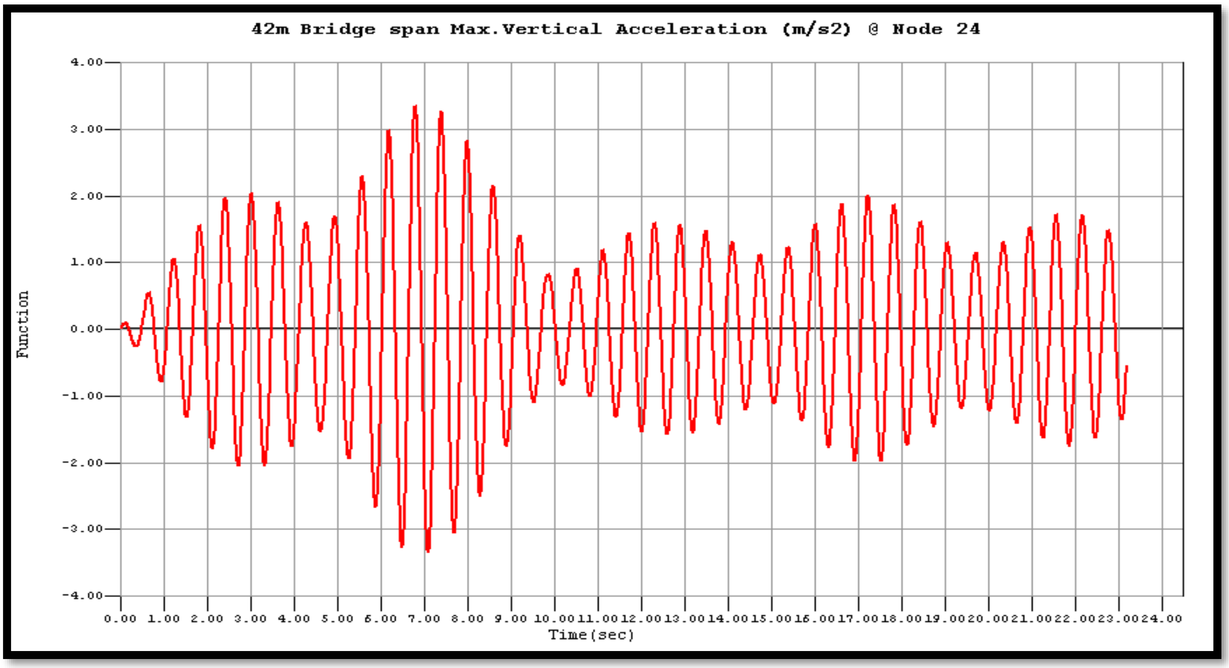
Figure 2.2 MIDAS/Civil Model for 80m span suspension footbridge.

APPENDIX B: Maximum Vertical Acceleration for Walking pedestrians (m/s^2)

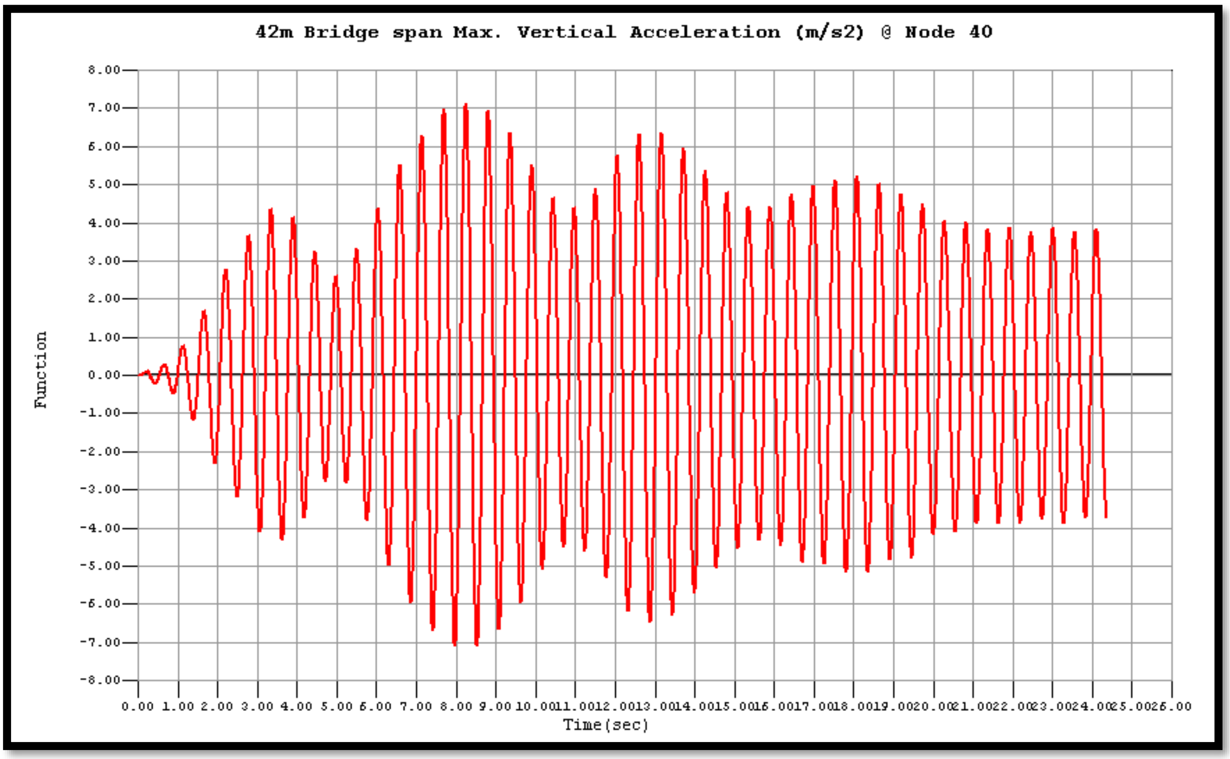
B.1 Number of pedestrians (N=2), 5 Percent Cable Sag and 26mm cable diameter



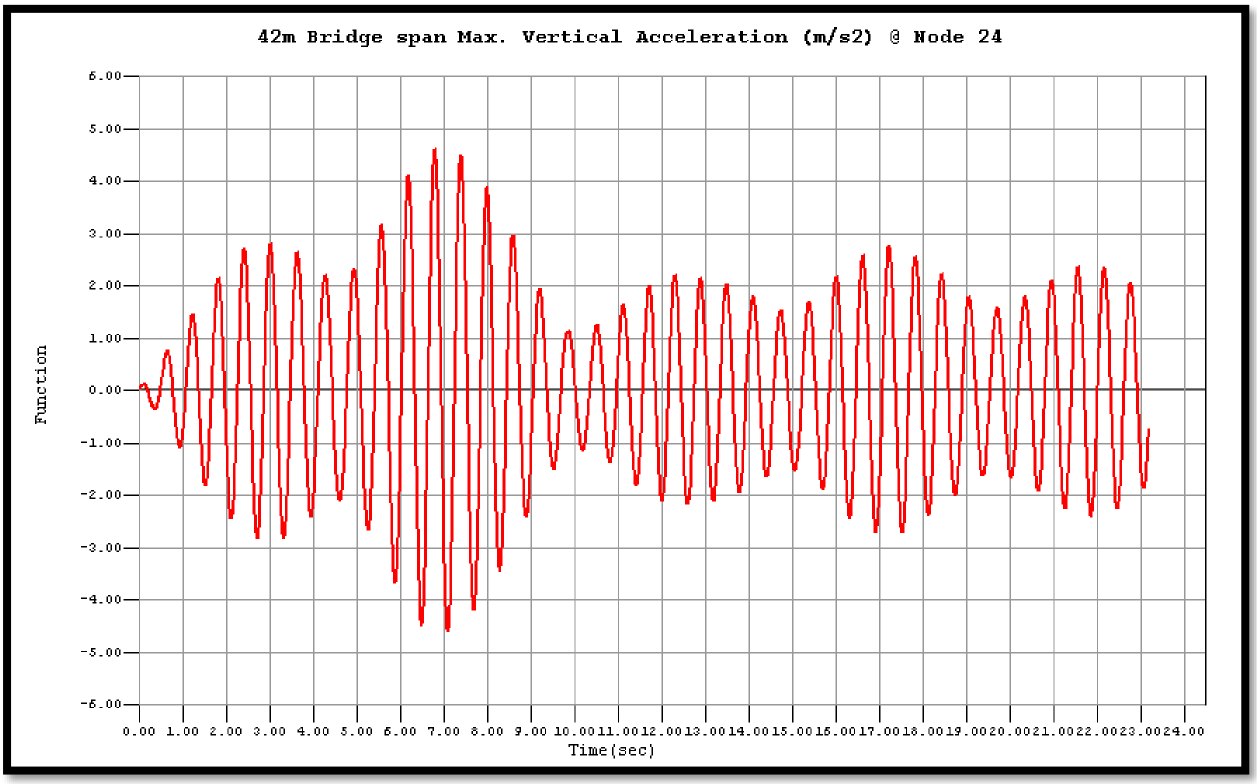
B.2 Number of pedestrians (N=2), 7 Percent Cable Sag and 26mm cable diameter



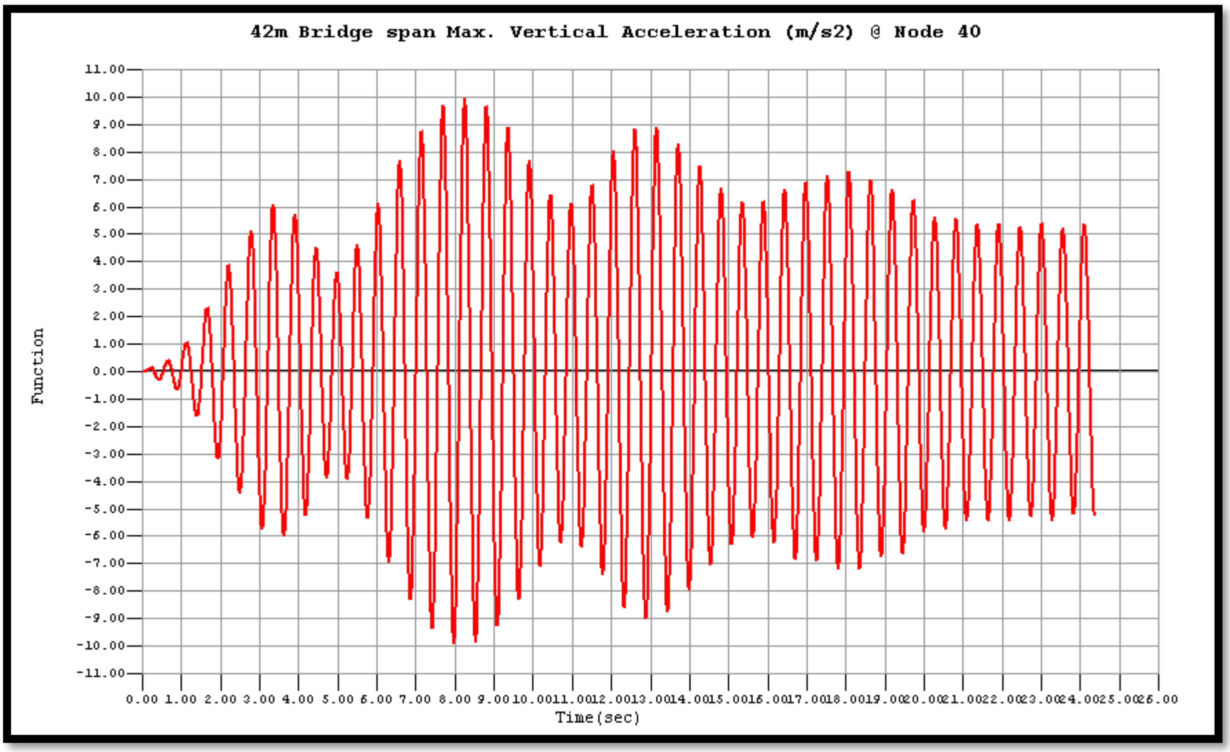
B.3 Number of pedestrians (N=4), 5 Percent Cable Sag and 26mm cable diameter



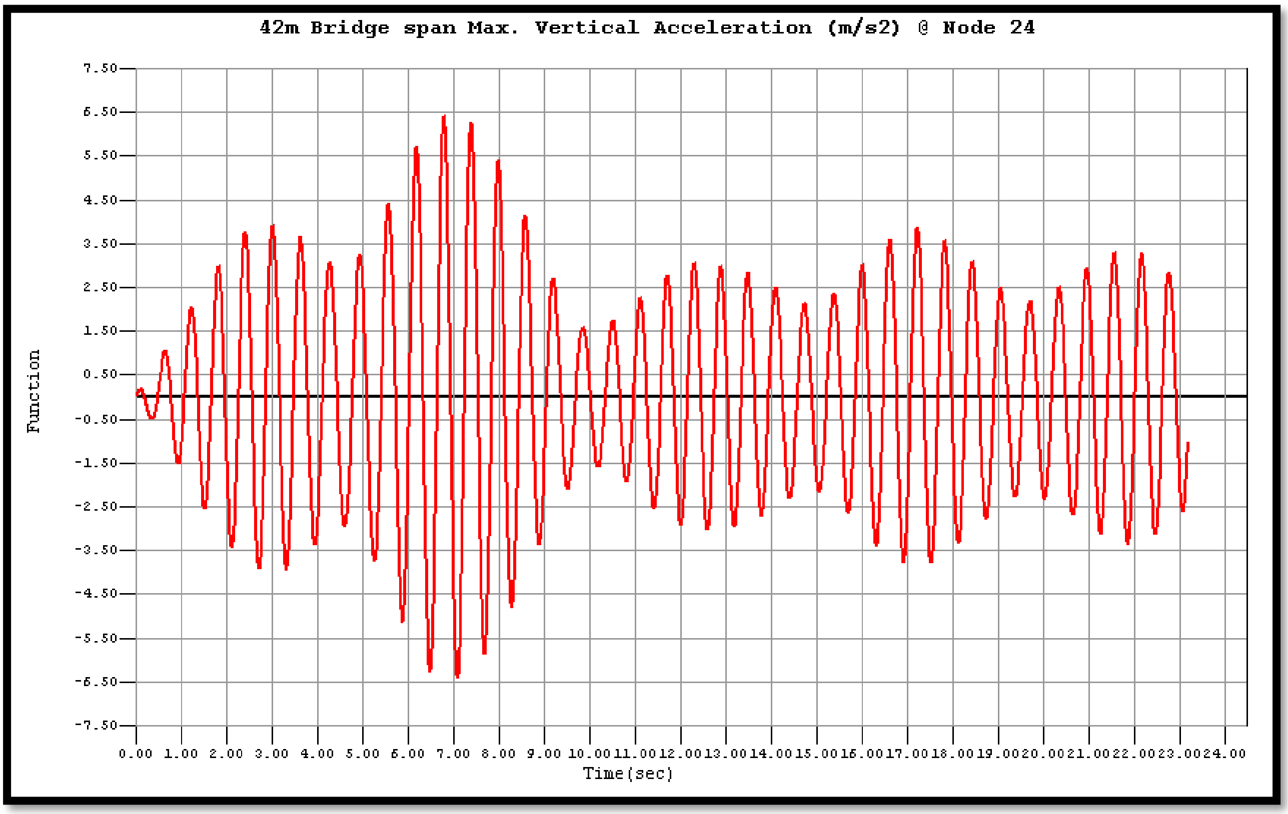
B.4 Number of pedestrians (N=4), 7 Percent Cable Sag and 26mm cable diameter



B.5 Number of pedestrians (N=8), 5 Percent Cable Sag and 26mm cable diameter

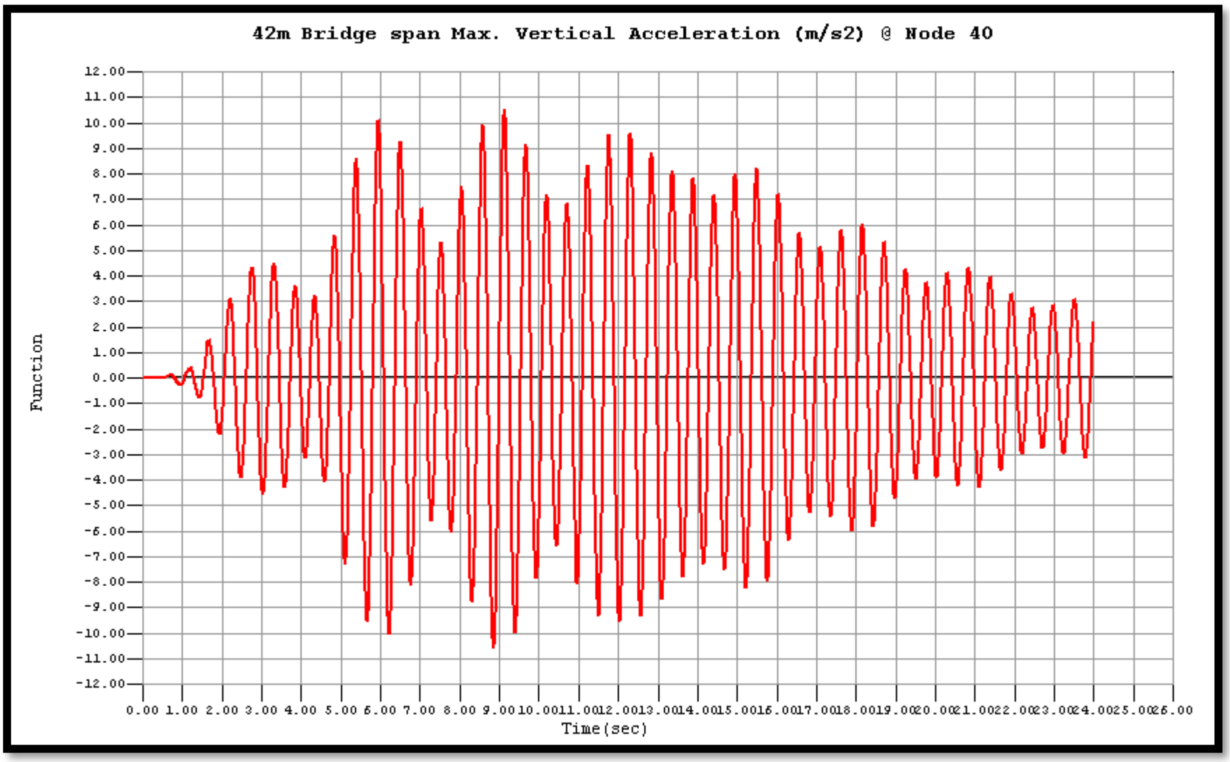


B.5 Number of pedestrians (N=8), 7 Percent Cable Sag and 26mm cable diameter

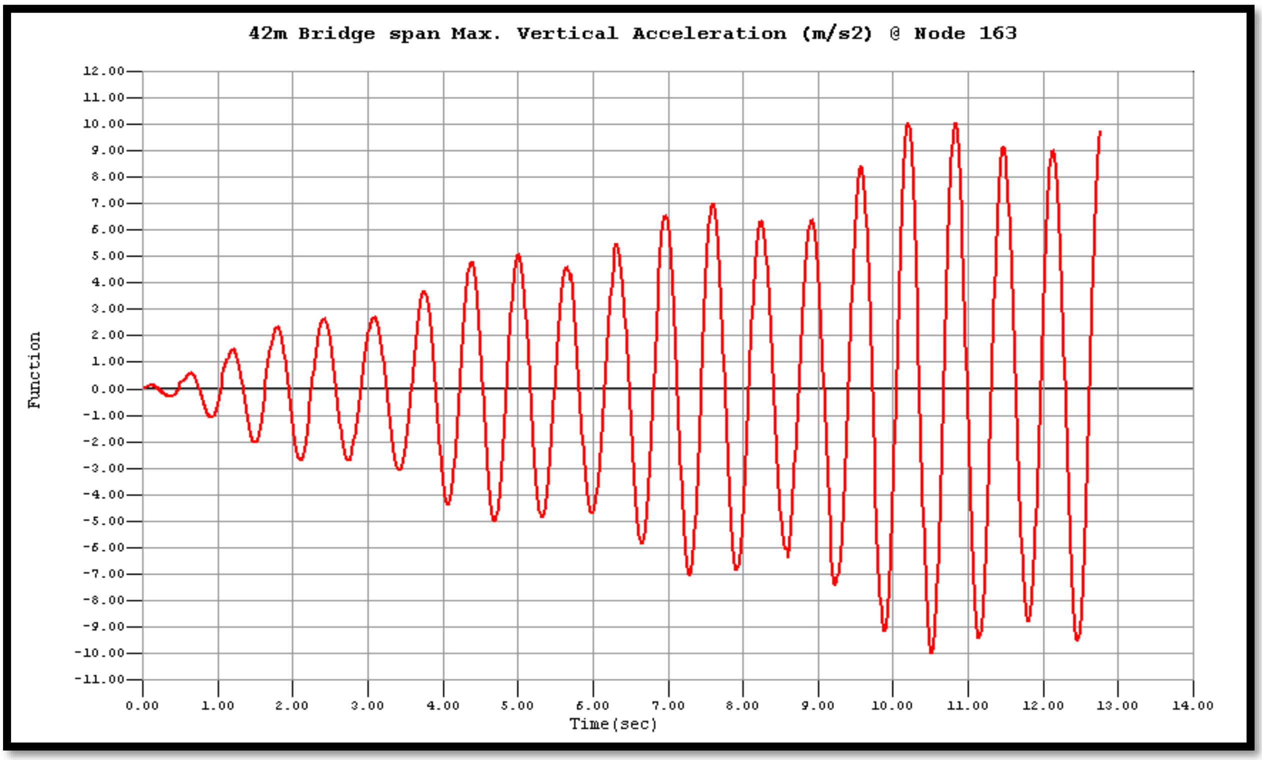


APPENDIX C: Maximum Vertical Acceleration for Jogging pedestrians (m/s^2)

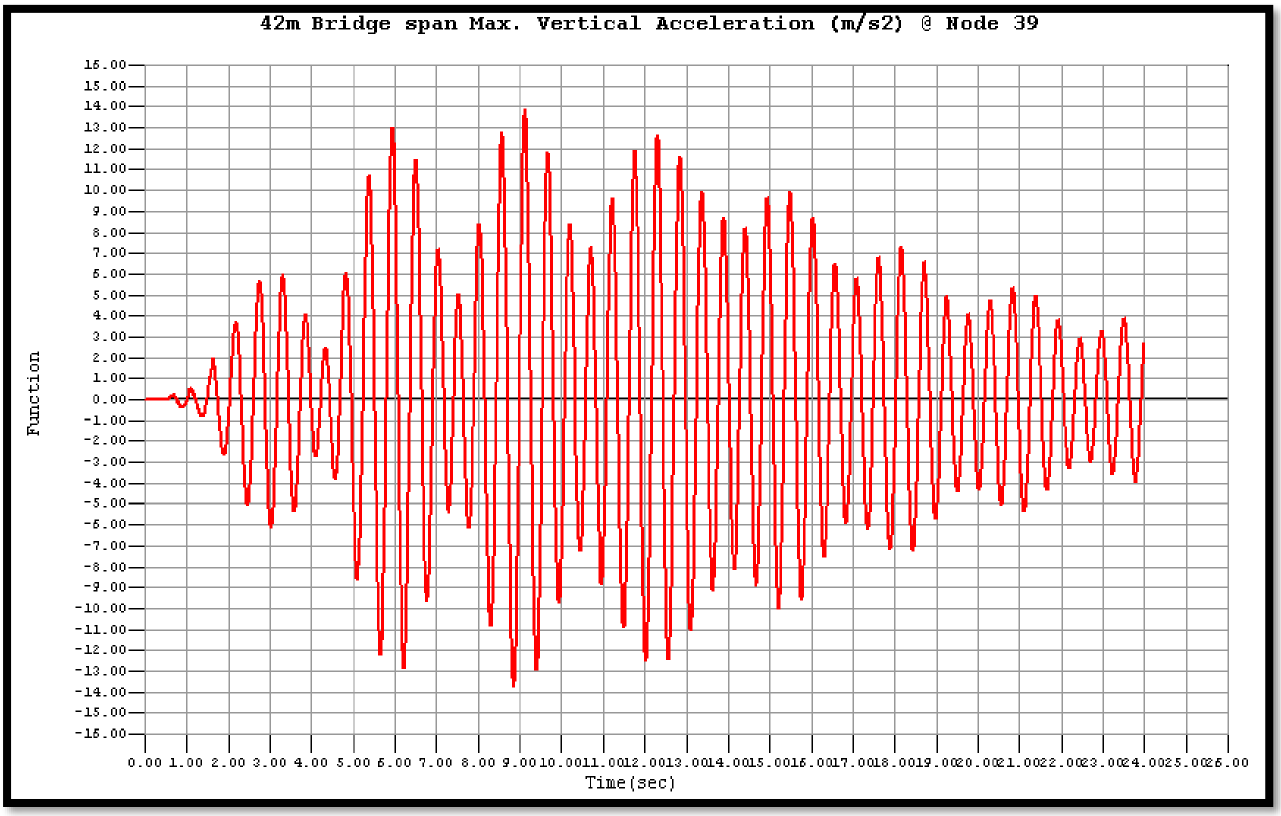
C.1 Number of pedestrians (N=1), 5 Percent Cable Sag and 26mm cable diameter



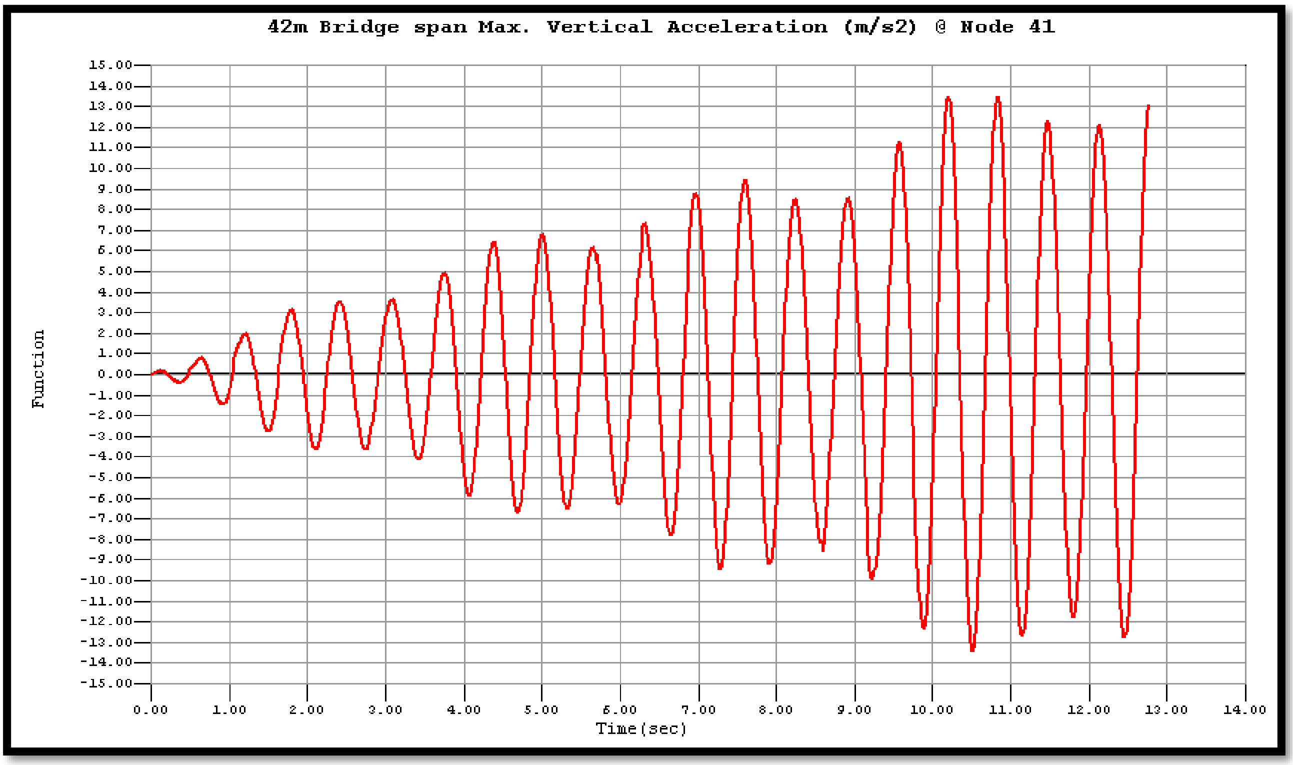
C.2 Number of pedestrians (N=1), 7 Percent Cable Sag and 26mm cable diameter



C.3 Number of pedestrians (N=2), 5 Percent Cable Sag and 26mm cable diameter



C.4 Number of pedestrians (N=2), 7 Percent Cable Sag and 26mm cable diameter



APPENDIX D: Cable size and quantity selection example for span of 42m

- To select the size and quantity of cables, first calculate the load demand on the cables. Using a one meter wide walkway width:

1. Bridge Layout

Span,L	=	42	m
Deck Width	=	1.06	m

2. Check Height Difference

Max Height Difference = L/25 = 1.68 m (From manual)
Height Difference (ΔH) = 0 m (In this study)

0 <= 1.68 **Ok**

3. Loading

Dead Load (DL)

(The dead load is the sum of all self-weights of the bridge materials)

Loading

Suspenders=	0.006	KN/m
Deck =	0.261	KN/m
Cross beam =	0.057	KN/m
Cable =	0.162	KN/m
Total =	**0.488**	**KN/m**

Live load (LL)

$$w = 4.07\frac{KN}{m^2}.\left(0.25+\frac{4.57}{\sqrt{A1}}\right), \quad w \geq 3.14\frac{KN}{m^2} \quad and \quad w \leq \ 4.07\ \frac{KN}{m^2}$$

W=the design live load (KN/m^2)

A1=the walkway area (m^2)

LL= 3.785 KN/m

Total distributed load = DL+LL = 4.273 KN/m

4. Cable Sag Values

Design load sag = 5%*L = 2.1m

5. Cable Force Analysis

Right & Left Tower Cable Angle	=	11.33 Degree
Horizontal Tension, Ph Mtotal	=	448.709 KN
Vertical Tension, Pv Mtotal	=	89.741 KN
Left & Right Axial Tension, Pt Mtotal	=	457.867 KN
Maximum Cable Tension, Pr	**=**	**458 KN**

For this project, 26mm diameter and 32mm diameter cables are available with breaking strengths of 396 kN and 585 kN, respectively. The required cable safety factor is 3.0. (From Manual)

6. Cable Selection

	Diameter	Breaking Strength
Currently available cable:	26mm	396 KN
	32mm	585 KN

Determine the number of cables required for each cable diameter:

Number of Cables Required, Cr= $\frac{\mathrm{Pr}*FS}{\mathrm{breaking}\ Strength}$

26mm = 3.470

32mm = 2.349

The minimum number of cables (4) controls for both 26mm and 32mm diameter cable. Since both cables have sufficient strength, select the 26mm diameter cable.

7. Cable Safety Factor

	Cable No.	Size	Strength	Total Capacity
Handrail =	2	26mm	396	792 KN

Walkway =	2	26mm	396	792	KN

Maximum Cable Capacity, Pn = 1584 KN

Factor of sefety $= \frac{Maximum\ Cable\ capacity\ (Pn)}{Maximum\ Cable\ Tension\ (Pr)} = \frac{1584}{458} = 3.45$

$= 3.45 > 3$ **Ok**

APPENDIX E: Cable size and quantity selection example for span of 100m

1. Bridge Layout

Span, L = 100 m
Deck Width = 1.06 m

2. Check Height Difference

Max Height Difference = L/25 = 4 m (From manual)
Height Difference (ΔH) = 0 m (In this study)

0 <= 4 **Ok**

3. Loading

Dead Load (DL)

(The dead load is the sum of all self-weights of the bridge materials)

Loading

Suspenders= 0.006 KN/m
Deck = 0.261 KN/m
Cross beam = 0.057 KN/m
Cable = 0.162 KN/m
Total = 0.488 KN/m

Live load (LL)

$$w = 4.07\frac{KN}{m^2}.\left(0.25+\frac{4.57}{\sqrt{A1}}\right), \quad w \geq 3.14\frac{KN}{m^2} \quad and \quad w \leq 4.07\frac{KN}{m^2}$$

W=the design live load (KN/m^2)

A1=the walkway area (m^2)

LL= 2.808 KN/m

Total distributed load = DL+LL = 3.296 KN/m

4. Cable Sag Values

Design load sag = 7%*L = 7m

5. Cable Force Analysis

Right & Left Tower Cable Angle	=	15.64	Degree
Horizontal Tension, Ph Mtotal	=	588.683	KN
Vertical Tension, Pv Mtotal	=	117.736	KN
Left & Right Axial Tension, Pt Mtotal	=	613.211	KN

Maximum Cable Tension, Pr = 614 KN

For this project, 32mm diameter cables are available with breaking strengths of 585 kN. The required cable safety factor is 3.0. (From Manual)

6. Cable Selection

	Diameter	Breaking Strength	
Currently available cable:	26mm	396	KN
	32mm	585	KN

Determine the number of cables required for each cable diameter:

$$\text{Number of Cables Required, Cr} = \frac{\mathrm{Pr} * FS}{breaking\ Strength}$$

26mm = 4.652

32mm = 3.149

The minimum number of cables (4) controls for 32mm diameter cable. Since cables have sufficient strength,select the 32mm diameter cable.

7. Cable Safety Factor

	Cable No.	Size	Strength	Total Capacity
Handrail =	2	32mm	585	1170 KN
Walkway =	2	32mm	585	1170 KN

Maximum Cable Capacity, Pn = 2340 KN

Factor of sefety $= \frac{Maximum\ Cable\ capacity\ (Pn)}{Maximum\ Cable\ Tension\ (Pr)} = \frac{2340}{614} = 3.811$

= 3.81 > 3 **Ok**