MINOR DISSERTATION

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ii. ACKNOWLEDGEMENT

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4	Bukhosi Nyoni	UCT MSc Candidate	Assistance with the Field Work
5	Andre Goodhead	UCT MSc Candidate	Assistance with the Field Work

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v. ABSTRACT

The focus on the condition and performance of existing structures has increased due to the growing number of structures approaching or exceeding their design life. The challenges associated with assessment of existing structures include deterioration, changes in loading conditions, a change in the function or the structure reaching the latter portion of its designed service life. In order for bridge authorities to better determine how to deal with existing bridge structures, there must be a coherent means of determining, measuring and benchmarking the current condition and performance of the structure. The current study proposes and demonstrates an integrated visual inspection based condition assessment and vibration based condition assessment of railway bridges. The methodology suggests at a systematic visual assessment combined with the development of a finite element model which will be calibrated by using modal parameters ascertained from vibration based testing of the rail bridge. The bridge which was used as a case study was the Kalbaskraal railway bridge located in Malmesbury.

The proposed methodology consists of the following steps:

1) Initial Assessment
2) Development of a Finite Element Model
3) Detailed assessment and Ambient Vibration Field Testing
4) Analysis of Modal Parameters
5) Verification of FEM by Modal Parameters
6) Setting up Load Configurations
7) Assessment of structural response
8) Assessment of Serviceability limit state of bridge

The overall outcome of the study yielded an effective result in that the conclusions drawn from the outcomes of the methodology correlated well with previous studies. The structure under its current operational load of 16ton/axle wagons performed within the allowable serviceability limit state. A proposed increase to 22.5ton/axle loads identified that the bridge would be performing on the boundary or above the allowable serviceability limit state and that retrofitting may have to be considered for the bridge to effectively support the additional load.

The results derived from this study can be extremely valuable in the bridge management process as the information on the condition of the bridge can aid bridge authorities in their decision making processes.

CONTENTS

1 INTRODUCTION

1.1 Background

Structural engineers are increasingly working on existing structures due to the growing number of structures approaching or exceeding their design life. The challenges associated with assessment of existing structures include deterioration in a manner that was not fully understood during design, a change in loading conditions, a change in the function of the structure or the structure reaching the latter portion of its designed service life. In order to better determine how to deal with an existing structure there must be a coherent means of determining and measuring the current condition and performance of a structure.

South Africa's railway network consists of various types of bridges. Many of the older bridges were wrought iron truss bridges which are reaching or exceeding their design life. They are generally constructed of riveted built up steel sections. With rail traffic and loading increasing due to economic growth, these older structures now have to handle higher loading conditions in a deteriorated state. There are proven methods of determining the current performance and condition of bridges however South African Authorities have not yet holistically adopted the new approaches. Current condition assessments are generally visual and are based mainly on expert engineering judgement. The traditional code-based approach is generally used and this in most cases is not sufficient to provide a realistic structural assessment of a bridge.

The aim of this study is to demonstrate the value of the integration of visual condition assessment of railway bridges and vibration based condition assessment.

1.2 Purpose of Study

The study proposes and demonstrates an integrated visual inspection based condition assessment and vibration based condition assessment of railway bridges.

1.3 Objectives

- Propose a systematic procedure for the condition assessment of railway bridges
- Application of the procedure on the Kalbaskraal Rail bridge

1.4 Limitations

The limitations of this study is a follows:

- Non-destructive, destructive and Material testing were not available in the implementation of this study.
- Fatigue was not considered in this study

2 LITERATURE REVIEW

2.1 Bridge Management

2.1.1 Principles

In recent years, the problem around the sustainability of infrastructure over its service life has compounded. Infrastructure throughout the world are being investigated more intently as they reach the end of their service life. This is generally due to the fact that they are deteriorating and the capital cost for replacement is very high. In most cases, such as bridges, they are required to stay in operation well beyond their design life thus they would require some kind of retrofit or a complete replacement. The compounding problems in infrastructure over time can be attributed to several reasons, such as:

i. Under investment in public works programmes
ii. Failure to recognise the importance of maintenance
iii. Cut backs on maintenance budgets
iv. Tendency for authorities to defer maintenance
v. Failure to recognise obsolescent of infrastructure (Hudson, et al., 1997)

A well thought out infrastructure management system based on technical, financial and strategic outcomes would address these areas and assists in the decision making process of authorities. The optimum infrastructure management system would consist of a set of processes and tools to enable the optimum use of available funds to maximise the performance of the infrastructure (Hudson, et al., 1997). This however can only be implemented with a sound understanding of the requirements of the infrastructure. In the case of bridges for example we must have a means of determining the remaining life which is dependent on:

i. The effects of loading
ii. Type of construction
iii. Material Deterioration mechanisms
iv. Damage experienced over the structures life
v. Fatigue
vi. Etc.

In the life cycle management process, key decisions must be made on how the bridge will be managed to ensure effective utilisation and safety. This should be based on the condition and performance of the structure to determine if it is performing as per its design, it is fulfilling its function optimally and is in a safe condition. These matters all relate to the overall understanding of the state of the infrastructure and understanding the inputs required to make overall decisions on a way forward. Theoretically these matters can be addressed by a systematic approach to managing infrastructure. Below is a graph indicating the service life of a generic unit of infrastructure which makes a comparison between a proactive and reactive strategy.

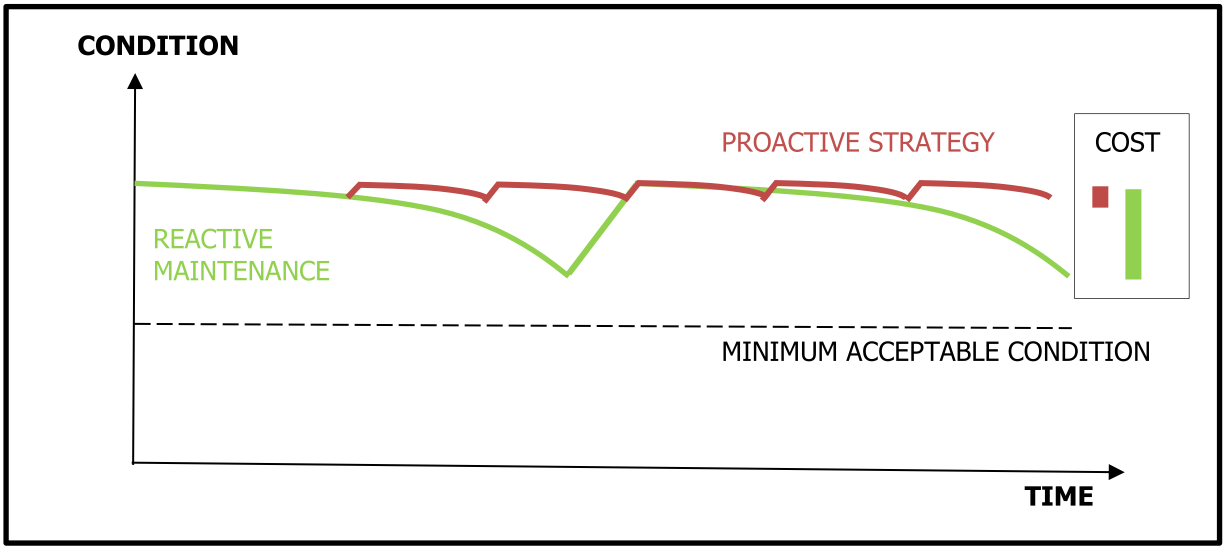


Figure 1: Figure illustrating a proactive vs reactive asset life cycle. (Nordengen, 2013)

In order to ensure that a structure performs as predicted, a sound proactive management/ conservation strategy is needed which incorporates the development of sound routine inspections & maintenance programmes supplemented a sound condition assessment strategy to understand the status of the structure before a decision is made to repair, rehabilitate, replace or upgrade.

2.1.2 Bridge Management Systems

A Bridge management system is a systematic means of optimising the management of bridges through its lifecycle. As bridges become older, maintenance costs become higher and the challenges for the funding for repairs become problematic. A bridge management system helps to systematically inspect, understand the bridge condition, plan maintenance and prioritise repairs on a bridge network. It fundamentally utilises an IT system or data base of information acquired from process (eg: inspections, condition assessments, monitoring systems, FEM tools) put in place by technical experts which yield results that can be used in the decision making process for the management of a network of bridge structures. The systems and procedures provide valuable information for optimisation of maintenance and future planning (Nordengen & de Fleuriot, 1998).

Most BMS are based on visual inspections with diagnostic inspections only implemented once a larger scale rehabilitation project is initiated. This thesis will be focused on the importance of the outcomes that can be achieved from an early benchmark detailed condition and performance assessment as part of the condition module.

A bridge mangement system was developed by the CSIR in the mid-90's for road and rail authorities in South Africa. The system was implemented for authorities such as the National Roads Agency of South Africa and Spoornet. The framework used for a BMS is illusterated in the figure below.

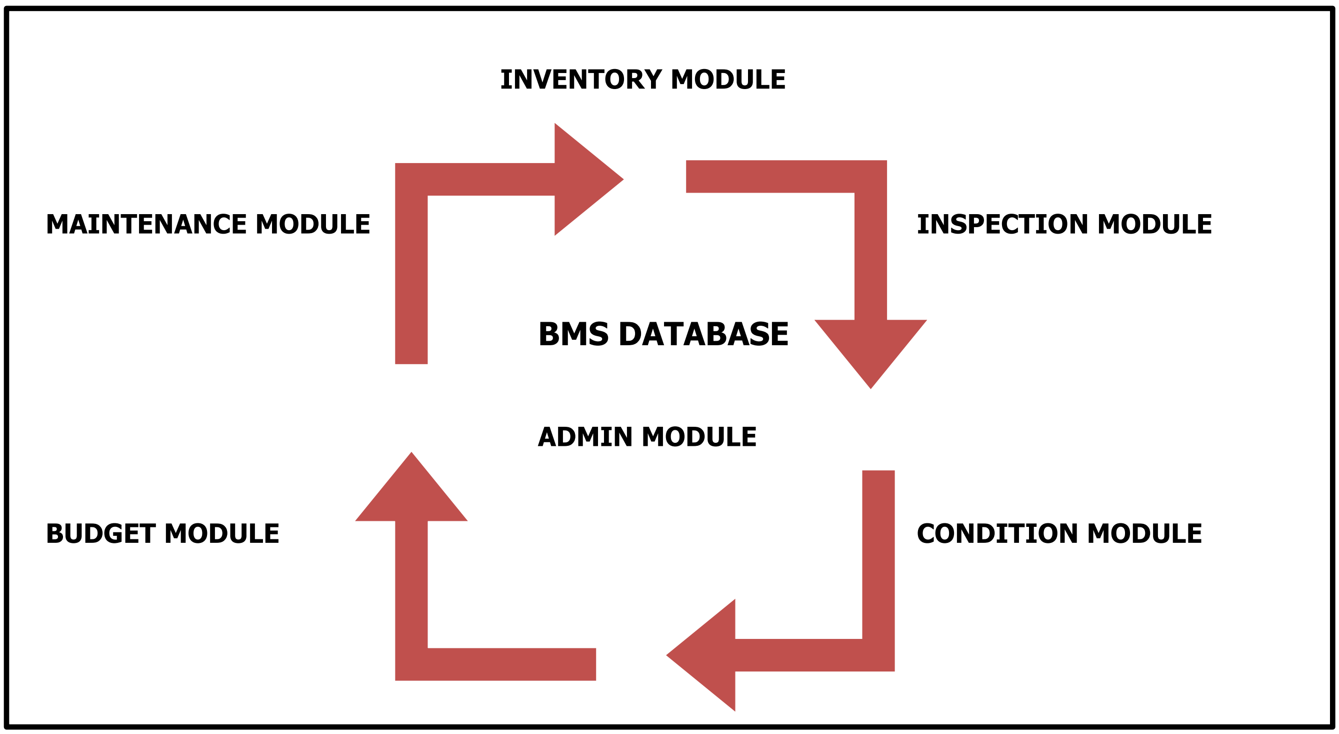


Figure 2: BMS framework Developed by CSIR for South African Authorities (Nordengen & de Fleuriot, 1998)

The system developed uses the basis of an IT system and professional expertise to document, plan and implement the effective management system.

The development of a BMS requires the consolodation of the network inventory. This involves the collection of the bridge information within the network to form a database. Comprehensive details the bridge dimensions, bridge type, field measurements and as built drawings to name a few would be useful. Information on design aspects such as loading would be significant. The inspection module involves a systematic means of inspecting the bridges by competent inspectors and recording information to acertain the qualitative condition of the bridge. The BMS system in question, requires subdividing the bridge into 21 components rangind from the foundations, bearings, deck, joints, drainage, etc. Principle inspections are carried out every 3 to 5 years for identification and monitoring inspections are carried out to assess the progression of defects identified in the principle inspections. The condition module looks prioritising the aspects of the brigdes by means of a specifically developed rating system. The system uses a priority index which is based on the calculated condition of the 21 components. The standardised inspection sheets were designed in order to avoid any oversights and the rating system was based on categorising defects by their DER:

- D represents the degree or severity of the defect
- E is the extent of the defect on the item under consideration
- R is the relevancy of the defect. This rating considers the consequences of the current status of the defect with regard to the serviceability of the bridge

Priority can be given to the significant components of the bridge such as piers. The priority index identifies the worst components of the bridge and prioritises based on this ignoring the components in good condition. The concept relates the philosophy that a chain is only as strong as its weakest link. The condition rating also considers all components of the bridge in the calculation to give an overall condition rating of the bridge. The budget module is used to assign the repair work and funding to the bridges over the upcoming years. Cost estimates are developed using a cost database and planned repair combined with the prioritisation process in the condition module to assign work. The relevency of the components is then assessed to determine the most effective risk reduction to the structure while also considering the cost, to optimise the programming of the maintenance schedule. The maintenance module is the system used to record the work done, cost and other significant date related to the maintenance of a bridge to ensure that the information can be noted for future use and to close off the defects (Nordengen & de Fleuriot, 1998).

2.2 Condition Assessment

The bridge management system described above is developed based on a wider framework illustrated below:

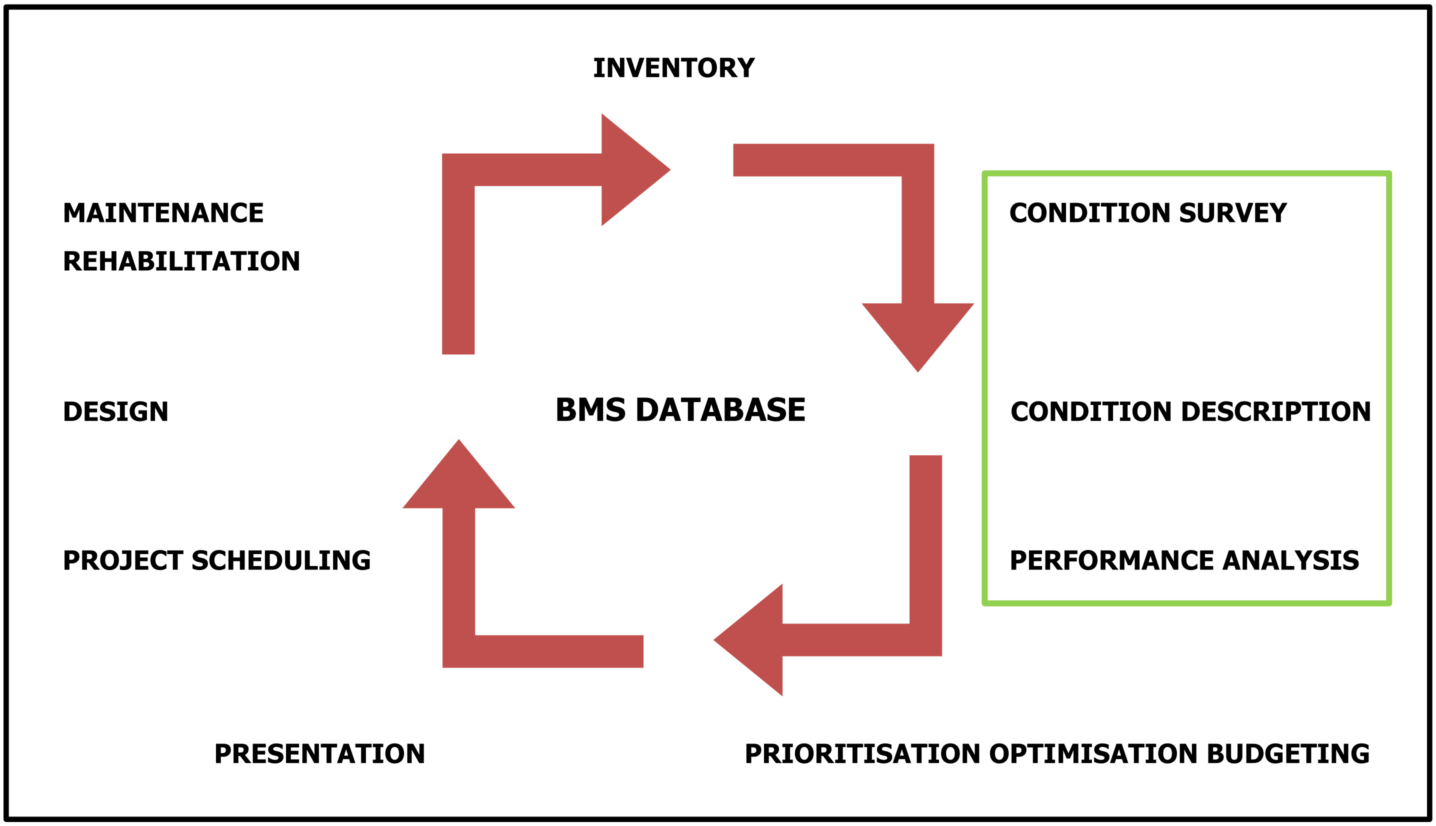


Figure 3: An overall BMS framework (Nordengen, 2013)

Condition Assessments are a key component of the Infrastructure Management process as it provides an up to date benchmark of the current state of a structure and provides valuable information to the owner for management of the structure. Although a structure has been designed for certain design criteria, this could change for different reasons throughout the life cycle of a structure. The structure itself could also change due to deterioration or physical damage. It is critical to understand the limitations of high risk structures such as steel rail bridges where the consequence of failure could be catastrophic. The reasons for condition assessment could be:

i. Deterioration of structural members (ie: corrosion)
ii. Mechanical Damage
iii. Change of the structures function
iv. Change or increase in loading

(Rathnayaka & Dissanayake, 2008)

2.2.1 Initial/Principle Assessment

The initial assessment is a qualitative exercise in evaluating the condition of the bridge. This will help identify significant changes, defects or unforeseen aspects which may trigger a detailed assessment. It is a means to identify aspects that may affect the bridges structural behaviour or capacity. The initial condition assessment encompasses the investigation of drawings, records of the structure and implementation of a visual inspection. A strategic visual inspection identifies defects that have manifested on the surface of a structure which may lead to defects or changes in the condition of the members, the fixity of connections and the condition of the supports. Conclusions regarding defects and problems can however only be drawn from the surface condition if the mechanisms of deterioration, failure and structural behaviour are understood by the inspector, thus the investigation of the bridge drawings and documents are essential. The information gained from the initial assessment can then be used in the detailed assessment (if required) and in the structural analysis (Rathnayaka & Dissanayake, 2008).

2.2.2 Detailed Assessment

The purposed of the detailed assessment is to gain further understanding into the structural member conditions, defects, connections and the supports of the bridge structure that has been identified by the initial assessment. The most symptomatic defects for steel rail bridges would be corrosion and the condition of welded and bolted connections. These may significantly affect the performance of structural members on the bridge as corrosion causes a loss in the cross sectional area in a member, which directly affects its structural capacity and a weak connection affects the fixity of a member which causes it to behave in a manner for which it was not designed. A detailed assessment would generally be supplemented by appropriate field testing such as non-destructive testing, strain measurements, deflection measurements, sampling for laboratory tests on materials or accelerometer measurements to ascertain modal parameters (Rathnayaka & Dissanayake, 2008).

2.2.3 Non-destructive Testing

Non-destructive testing (NDT) plays a key role in the assessment of structures. It provides a means of assessing defects on a structure that may not be visible from the surface level and could provide information on the in-situ material properties. NDT should be used as part of a complete assessment programme to utilise its full potential and the selection of the NDT test should be based on the desired output and the required precision. It is thus advisable to base the selection of the appropriate NDT methods on the visual assessment, expert advice to understand its limitation and a feasibility study (McCann & Forde, 2001).

A summary of some of the NDT methods available for civil engineering structures are described in table 1. The table is based on the review of NDT methods by McCann & Forde and provides some background and attributes of the different techniques that can be considered during the method specification for a condition assessment.

Table 1: Review of Non-Destructive Testing methods

NDT Method	Attributes	Parameter Measured	Cost
Sonics	Moderately slow; gives useful information on major elements	Wave velocities through structure	Moderate
Ultrasonic	Relatively quick but gives no information on major elements	Wave velocities through structure	High
Radar	Quick; can give good penetration; can give good image of internal structure but requires skilled interpretation	Electromagnetic wave velocity	Moderately high
Conductivity	Quick; gives relative conductivities over a large area to a maximum depth of 1.5 m	Relative conductivity	Low
Impact Testing	Gives some indirect measure of current condition but is difficult to quantify	Mode shapes and/or signature	Moderate
Vibration Testing	Gives some indirect measure of current condition but is difficult to quantify	Mode shapes and/or signature	High

2.2.4 Structural Modelling, Validation and Analysis

In order to provide meaningful input to a condition assessment, an understanding of the structural performance of the real structure must be assessed. Using the information from detailed drawings and the detailed assessment, a model can be developed for the structure, based on the requirements. A 3D Finite Element Model has been suggested based on research to be the best option as it is a very powerful analysis tool for structures with complex geometry. The complexity however comes with the assumptions and uncertainties made in the analytical model which will most likely vary from the real structure. This could be support conditions, joints, connections damage or deterioration (Rathnayaka & Dissanayake, 2008) .It is thus important to use the information from a well conducted detailed assessment in the validation of the model.

The validation process is the iterative process of tuning the uncertain parameters within the model with experimental results from field testing in order to ascertain the most accurate representation of the real structure. This can be done by comparing static or dynamic measurements from the real structure to the responses from the model (Rathnayaka & Dissanayake, 2008). The selected responses for the validation process in this thesis will be the modal parameters (ie: mode shapes and frequencies). This would require the use of operational modal analysis which is a form of dynamic testing to acquire the modal parameters of a structure.

Once a valid model is established, various structural analysis scenarios can be run to assess the current capacity of the bridge, its response to different load configurations and the identification of areas of concern which may require supplementary attention. This would assist in the future use/development of the bridge, planning of traffic on the bridge and assist the owner's in the decision making process for management and investment into the bridge (ie: repair, replace, upgrade).

2.3 Modal Analysis

Experimental and Operational modal analysis is based on structural dynamics principles. These principles are applicable to all structures and are inherent properties of the structure.

2.3.1 Overview of Structural Dynamics

Structural Dynamics refers to the study of the motion of structures and the understanding of the vibration of structures when subjected to a force. The governing factors that are considered in a dynamic system are the mass, its stiffness and damping when an excitation force is applied. These properties are used to describe a system by applying Newton's second law:

$$m\ddot{x} + c\dot{x} + kx = f(t)$$

Equation 1

m = Mass of the Structure

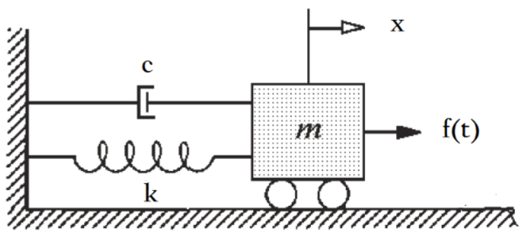
c = Inherent Structural Damping

k = Stiffness of the structure

f(t) = Induced force

When the system is put into motion by a force, the system mass produces an inertia which contributes to the sustaining the motion. The damping represents the natural dissipation of energy of the system and the stiffness enables the storage of potential energy (Chopra, 1995). All real structures have these properties which contributes to its modal parameters.

The natural frequencies of a structure are unique signatures developed by the attributes of the structure and is the frequency at which a structure tends to vibrate without any driving harmonic force. As a structure reaches its natural frequency, the vibration of the structure is amplified significantly. This is known as resonance and can be better described as the sustained extreme oscillated motion of a structure due to its inertial and elastic properties under the ideal harmonic loading conditions.



$$x(t) = \underbrace{Ae^{-\zeta\omega_n t}\sin(\omega_d t + \phi)}_{\text{transient}} + \underbrace{X\cos(\omega t - \theta)}_{\text{steady state}} \qquad X = \frac{f_0}{k}\frac{1}{\sqrt{(1-\beta^2)^2 + (2\zeta\beta)^2}} \quad \text{and} \quad \theta = \tan^{-1}\left(\frac{2\zeta\beta}{1-\beta^2}\right)$$

Figure 4: Single Degree of freedom dynamic system
(Chopra, 1995)

In a simple single degree of freedom system, the natural frequency is governed by the following parameters:

$$w^2 = \frac{k}{m}$$

w = Natural Frequency Equation 2

k = Stiffness of Structure

m = Mass of Structure

If we observe the complete complex differential equation for a displacement on a damped single degree of freedom system, a coefficient is noted that is referred to as the amplification/magnification factor which interprets the amplification of the vibration of this type of structure. It is noted that whilst the driving frequency over the natural frequency (β) increases the amplification of the vibration, the damping properties (ζ) reduce the amplification as can be seen in the plot below (Chopra, 1995) :

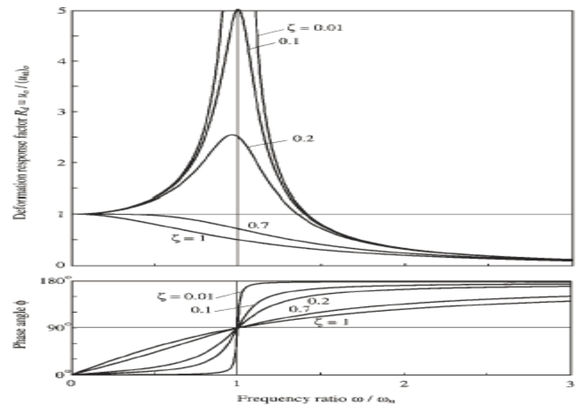


Figure 5: Figure illustrating the reduction effect of damping on the amplitude of the natural frequency of a structure (Chopra, 1995)

When looking at MDOF the problem becomes more complex as there are more DOF's however the principles remain the same. The various properties for each degree of freedom is represented in a matrix form of the equation of motion. The size of the mass, damping and stiffness matrices are based on the number of degrees of freedom. Most real civil structures are MDOF systems thus it is essential to understand the basic principles.

Methods to solving these dynamic problems are split up into 2 categories namely; Modal superposition and Direct Time Integration Methods.

2.3.2 Modes

Modal analysis is an extension of structural dynamics in that the modal parameters can be seen as signatures of the structure. Modes are used to describe the resonance of a structure. The modes are dependent on the structures mass, stiffness, damping and boundary conditions. When a structure reaches a natural frequency the vibration shape will tend to be dominated by its resonant mode shape. The modal parameters are achieved by applying an excitation to the structure and evaluating the deflection shape by measuring the relative positions of different points on the structure at instances in time. The excitation of fundamental or low frequency modes, which are more simplistic than high frequency modes and generally mimicking basic translation, bending and torsion. Higher frequency modes become complex in shape and are difficult in cases to describe (Schwarz & Richardson, 1999).

2.3.3 Frequency Response Functions

The frequency response function is defined as the representation of the relationship between a single input degree of freedom and a single output degree of freedom in a direction on a structure as a function of frequency. The output measures the displacement, velocity or acceleration response of a structure. The FRF is mathematically defined as the fourier transform of the output response divided by the fourier transform of the input force. (Schwarz & Richardson, 1999).

FRF's real and imaginary complex components can be represented graphically in different ways as illustrated below:

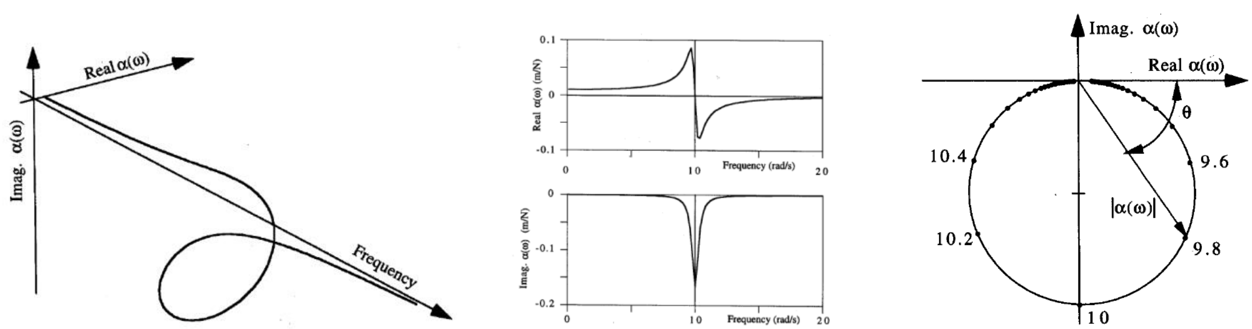


Figure 6: The various graphic representation of FRF data

The FRF is also useful in that the high narrow peaks in the functions identify the aplifiers. These aplifiers at specific natural frequencies are a result of a small input force causing a significantly large response from the structure. An exitation at these frequencies will cause the structure to resonate and has the opposite effect when it approaches an inverted peak. These peaks are demonstrated in the FRF below:

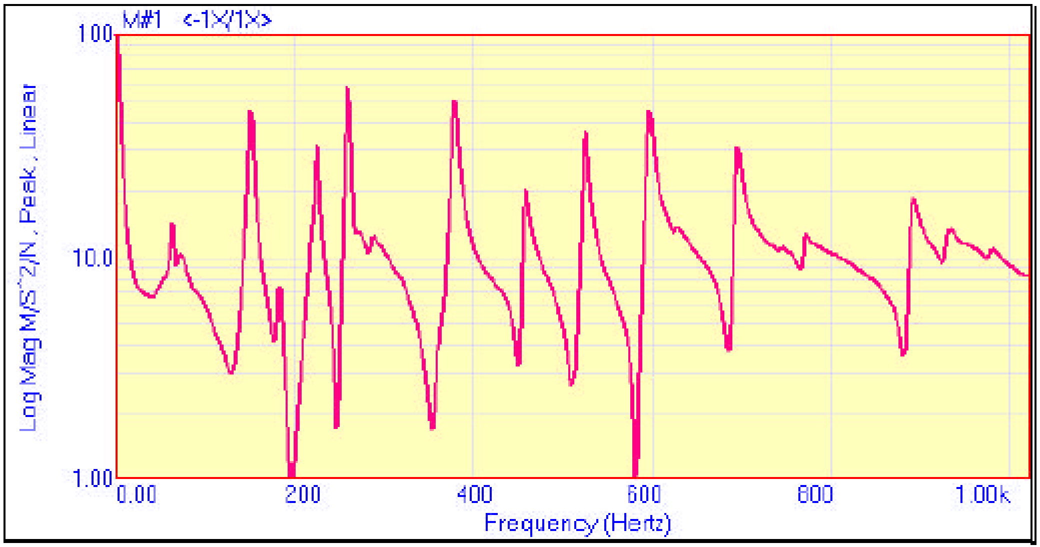


Figure 7: Typical Frequency Response Function

A FRF is typically generated from the outputs of the FRF matrices which represents the relationship between the input reference point and the response point. An FRF can be generated for multiple response points as illustrated from the overall FRF matrix such as the configuration illustrated below:

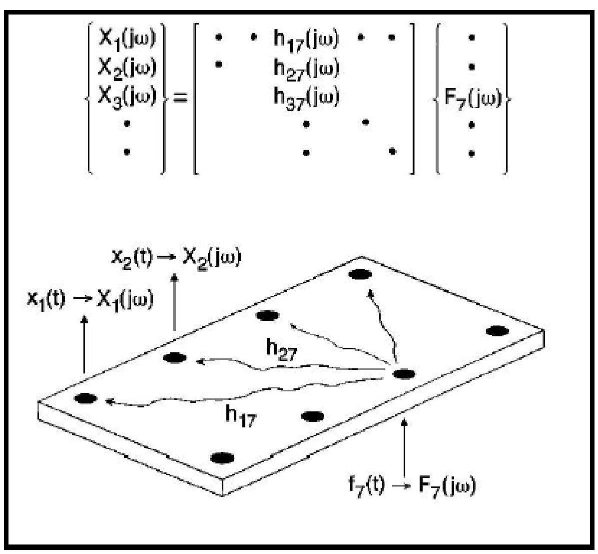


Figure 8: Demonstration of the input output relationship in the development of an FRF

2.3.4 Modal Testing

Modal testing encompasses the measurement and analysis of FRFs. The extraction of modal parameters can be achieved using different methods.

Ambient Vibration Testing or Operational Modal analysis seeks to determine modal parameters by response testing only. This entails collecting vibration data, using response transducers and signal analysers, from a structure under operational conditions. Under operational conditions the excitation has a spectral distribution to which modes are weighted. The measured response are thus the properties of the modes and not that of the excitation. The modal parameters, such as the natural frequencies, mode shapes and damping, achieved by this technique can then be used for system identification and FEM updating.

Forced vibration testing involves the artificial excitation of the structure by an impact or broadband shaker to produce the input force. The output response in most cases are measured using accelerometers as response transducers at specified points on the structure (identifying a sample of the structures DOF's) and using a signal analysers the FRF's are measured using the input and output pairs. The modal testing requires measurement of at least a single row and column of the FRF matrix (Schwarz & Richardson, 1999).

The use of a shaker is a useful method of exciting a structure as it provides a broad range of frequencies, it can be controlled and it provides the requires energy density. It can be used in a SIMO configuration or a MIMO configuration with the use of multiple shakers. A broad range of excitation signals can be programmed using a shaker and FFT analyser which have various advantages. They range from:

- Transient
- True Random
- Pseudo Random
- Burst Random
- Chirp
- Burst Chirp

The methodology for estimating the modal parameters are achieved by curve fitting techniques which link the FRF's to mathematical functions by minimising the squared error. Local single degree of freedom methods focus on estimating one mode at a time where as multiple degree of freedom methods estimate this for two or more modes simultaneously. SDOF methods are used for low modal density FRF's whereas MDOF are used for higher modal density FRF's. The identification of these peaks are known as peak picking. As discussed in 2.3.2, the resonance peaks in the FRF's are a close approximation of the modal frequency and is thus referred to as such. The width of the resonance peaks represents the approximate damping and is calculated using the half power bandwidth method (Schwarz & Richardson, 1999). These methods are simple and can generally be calculated by hand however there are effective software packages available to assist in analysing and calculating these modal parameters.

2.4 Structural Modelling and Loading

In determining the performance of a structure, structural analysis is imperative to understand the forces, stresses or deflections that the structural elements are exposed to and thus determining how it manages the loading. When analysing a complex structure, such as a bridge, it is common practice to use finite element modelling in the design process. However, when trying to understand the real performance of an existing structure, this tool must be used in a different manner

2.4.1 Finite Element Modelling and Updating

Finite element modelling is an exceptional analytical tool however when comparing analytical and experimental results there are bound to be discrepancies due to assumptions in the model or uncertainties due to the real properties or characteristics of the structure. Finite element model updating is thus the systematic approach in improving the model using the experimental results (Brownjohn, et al., 2003).

This is achieved by modifying or tuning the uncertainties as quantitative parameters to minimise the discrepancies. The preparation of a model for updating requires specific consideration which may be different than a model being used for design. An example of this would be the representation of damage or deterioration on a structure. This is generally not a parameter considered in a normal analytical model, however, defects or deterioration in a structural member may cause the element to react differently to stress. Loose bolts or defective welds may affect the fixity of certain connections. These aspects factor into the uncertainties and if their parameters are effectively updated, can assist in tuning the model. An example of parameterising uncertainties could be the boundary conditions of a bridge which in most cases are its supports. It is common practice to model these boundary conditions as springs. In a real structure it is unlikely that a support is fully pinned or fully fixed, thus modelling the support as a 3D spring allows the analyst to update the stiffness of the spring in translation and rotation in the model (Brownjohn, et al., 2001).

An overview of the FEM updating procedure is as follows:

i. The selection of responses that will be used as reference data from the experimental data (ie: modal frequencies)
ii. The selection of parameters (ie: uncertainties) to update on the FEM
iii. Model modification/tuning which is the systematic iterative process of updating the model to minimise its discrepancies in relation to the experimental data.

The response parameters when using EMA for the FEM updating process would generally be the modal parameters extracted from the experimental data. The selection of the updating parameters, however, requires thought and good judgement as it is crucial to ensure a well-conditioned model. The key considerations are the number of parameters and then which parameters should be selected. The sample of parameters selected should be kept small and the selected parameter should produce genuine and logical improvement of the model. The modification of various parameters that are not effective would result in meaningless results. When conducting model tuning by a manual process, a preliminary estimation of the selected parameter values needs to be done in order to ensure that the updated values do not unreasonably vary from the true value. An upper and lower boundary of the parameters should also be ascertained to ensure that the possibility of implausible updated parameter values are negated. This will encourage convergence of the result to an acceptable level (Brownjohn, et al., 2001).

2.4.2 Loading on Rail Bridges

The approach to loading for the design of rail bridges are typically based on loading models developed for the limits of rail traffic within the region. Their rationale is stipulated in the regions accepted codes such as BS 5400, Eurocode 1991-2 or SATS 1983. The basic concepts however are very similar as they are based on fundamental principles. The selected load models do not describe the actual loads however they describe a conservative representation of the loads for design purposes. To accurately ascertain the loads, more in depth study is required to understand the actual trains moving over the bridges, its dynamic effects and the time based progression of loading over the bridges. This is required for a true representation of the condition and performance of a bridge.

Understanding loading models and consideration, as stated previously, is helpful in setting up the actual loads. The Eurocode will be used as a representation of the loading considerations for this review.

The main considerations for a straight steel railway bridge is as follows:

- vertical loads
- dynamic effects
- nosing forces

- traction and braking forces,

2.4.2.1 Vertical Loading

Considering vertical loads there are 5 load models:

1. Load Model 71 represent normal rail traffic on mainline railways
2. Load Model SW/0, which could be relevant for continuous bridges
3. Load Model SW/2 to represent heavy loads
4. Load Model HSLM represent high speed (>200 km/h) passenger trains
5. Load Model unloaded train to represent the effect of an unloaded train.

Load model 4 will not be discussed as the situation under consideration is not a high speed train but a freight train. The application of these loading models must also be considered in order to generate the most unfavourable situations thus the Eurocode must be consulted when applying these scenarios (Sanpaolesi & Croce, 2005).

Figure 9, represents Load Model 71 which is used as the static vertical load configuration for normal rail traffic. It consists of a 4 axle locomotive of 1000kN with a uniformly distributed load of 80kN/m. If the loading of the trains are different the code stipulates a factor, α, which can be multiplied by the loads (Sanpaolesi & Croce, 2005).

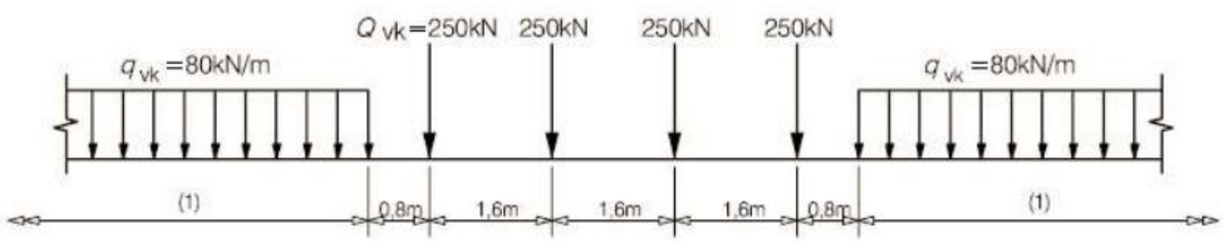
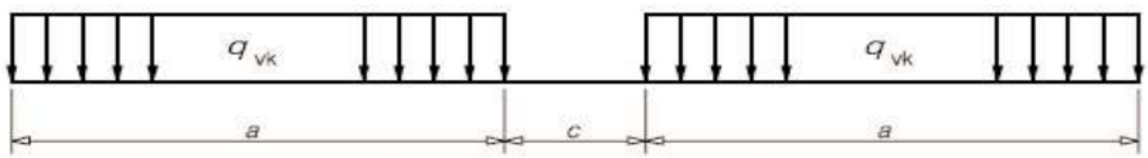


Figure 9: Eurocode Model 71 load configuration for rail bridges

Figure 10, represents load model SW/0 & SW/2 which is used as the static vertical load configuration for bridges constructed as continuous beams and for bridges that is planned to carry heavy rail traffic, respectively. The loading is configured as 2 unlimited stipulated uniformly distributed loads spaced apart by a specified distance c (Sanpaolesi & Croce, 2005).



Load Model	q_{vk} [kN/m]	a [m]	c [m]
SW/0	133	15,0	5,3
SW/2	150	25,0	7,0

Figure 10: *Eurocode SW/0 & SW/2 load configuration for rail bridges*

The unloaded train scenario representing vertical static load of an unloaded train is simply an unlimited uniformly distributed load of 10kN/m.

The distribution of axle loads, excluding the effects of eccentricity are split over 3 adjacent sleepers with 50% of the load on the sleeper under the load and 25% of the load distributed to the 2 adjacent sleepers respectively. The dynamic effects of the trains are taken into account by a Dynamic Magnification factor. This factor represents a conservative estimation of the effects of additional impact produced by the wheels of the train moving over the bridges. It also assumes that the risk of resonance in the structure is negligible (Sanpaolesi & Croce, 2005).

2.4.2.2 Horizontal Loading

Centrifugal forces are an essential consideration when there is a cant in the rail line or if there is a bend. This will not be discussed in this review as the scenario in question pertains to straight bridges and further insight can be gained from the Eurocode. The nosing force are always be combined with a vertical traffic load and are taken as a concentrated force acting horizontally, at the top of the rails, perpendicularly to the centre-line of track. It shall be applied on both straight track and curved track. Traction and braking forces are the longitudinal forces generated from the actions of the train. They are considered as uniformly distributed loads over the corresponding influence length acting at the top of the rail. The forces for design purposes as stipulated by the Eurocode are as follows (Sanpaolesi & Croce, 2005):

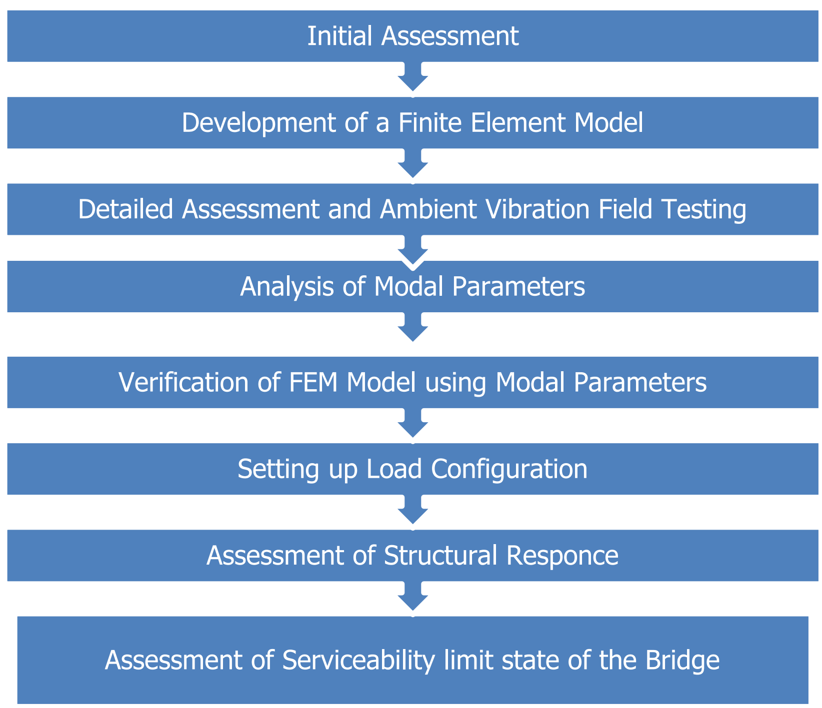
Traction force :	$Q_{lak} = 33$ [kN/m] $L_{a,b}$ [m] $\leq$ 1000[kN] for Load Models 71, SW/0, SW/2, "unloaded train" and HSLM
Braking force :	$Q_{lbk} = 20$ [kN/m] $L_{a,b}$ [m] $\leq$ 6000[kN] for Load Models 71, SW/0 and Load Model HSLM $Q_{lbk} = 35$ [kN/m] $L_{a,b}$ [m] for Load Model SW/2 $Q_{lbk} = 2,5$ [kN/m] $L_{a,b}$ [m] for Load Model "unloaded train"

2.5 Study Overview

Utilising techniques discussed in the literature review, this study will propose a procedure of visual assessment combined with vibration based acquisition of the structures modal parameters for the calibration of a finite element model. The working loads will then be applied to the structure to determine its overall performance under serviceability limits.

3 CONDITION ASSESSMENT METHODOLOGY

The condition assessment methodology will be based on various sources, papers and previous cases assessed in the literature review such as work done by (Rathnayaka & Dissanayake, 2008), (Moyo & Tait, 2010) and (Brownjohn, et al., 2001). Techniques will be combined from visual assessment methods used by the CSIR in their BMS and EMA methods developed and tested for performance assessment. This study has drawn from this framework laid by these sources to develop the following procedure or method for assessment of the Kalbaskraal Rail Bridge.



The initial assessment starts off with a study of the available drawings and records of the bridge. There are detailed previous studies conducted on the bridge which bring significant value to this exercise, thus it was also assessed. This information would provide information on the bridge layout, geometry, structural elements, connections supports and materials. These are all key components in the understanding of the bridge and in the development of the finite element model.

A visual assessment would then be conducted on the structure. The visual assessment would be used to identify signs of defects, deterioration and any variations from the drawings. The method adopted for the visual assessment would be the one developed by the CSIR for the inspection module of the bridge management system developed for South African Authorities. It would classify the 21 main components of the bridge by using the DER system (Nordengen & de Fleuriot, 1998). In order to achieve this a photographic record of all the components of the bridge would be developed and categorised for assessment. Based on this an overall qualitative assessment of the bridge can be achieved and a record of all defects or anomalies can be noted which may be of significance in the development of the finite element model.

The development of the finite element model has been slotted in at this stage as it is believed that it would optimise the modelling process. Setting up the FEM at this stage provides an opportunity for the practical planning of the field testing and the alignment of that exercise with the model updating process. Since the layout, geometry, structural elements and other information are made available from the initial assessment; this is a viable approach in the overall process. The software package used for the development of the model was Sofistik CAE. At this stage the assumed uncertainties would have been modelled such that they would be available for modification at the FEM updating stage (i.e. supports and other boundary conditions identified could be modelled as springs). The assumptions in the FEM will be discussed further under Model Setup.

The next stage would be the field testing in aid of system identification. This involves setting up measuring equipment to extract data from the actual structure to determine the modal parameters. An Operational Modal Analysis using ambient vibration was the technique used for this exercise. This would involve recording the output response of the structure resulting from ambient conditions and processing the signals using a signal analyser to ascertain the frequency response functions. The setup and arrangement of the equipment is discussed further under Modal Analysis.

The time step data recorded in the field testing would then be transformed to frequency response functions and analysed using ME-Scope Software. The Modal parameters would then be acquired using the data analysis using the software. The mode shapes are determined by positioning the degrees of freedom recorded in the field testing on a model structure built in ME-Scope. The natural frequencies would be acquired by a curve fitting process. This is discussed further in the Modal Analysis section. The modal parameters would then be used for calibration of the finite element model.

On acceptance of the FEM, the loading and load combinations were defined. The loading was based on the current and future loads expected on the structure. The predicted loading was sourced from previous studies on the bridge by (Moyo & Tait, 2010). The Sofistik Running loads package was then used to then simulate the load cased a moving load on the structure. This provides a more accurate simulation of the real situation as it loads each subsequent point retaining the reactions from the load in its previous position. The actions such as stresses, deflections, moments and forces assessed based on the output of the FEM

Deduction would be drawn from the structures calculated response and used to determine whether the structural performance is acceptable and meets the required standards for operation.

4 BACKGROUND OF KALBASKRAAL RAIL BRIDGE

4.1 Bridge Description

The bridge under assessment for this study is the Kalbaskraal Bridge located in Malmesbury. The bridge has two 32m spans supported on concrete abutments and extends across the Diep River located just before entering the small town of Kalbaskraal. The bridge is of a complex truss construction and is made up of built up wrought iron sections. Based on previous studies, the bridge was relocated to its current position in 1932 after being in operation since 1906, where it spanned over the Blood River in the Beaufort West region. This study will focus on the eastern span of the bridge in order to correlate with the results attained from previous studies.

Figure 11: Image of the Kalbaskraal Rail Bridge

4.2 Bridge Characteristics

4.2.1 Structural Elements

The bridge configuration consists of two main trusses spanning 32m at a height of 2.58m. The top and bottom chords of the truss are built up channel sections connected by T-section members which form the internal members of the truss. The end posts of the truss are robust built up channel sections reinforced with small criss-cross rectangular plates at the open end of the channel. The trusses support the 4.5m long built up I-section cross girders of variable depth, on its main lower chord member. The cross girders are spaced at 1.065m along the length of the truss. The cross girders support the I section rail beams which in turn carries the timber rail sleepers. The sleepers are spaced at approximately 0.65m intervals along the rail beam and supports the main rails. The bridge also has a bracing system which consists of 6 sets of criss-cross angle sections connected to the bottom channel of the truss. This configuration can be seen in the figure below.

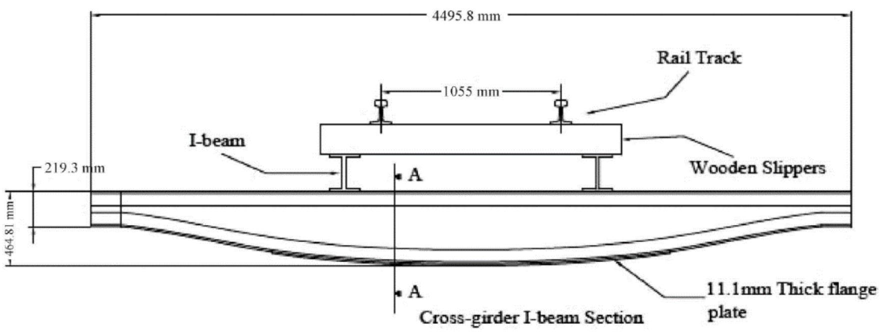


Figure 12: Cross section taken through the cross beam on the Kalbaskraal Bridge

4.2.2 Materials

As stipulated in the background, the bridge is constructed of wrought iron. In an ideal situation, it would be preferred to have the materials tested to ascertain exact properties. Metallographic tests would be used to determine the grading of the steel and a tensile test would be used to determine yield strength, ultimate limit strength and ultimate elongation (Rathnayaka & Dissanayake, 2008). NDT tests would also be used to determine localised damage or overall deterioration. Unfortunately, material testing was not available for this study. It was thus decided that a visual assessment approach would be used to identify localised damage (discussed further in section 4) and assumed material properties were used for analysis. The material properties were adopted from a previous study implemented on the bridge by (Moyo & Tait, 2010). The assumed material properties were as follows:

Table 2: Assumed Material Properties

Material	**Density (kg/m3)**	**Modulus of elasticity (GPa)**
Steel	7700	200
Wrought Iron	7800	210
Timber	800	16

5 INITIAL ASSESSMENT AND FIELD MEASUREMENTS

5.1 Initial Assessment

5.1.1 Assessment of Record and Drawings

When planning and conducting a condition assessment on a structure it is key to have a good initial understanding of the geometry and characteristics of the bridge. It provides qualitative input to assist the judgement of the engineer assessing the structure. The following documents and drawings were reviewed for planning of the condition assessment:

Table 3: Documents and drawing reviewed as part of initial assessment

No.	Document Type	Description	Reference Number
1	Drawing	Cape Government Railways : General Arrangement	B 1/582
2	Drawing	Cape Government Railways: Details of Main Girder	B 3/582
3	Drawing	Locomotive: Diesel Class 35000	N/A
4	Drawing	Wagon	CCR-1
5	Journal Paper	Structural Performance Assessment and Fatigue analysis of railway bridge (Moyo & Tait, 2010)	N/A
6	Thesis	Operational Modal Analysis on the Kalbaskraal Railway Bridge (Naraghi, 2012)	N/A
7	Thesis	Dynamic Loading of the Kalbaskraal Railway Bridge FE Model (Welihockyj, 2013)	N/A

NB: The majority of the drawings listed above were of a poor quality. The legible information was extracted and assessed, and supplemented with the previous works and on-site measurements. The drawing is available in Annexure C.

5.1.2 Visual Assessment

In a visual assessment it is important to extract relevant and tangible information of the defects on the bridge. This is in order to adequately determine and prioritise maintenance on the structure. In the case of this project it also serves as input into the modelling of the structure to ensure that a good representation of the actual structural behaviour is achieved. As a result, a systematic process was selected to ensure that these criteria were met.

The inspection strategy was thus sourced from the STRUMAN BMS strategy developed for South African Authorities and it provides an effective process for the visual assessment of bridge structures. It is currently utilised in various municipalities and provides the backbone for BMS operating in the country.

The inspection procedure guides the bridge inspection by a standardised inspection sheet which has 21 common bridge structural elements listed for assessment. These are elements ranging from the foundations to the beams and columns of the structure. The procedure then employs a rating system called the "DER rating system" which has been discussed in the literature review. The rating system is summarised as follows:

- D represents the degree or severity of the defect
- E is the extent of the defect on the item under consideration
- R is the relevancy of the defect. This rating considers the consequences of the current status of the defect with regard to the serviceability of the bridge and the safety of the user (pedestrian, cyclist, motorist, and passenger).
- U is the urgency to carry out the remedial work to repair the defect. This rating considers possible future events that could adversely affect the defect, and provides a procedure for applying time limits on the repair requirements.

Category	X	U	0	1	2	3	4
Degree/ Severity (D)	N/A	Unable To Inspect	No defect	Minor	Fair	Poor	Severe
Extent (E)				Local	> Local	< General	General
Relevancy (R)				Minimum	Moderate	Major	Critical
Urgency (U)	Make Safe (MS)	Record (R)	Monitor	Routine	< 10 yrs	< 5 yrs	ASAP

Figure 13: DER Rating System

This provides a comprehensive visual understanding of the potential defects that can be used for the maintenance process and can be incorporated into the modelling of the structure.

A visual inspection using the above inspection method was carried out on 5 May 2016 and the detailed inspection sheet has been attached as Annexure A.
The salient outcomes of the inspection are as follows:

- The longitudinal and Transverse members were classified as critical members. Overall they were established to be in a good condition. Localised corrosion was identified to some extent on most members. Loose bolts were identified at the connection between the transverse and truss members. This may affect the stiffness of the structure. There are also missing rivets on specific members. The maintenance walk way on the road side of the bridge is in poor condition as some members are loose. This is quite dangerous and is a safety hazard.
- Specific sleepers have deteriorated and are even hollow in some cases. This is a major safety concern as they support the rail directly
- Abutments were in excellent condition. There were no signs of significant concrete deterioration or permanent settlement. There were however rust and water staining from the steel structure.
- The bearings were in an acceptable condition however showed signs of pitting corrosion. The bearings are required to be monitored.
- The lateral bracing is intact. No sign of major deterioration or corrosion.

This initial visual assessment provided a good qualitative understanding of the possible defects on the structure. If the inspection is conducted for maintenance planning, it would be recommended that a member sheet be developed to catalogue the damage on each member for maintenance planning, which would be the next step.

5.2 Ambient Vibration Field Measurements

Ambient Vibration Testing or Operational Modal analysis seeks to determine modal parameters by response testing only. This entails collecting vibration data, using response transducers and a signal analyser, from a structure under operational conditions. The accelerations are capture with reference to a reference accelerometer which will then be used in the data analysis in the development of the frequency response functions. The FRF data can then be used to determine the modal parameters by curve fitting and assessment of the recorded degrees of freedom.

5.2.1 Procedure

To ensure efficient and effective implementation of the testing, adequate preparation is required. This involves the following:

- Defining the Test objectives
- Defining test methodology and strategy to achieve objectives
- Equipment and Setup
- Planning and Logistics

5.2.1.1 Test Objectives

- The extraction of the time step acceleration data from the Kalbaskraal Railway bridge using ambient vibration testing for system identification

5.2.1.2 Test Methodology and Strategy

The test methodology to be utilised is ambient vibration testing which involves using accelerometers connected to a 16 channel data acquisition system to extract the vibrations, relative to a reference accelerometer, of the bridge excited by the ambient conditions. Each accelerometer measures vibration in a single direction thus the accelerometers will be configured in pairs to record vibration in 2 directions namely the global Z & Y directions. This is to ensure all the critical vertical and transverse modes are not missed.

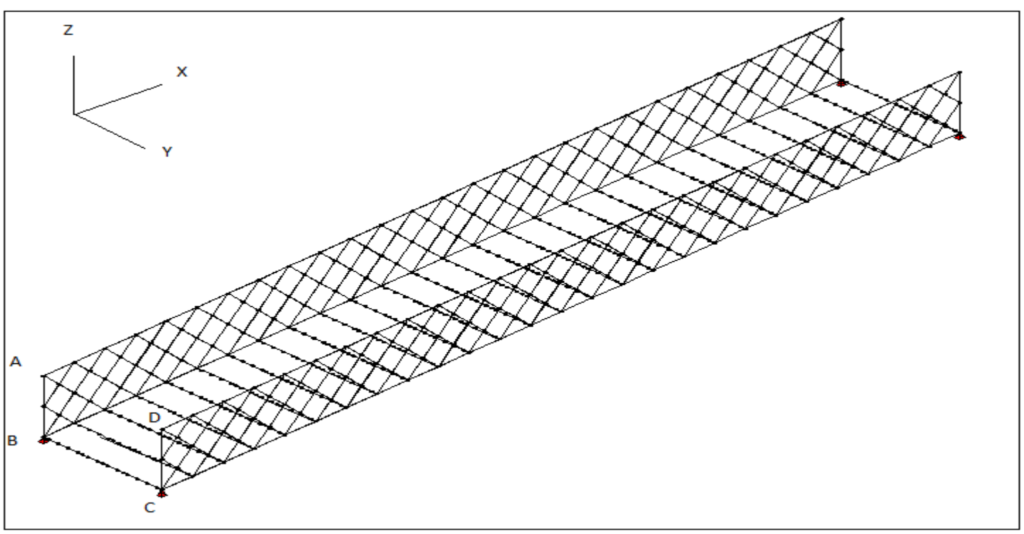


Figure 14: Diagram of Kalbaskraal bridge used for Fieldwork planning

The 2D configuration of the accelerometers will be achieved by mounting them to a customised plate as demonstrated in the image from the actual test below:

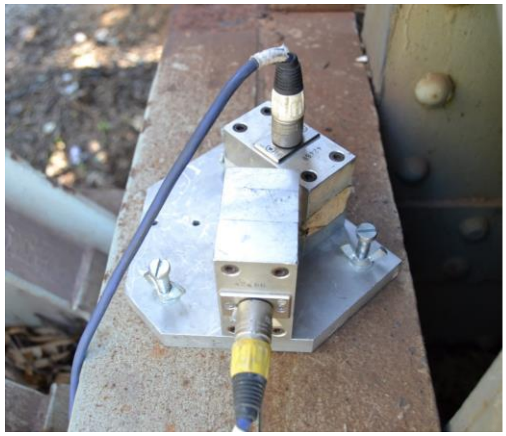
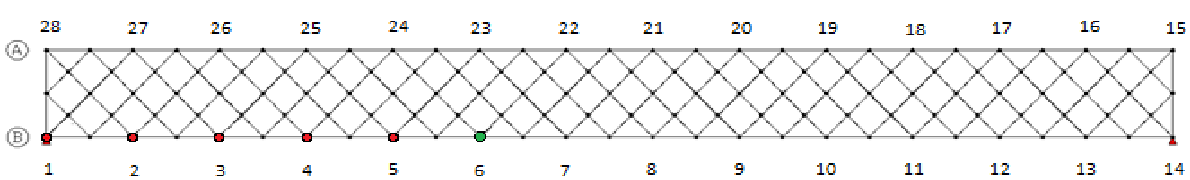


Figure 15: Image of accelerometer pair

The University of Cape Town Civil Engineering laboratory possesses 13 working accelerometers. Two of these accelerometers will be used as the reference accelerometers which will be placed at a distance approximately ¼ of the span away from the support and shall not be moved throughout the testing. As a result, there would be 5 pairs available for measurement at the designated positions. Measurements were taken at specified points on the main trusses as it will provide sufficient information to properly assess the mode shapes in the analysis programme. The testing required 6 setups per truss with the accelerometer pairs being placed at every second connection of the internal truss members to the main truss chords. Each test ran for 15 minutes (ie. 900 seconds) to collect 112 x 8 second spans which would be sufficient for accurate analysis. The setup for the first test run is shown below to illustrate configuration on the actual structure:

🔴	Movable accelerometer Pair
🟢	Reference Accelerometer Pair



Setup AB

The actual placement of the accelerometers on site is illustrated in the image below:

Figure 16: Image of actual positioning of accelerometers on site

5.2.2 Equipment

The equipment required for the testing and its configuration is illustrated in the figure

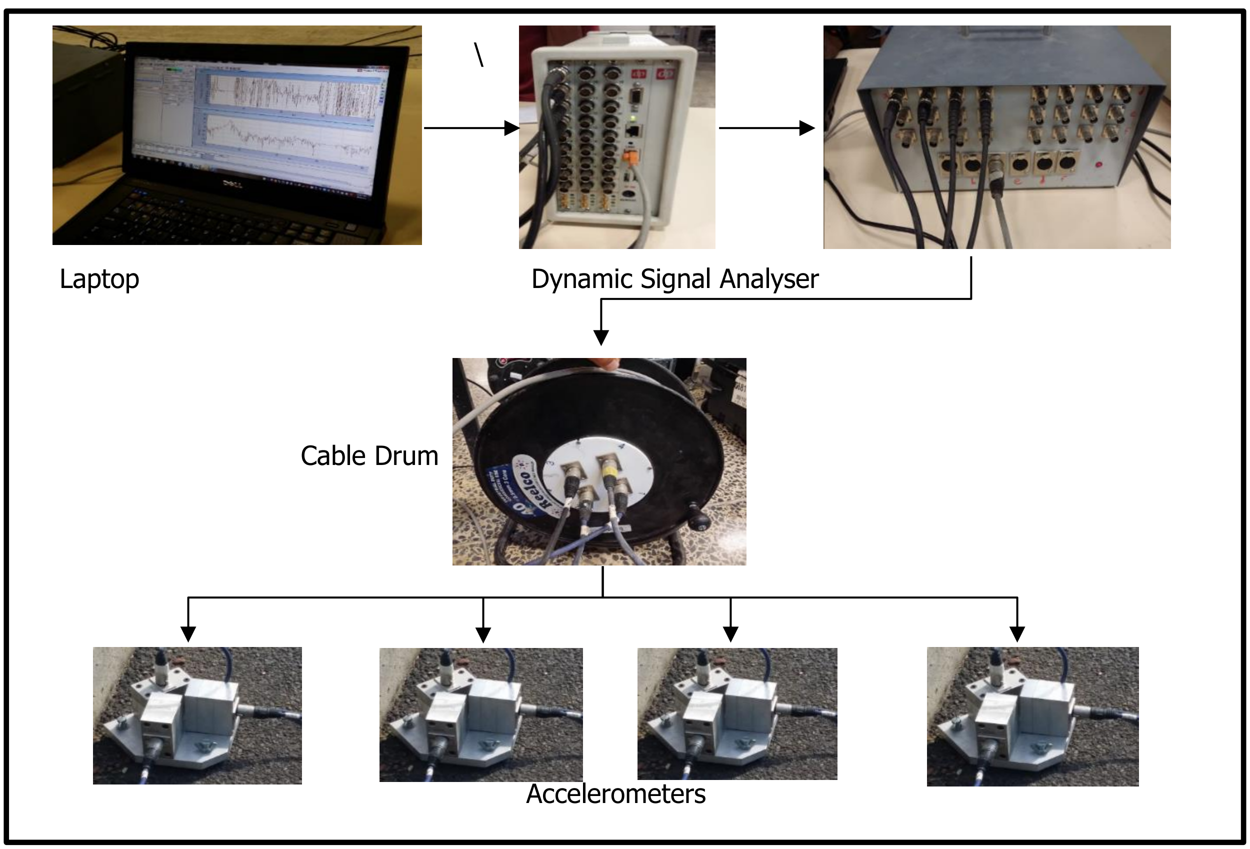


Figure 17: Equipment configuration for OMA

6 MODAL ANALYSIS

The modal parameters are inherent properties of the structure and modal analysis refers to the assessment of these extracted properties in order to use it in some form or another. The modes are dependent on the structure's mass, stiffness, damping and boundary conditions. When a structure reaches a natural frequency the vibration shape will tend to be dominated by its resonant mode shape. In the case of this report the modal parameters was used as calibration parameters for the finite element model. The parameters were acquired using ambient vibration testing which is a methodology which determines modal parameters by response testing only (as explained in Chapter 5.2).

The methodology for estimating the modal parameters are achieved by curve fitting techniques which link the FRF's to mathematical functions by minimising the squared error. As discussed in the literature review, the natural frequencies can be identified by sharp peaks in the overlaid FRFs. The identification of these peaks is known as peak picking. The width of the resonance peaks represents the approximate damping and is calculated using the half power bandwidth method (Schwarz & Richardson, 1999).

Once the natural frequencies are identified, a model is then developed in the modal analysis software where the recorded DOF's (characterised by their own FRFs), are assigned to the allocated points on the model structure. The mode shapes can then be identified for each specific natural frequency. The curve fitting, peak picking, identification of mode shapes and damping can be calculated using effective software packages whereby these techniques are used as the basis for its analysis and calculation of the modal parameters. Further detailed explanation of these techniques can be referenced from (Schwarz & Richardson, 1999)

MEScopeVES is the software that has been selected for the data analysis and modal processing. The software provides the platform for analysing modal shapes and natural frequencies sourced from the collected field data. The MEScopeVES Package allows for the swift transformation of time domain data to ODS FRFs for FRF based curve fitting and allows for multi-reference modal analysis which is critical as there is a limited amount of accelerometers available for field testing.

The drawing module allows features for the construction of a structure model using simple CAD operation tools. Strategic nodes on the structure are then selected for field testing and once the data is collected from the actual structure the nodes are assigned a degree of freedom which allows for the animation of the mode shape. In the field measurements for the Kalbaskraal Bridge it was decided, based on the available equipment, time and data requirements that every second connection on the main trusses will be used as active nodes. The Structure Model was thus constructed to suit this set up thus each truss has 28 nodal points. Each of these nodes were assigned a vertical and horizontal degree of freedom to illustrate the relative position of the node at a point in time. The ME Scope model of the Kalbaskraal bridge is illustrated in the image below:

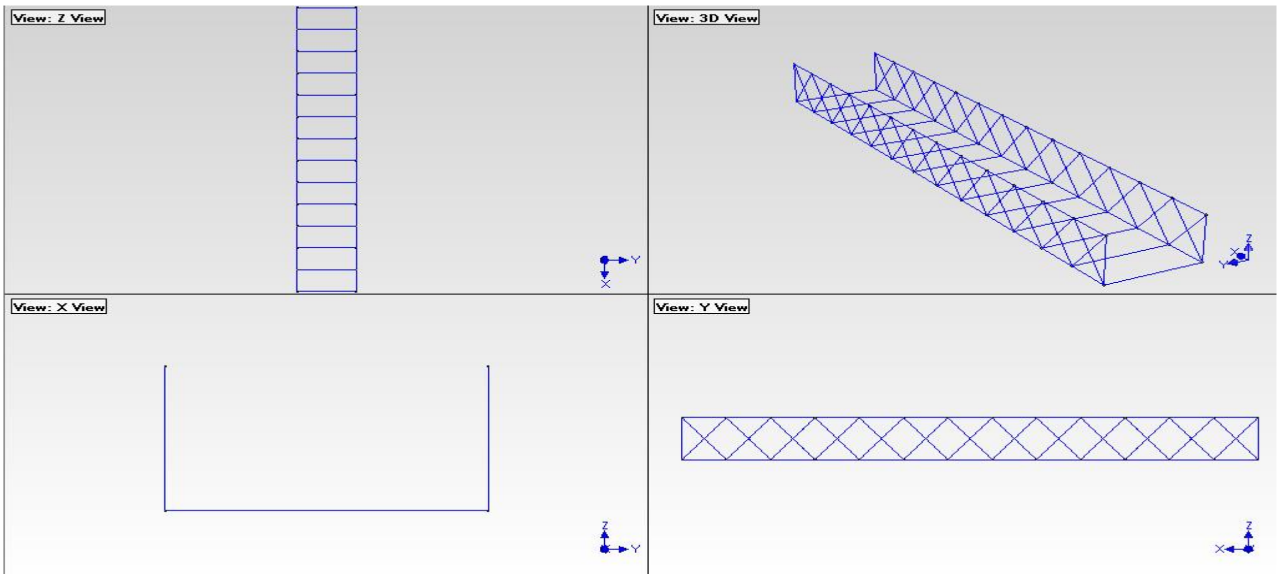


Figure 18: ME Scope Structure Model of the Kalbaskraal Bridge

In the field measurements 112 readings were taken using the accelerometers. This includes both vertical (z) direction and Horizontal (y) directions. Each reading records the accelerations in the time domain (i.e. Trace) for a period of 15 minutes to ensure an accurate representation of the response of the structure. Each data set was then converted to a UFF format which can be imported into ME Scope as data blocks. An image of the imported time domain data for a set of readings is illustrated in the image below:

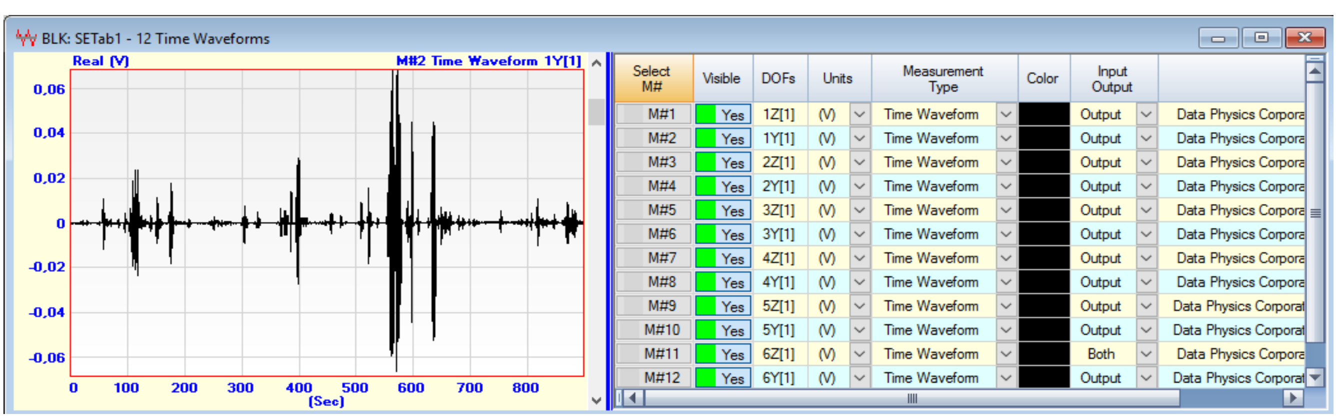


Figure 19: Time Domain data acquired from ambient vibration testing

At this stage, each trace was then assigned its respective degree of freedom. Each trace was also defined as outputs aside from the reference point, which was assigned as both input and output, as this satisfies the requirements of Operational Modal Analysis. MEScopeVES provides the function for the transformation of the time domain data to ODS FRF's for further analysis and curve fitting. The fundamental principles for the transformation of Time domain data to FRF data has been discussed in detail in the literature review and the programme is built on these principles. In all transformations the spectral block size was specified to be 4096 with an overlapping percentage of 66%. Cross and auto spectra data was also included to assist with the scaling process of the different data sets for animation of the mode shapes. An example of a generated ODS FRF is illustrated in the image below:

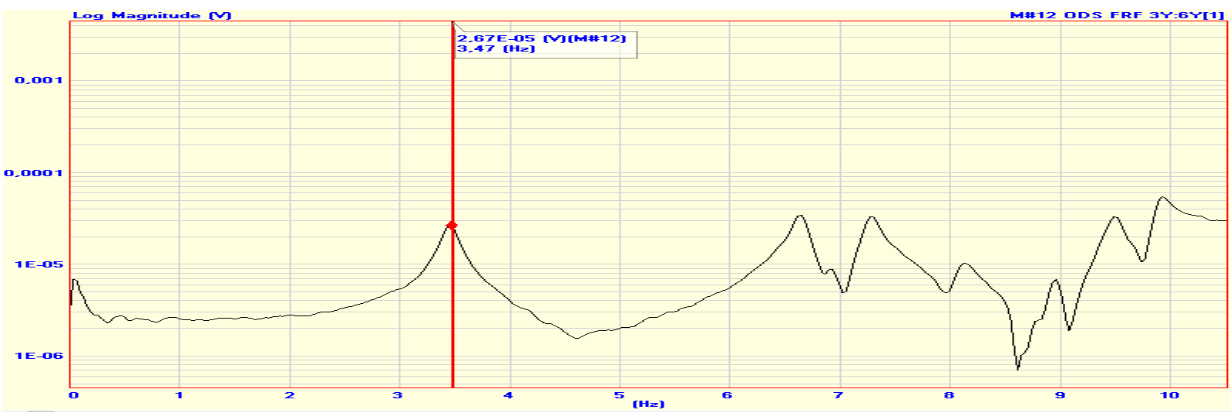


Figure 20: ODS FRF processed in MEScope on the Kalbaskraal Bridge

At this stage, a rudimentary peak picking process, using the line curser, can be done to roughly identify the natural frequencies.

Each FRF is then assigned to their respective nodes on the structure model, via an animation equation. This provides the nodes on the model with their respective degrees of freedom. The deformation of the structure at the frequency peaks is then assessed to determine the dominant mode shapes.

The next step would be curve fitting which is the process of fitting a parametric model to the experimental data for analysis purposes. MEScopeVES provides a variety of methods to ensure effective curve fitting. The process for curve fitting for this project is as follows:

- Identification or counting of peaks in the imaginary domain
- Estimation of Frequencies and damping was achieved using the Complex Exponential method
- Residues were estimated using the Polynomial Method

An illustration of the parametric modal fit to an FRF is illustrated below. It is important to visually ensure that in this process the curve fits the peaks adequately for the accurate identification of natural frequencies and forms well around the existing data to ensure effective estimation of damping. This can be assessed in further detail using Modal Assurance Criteria.

Figure 21: Outcome of Curve Fitting process in MEScope

7 FINITE ELEMENT MODELLING AND UPDATING

7.1 Model Setup

The Finite Element Model (FEM) was developed with the objective of effectively simulating the real life structure and determining its response to the applied working loads. This was achieved by constructing the model and systematically calibrating the selected parameters using the modal parameters measured from the real structure, in the operational modal analysis.

7.1.1 Model Layout

Sofistik CAE was the software used to develop the FEM and an image of the model layout is illustrated below:

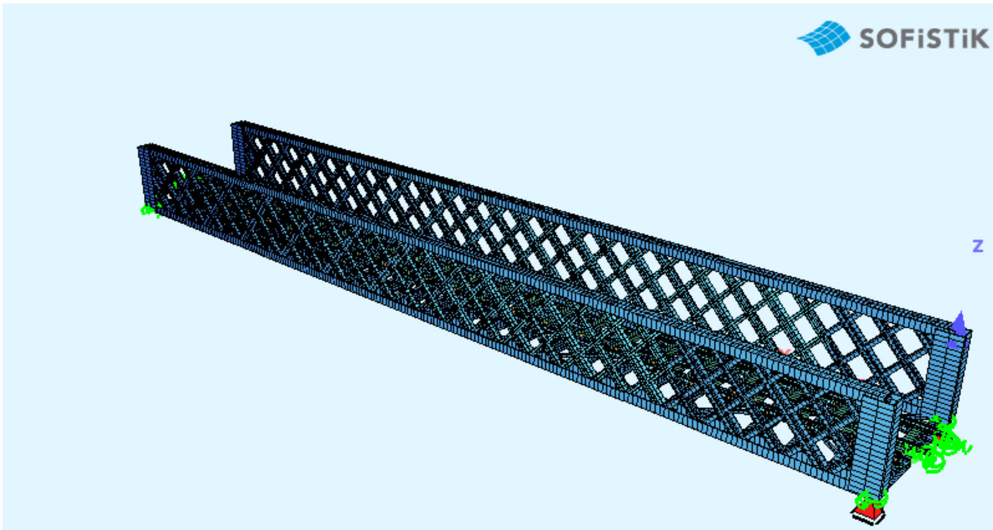


Figure 22: Image of Sofisik CAE FEM of the Kalbaskraal Bridge

The model was constructed based on the structure's physical geometry, on site measurements, the visual assessment, as built drawings and outcomes from previous studies. The trusses consist of beam elements which are rigidly connected. The rail way line is modelled as beam elements supported sleepers which in effect are also beam elements. The load path is as follows:

- Train Wheel Loads to Rail
- Rail to Sleeper
- Sleeper to Rail Beam
- Rail Beal to Cross Girder
- Cross Girder to Truss
- Truss to supports

7.1.2 Material and Sectional Parameters

The bridge is constructed of built up wrought iron sections and as previously stated, material testing was not available for this study. As a result, the assumed material properties were taken as follows for the purpose of modelling:

Table 4: Assumed Material Properties for FEM

Material	Density (kg/m3)	Modulus of elasticity (GPa)
Wrought Iron	7800	210
Timber	800	16

(Moyo & Tait, 2010)

The actual built up members on the bridge structure make the modelling process quite complex as they do not conform to the standard cross sections in the modelling programme. The modelling process is also tedious as some of the member cross sections vary along their length. To simplify the modelling process 'equivalent sections' were utilised. Equivalent sections are solid member cross sections with equivalent structural properties to the built up members. The equivalent sections used for this model was developed in previous studies on the Kalbaskraal bridge by (van Biljon, 2005). The equivalent cross sections used for the model can be referenced in Annexure B.

7.1.3 Connections and Supports

The connection of all beam elements aside from the cross girder to the lower truss chord were considered as rigid. In the study conducted by (Moyo & Tait, 2010), it was identified that the connections between the cross girder and lower truss chord were connected by bolted angles and a portion of the bolts were loose. This was confirmed by the visual assessment conducted for this study and would imply that the connections are not rigid. The same assumption was then made for this study, as in the (Moyo & Tait, 2010) study, to release approximately 50% of the connection in rotation to account for loose connections.

Another connection to consider is the effective continuity of the rail across each end of the bridge. The rail spans across the end of the bridge from a sleeper on the rail beam on the bridge, to a sleeper on the stone ballest. This is illustrated in the image below:

Figure 23: Image of rail line at the approach abutment

In the model, the rail started at the sleeper supported on the stone ballest. An observation was made on site when a train passed over the bridge and it could be seen that the sleeper deflects in the z direction. Deflection measurement equipment was not available for this study so the stiffness was estimated. As it is very difficult to measure the rail stiffness directly, a good calculated estimate was implemented. Intuitively it could also be considered that the rail at this point carried axial and rotational stiffness. The axial stiffness and rotational stiffness about the local x-x and y-y axis was estimated and the remaining translations and torsional stiffness was considered as infinitely stiff, for the simplification of the model. The stiffness of the rail across the end of the bridge was estimated as follows:

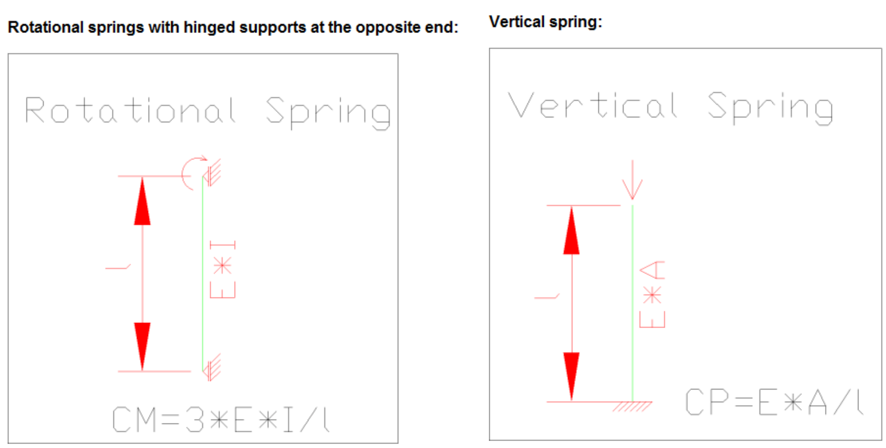


Figure 24: Sofistik CAE guides for calculation of spring stiffness

Table 5: Calculated spring stiffness of the rail line at the end of the bridge to ensure continuity

Stiffness Orientation	**I (mm4)**	**A(mm2)**	**E (Mpa)**	**Flexural Rigidity (Nmm2)**	**Axial Rigidity (N)**	**L (mm)**	**Stiffness Formula (End Considered Pinned)**	**Stiffness**
Local x-x Axis	1,84E+07	6485	2,10E+05	3,87E+12		650	3EI/L	1,79E+10
Local y-y Asis	3,21E+06	6485	2,10E+05	6,73E+11		650	3EI/L	3,11E+09
Axial		6485	2,10E+05		1,36E+09	650	EA/L	2,10E+06

The bridge supports would be modelled as springs as they would form the parameter for updating. This is discussed further in 7.2. Additionally, data from the study completed by (Welihockyj, 2013) referenced a deflection measurement at the central abutment. Deflection readings were taken at the central abutment when a train passed over the bridge and it was found that the abutment settled by 0.4mm. This was due to the settlement of the abutment on the founding river bed soil. Based on these results, an equivalent soil stiffness was calculated to account for this at the bridge support which would provide a spring stiffness at the bridge support in the z direction. The spring stiffness's at the bridge supports at the abutment was calculated to be 404500kN/m which would be divided equally between to 2 bridge supports. The spring support in the z direction at the approach abutment was considered to be fully fixed in translation.

7.1.4 Beam Elements and Meshing

A beam element is defined as 2 nodes and a straight connection. The connection is assigned attributes such as material properties and cross sectional properties. FE partitioning of the geometry is necessary for the purposes of computational modelling with each beam being partitioned approximately the same length as the cross section. Timoshenko beam theory was the general the basis for the elemental behaviour with transverse shear strain remaining constant through the cross section.

The general philosophy around the structuring a mesh is that the mesh should fine enough to obtain the most accurate results. The finer the mesh the more accurate the results. Contrary to this, the finer the mesh the longer the computing time. When defining mesh parameters, one must consider the required accuracy as the real structure is not idealised. There are various construction and on site factors which will skew results as the real structure react differently to the model due to uncertainties. An understanding of the resultant sensitivity and the point at which increasing mesh density becomes less valuable on the end result is essential.

The Sofistik 3D's Mesh generator was developed by the University of Linz and is used to automatically generate an optimised mesh. Two options area were available for the 3D-mesh generation. A structured mapped mesh generation based on the surfaces generated before, or an unstructured mesh generation for pure Tetrahedron meshes. As the quality of Tetrahedron is significantly less than that of Hexahedron, a denser element mesh would be required. Further technical detail on the mesh generator can be found in (SOFiSTiK AG, 2014).

7.2 Model Calibration

The selection of the parameters on the finite element model are based on inherent uncertainties on the model structure, as discussed in the literature review. The selected parameters for calibration on the Kalbaskraal Bridge model will be the 4 bridge supports. The uncertainty lies in the fact that the model assumes that the supports are either fully fixed, pinned and/or free in translation when in reality, all these parameters are actually partially fixed. This allows the modeller the freedom to adjust the rotational and translational stiffness if the supports are modelled as springs.

The updating process involved the adjustment of the spring support parameters in order to correlate the model's modal parameters with the real structure's modal parameters acquired from the Operational Modal Analysis. The adjustment of the stiffness's of the spring parameters would be implemented with the assumption that the model's elemental masses are accurate as they were extracted from the real structure or the equivalent sections developed from the real structure. It is a key principle that there is a proportionality between the square of the bridge's natural frequency and its inherent stiffness. As a bridge support parameter is stiffened, the affected principle mode's natural frequency would increase. An example of this principle on the Kalbaskraal rail bridge can be illustrated on its first mode which is a sway mode. If the rotational stiffness is increased at the supports about the z axis, the natural frequency for the first sway mode is increased, as it is one of the main parameters which affects lateral movement. It is thus clear that in the manual calibration of the FEM, a good understanding of the bridge response and engineering judgement is required by the modeller.

The following table identifies the fixed and adjustable parameter for calibration:

Table 6: Fixed and Adjustable Parameters for FEM Calibration

Parameter	Fixed or adjustable	Reason
Material Properties	Fixed	Assumed
Element Geometry	Fixed	Based on Measured or calculated equivalent sections
Fixity of Connections on Truss	Fixed	Assumed
Fixity of Connections at Cross Beam	Fixed	50% of connections released as recommended in the (Moyo & Tait, 2010) study
Translational fixity at Supports	Adjustable	Uncertain Parameter
Rotational Fixity at Supports	Adjustable	Uncertain Parameter

7.3 Loading

7.3.1 Approach

The locomotive and wagons using the route through Kalbaskraal carries various commodities. This study will focus on the coal wagon loading configuration. The loads will be set up based on the available wagon information. The load configuration consists of a Locomotive pulling a set of coal wagons which imparts a working axle load of 16.5 tons onto the rail. Based on the study conducted by (van Biljon, 2005), Transnet is looking at the feasibility of increasing the loading on the line to 22.5 ton per axle. The vertical loads on the structure are considered to be the most significant loads thus the axle loads will be modelled as point loads and divided between the 2 rails as follows:

Wheel Loads (N) = (Axle Loads[t] x 1000kg/ton x 9.8m/s^2) /2

The load cases were established in Sofistik CAE, and will consider 2 scenarios namely static and running loads. The static loads will be used to assess the response of the bridge under static conditions where the worst load case will be assessed. The running load type will be established to better simulate the real world scenario. Sofistik CAE running loads package simulates the different positions of the loads while retaining the deflection & stresses from the previous load moving across a point. The 2 scenarios will be compared in the assessment. The equivalent axle loads and its position on the wagons used for the model load cases are indicated in the figure below:

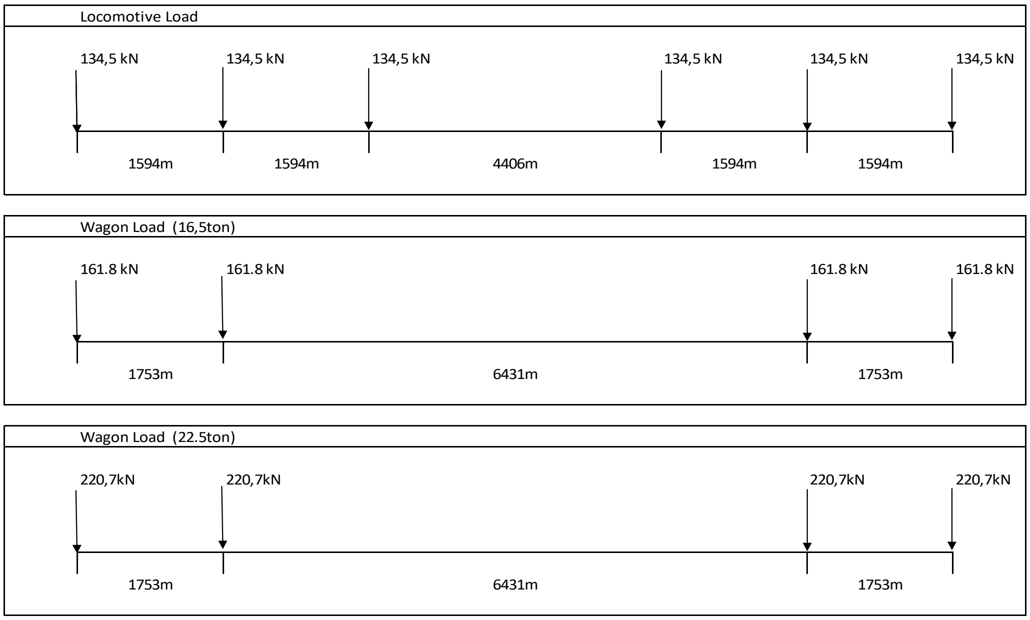


Figure 25: Equivalent Point loads for Locomotive, 16.5 ton/axle wagon and 22.5 ton/axle wagon

7.3.1 Static Load cases

The static load cases will be as follows:

- Load case 1: Locomotive at the start of the Bridge
- Load case 2: Locomotive at the mid-span of the Bridge pulling 16.5 ton/axle wagon
- Load case 3: Locomotive at the end of the Bridge pulling 16.5 ton/axle wagon
- Load case 5: Locomotive at the mid-span of the Bridge pulling 22.5 ton/axle wagon
- Load case 6: Locomotive at the end of the Bridge pulling 22.5 ton/axle wagon

7.3.2 Running Load

The running loads will be setup in the configuration of the locomotive and the wagons, the loads moving in 1m intervals. Sofistik sets each of the positions up as load cases which follow each other in the simulation. The images below illustrate of the locomotive and first wagon running load at different positions while moving across the bridge.

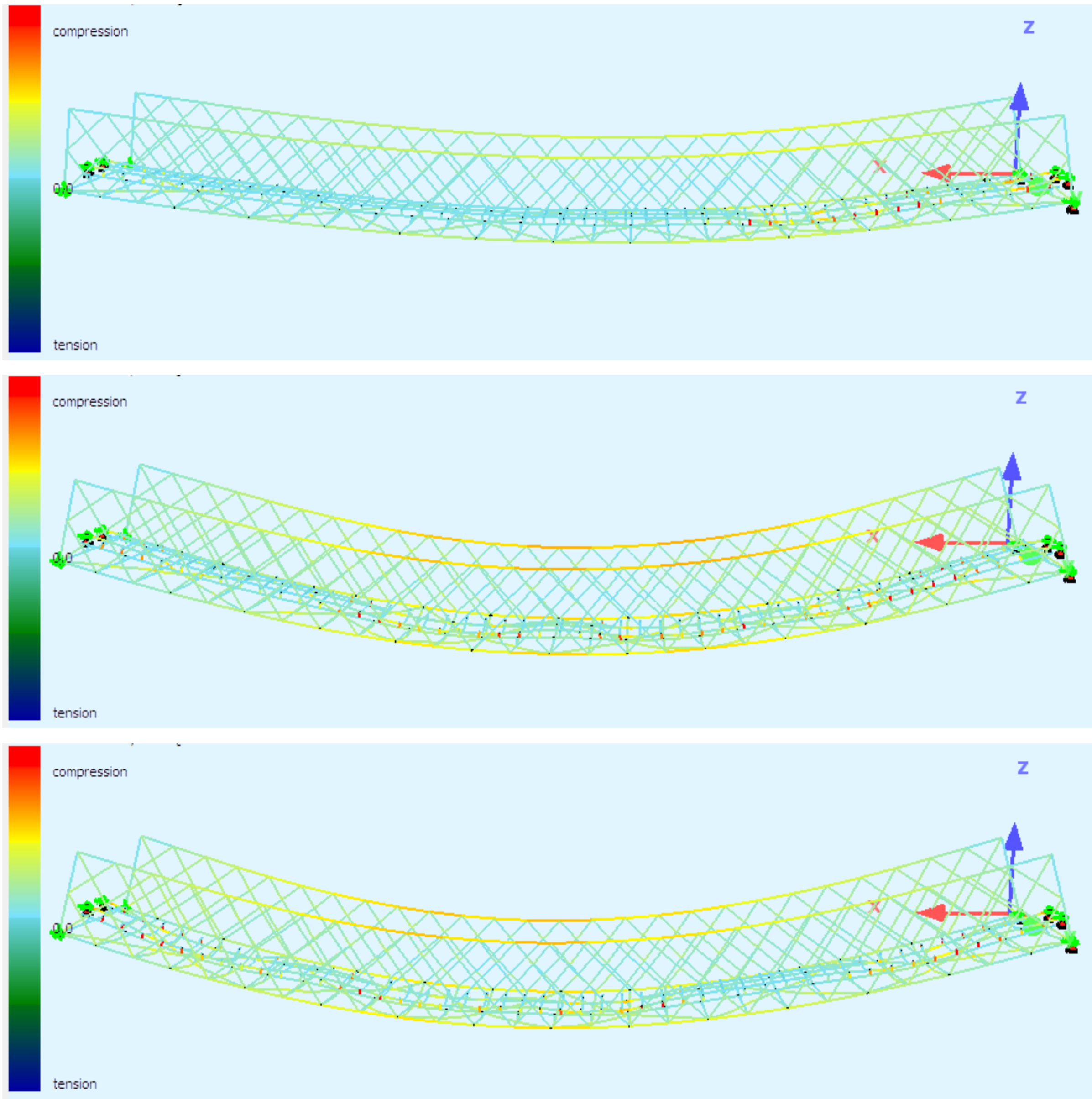


Figure 26: Demonstration of structural response to running loads in Sofistik CAE

8 ASSESSMENT OF RESULTS AND DISCUSSION

8.1 Operational Modal Analysis Outcomes

8.1.1 Frequency Response Functions

The FRF's were developed based on the time domain acceleration data acquired from the field measurements. The time domain data were collected in the following sets:

- Set AB1: 1 – 6 (5 Nodes)
- Set AB2: 7 – 11 (5 Nodes)
- Set AB3: 12 – 14 (3 Nodes)
- Set AB4: 15 – 19 (5 Nodes)
- Set AB5: 20 – 24 (5 Nodes)
- Set AB6: 25 – 28 (4 Nodes)
- Set CD1: 29 – 33 (5 Nodes)
- Set CD2: 34 – 38 (5 Nodes)
- Set CD3: 39 – 42 (4 Nodes)
- Set CD4: 43 – 46 (4 Nodes)
- Set CD5: 47 – 51 (5 Nodes)
- Set CD6: 52 – 56 (5 Nodes)

They were processed in MEScopeVES and 112 FRFs were overlaid. The FRF's were utilised for identification and analysis of natural frequencies which are a result of a small input force causing a significantly large response from the structure causing resonance.

It was noted during the assessment of the processed data that some FRF's were not behaving as expected. This could be as a result of a condition change on the day of testing, errors on site, loose structural elements, faulty equipment or movement of the reference accelerometer. Once animated, the movement of the nodes in the MEScopeVES structure model which the FRF's were assigned to were assessed. It was noted that the response from these specific nodes cause complex low frequency mode shapes which could not be interpreted.

On further study of the site setup, it was concluded that the reason for the unexpected behaviour of the FRF's and inconclusive mode shapes were caused by the positioning of the accelerometers. In the field testing the accelerometers were placed on the top edge of the cross beams at their connection to the bottom truss chord. As per the site inspection some of these cross beam connections had loose connections which could potentially skew the results. Additionally, the reference accelerometer was placed at one of these point which would explain the inconsistent and complex mode shapes.

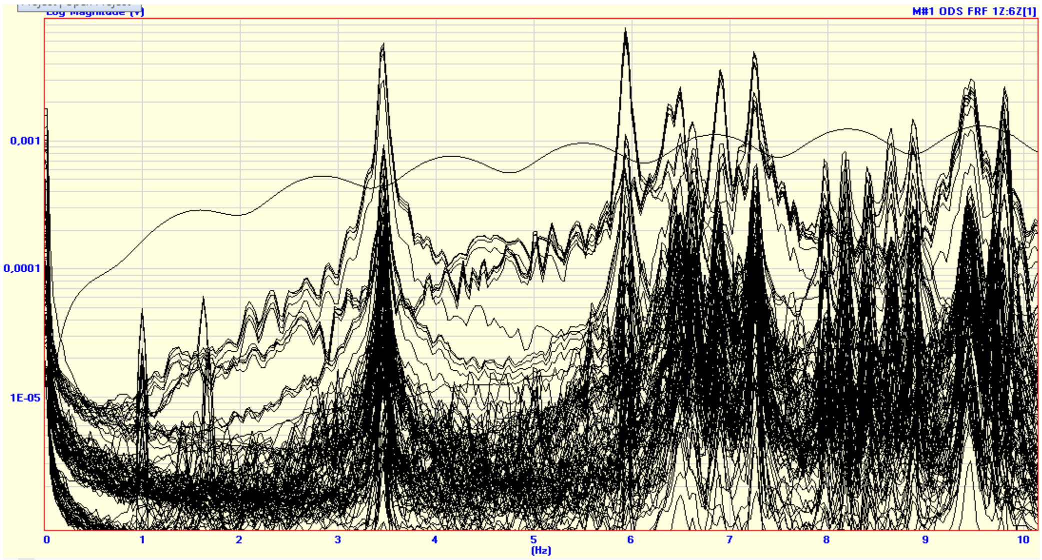


Figure 27: Original Overlaid FRF's from the ambient vibration test on the Kalbaskraal bridge

The overriding fact on examination of the animated mode shapes was that the first 5 mode shapes were unidentifiable and were complex shapes. This did not conform to the fundamental theory that the first mode shapes are generally simple in shape and generally become more complex at higher frequencies. The results of the first field test were thus ruled as inconclusive and it was decided that the field testing had to be redone. A second full field test was conducted and the results prove to be more promising. The measurements from the second field test was thus used to complete the study.

The data acquired from the second test produced much more acceptable data based on the clean FRF's produced. The mode shapes at the structures natural frequency peaks were identifiable and acceptable which were essential for system identification. A group of FRF's for specific degrees of freedom were not accepted and were discarded from the data. The remaining data blocks were used to interpolate the action at the undefined nodes. The accepted overlaid FRFs in figure 28 below are the magnitude representation of the field data for frequencies between 0 and 10Hz.

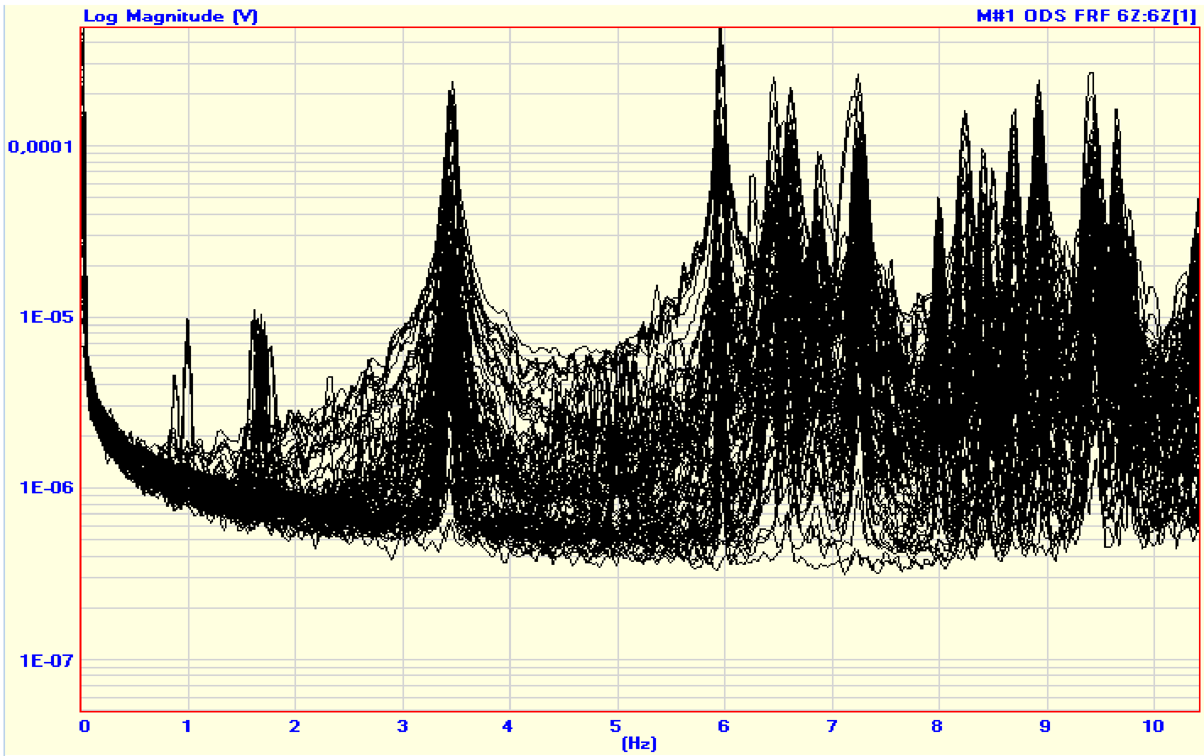
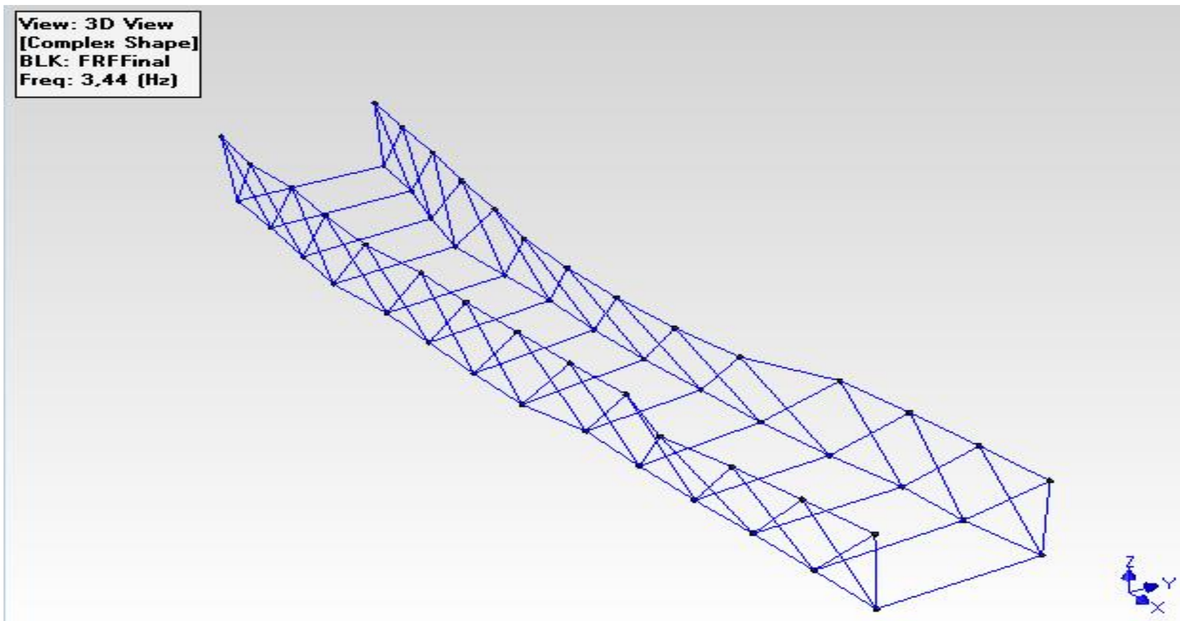


Figure 28: Overlaid FRF's from the ambient vibration test on the Kalbaskraal bridge

8.1.2 Natural Frequencies & Mode Shapes

The natural frequencies were acquired by automated peak picking. At these points the dominant mode shapes can be identified. In the analysis of the natural frequencies, the peaks were clear and easily identifiable. The frequencies acquired from the FRF data correlated well with the previous studies thus providing confidence in the quality of the clean data. A summary of the frequencies and comparison of the results to the previous studies are indicated in table 7.

In the analysis it is expected that the generated shapes would not be as clean as a theoretical model, however there should be a general pattern for the low frequency mode shapes. In the assessment of the mode shapes, the first five dominant mode shapes were identified. The first mode shape was identified at 3.44Hz and was the sway orientated mode with the bridge deforming laterally. The subsequent mode shapes were the vertical mode shape at 6.47Hz, a vertical mode shape at 6.88Hz torsional mode shape at 7.25Hz and the 2nd sway mode shape at 8.00Hz. The figure below illustrates the 1st sway mode shapes and the images of the remaining mode shape can be found in Annexure E:



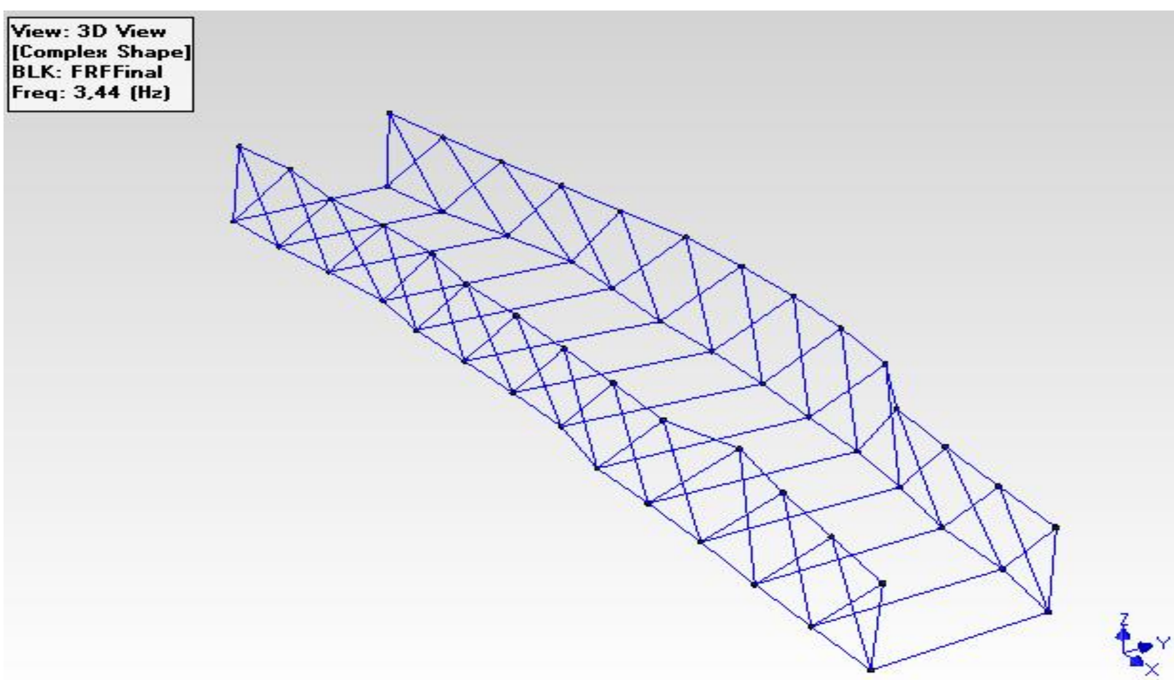


Figure 29: Image of the 1st sway mode shape at 3.44Hz for the Kalbaskrall rail bridge

A summary of the Mode shape descriptions, natural frequencies and comparison to previous study results can be found in table 7 below:

Table 7: Summary of Modal Parameters and Comparison with previous studies

Mode Reference	**Mode Shape Description**	**Natural Freq. (Hz)**				
		Current Study	**Welihockyj** (Welihockyj, 2013)	**Naraghi** (Naraghi, 2012)	**Moyo & Tait** (Moyo & Tait, 2010)	**Van Biljon** (van Biljon, 2005)
1	1st Lateral Sway	3,44	3,45	3,45	3,47	3,48
2	Complex	5,97		5,98		5,98
3	Vertical Bending	6,47	6,47	6,47	6,60	6,47
4	Vertical Bending	6.88		6.96	6.84	
5	1st Torsional	7.25	7,24	7,23	7,26	7,26
6	2nd Lateral Sway	8.00	7,33	8,00	9.42	7,94

The damping results acquired from the analysis is indicated in the figure below:

Select Shape	Frequency (or Time)	Damping	Units	Damping (%)
1	3,45	0,0331	(Hz)	0,959
2	5,97	0,0164	(Hz)	0,274
3	6,89	0,0333	(Hz)	0,483
4	7,26	0,0263	(Hz)	0,362
5	7,99	0,0306	(Hz)	0,383

Figure 30: MEScopeVES Structural Damping Results

8.1.3 Observations based on Mode Shapes

On assessment of the mode shapes it was found that the response from the bridge's two trusses were not exactly the same as one would expect. The response from the truss on the road side was significantly less than the truss located on the northern side of the bridge. This is evident on examination of the animated mode shapes generated in MEScopeVES. The this is demonstrated in the sway mode below:

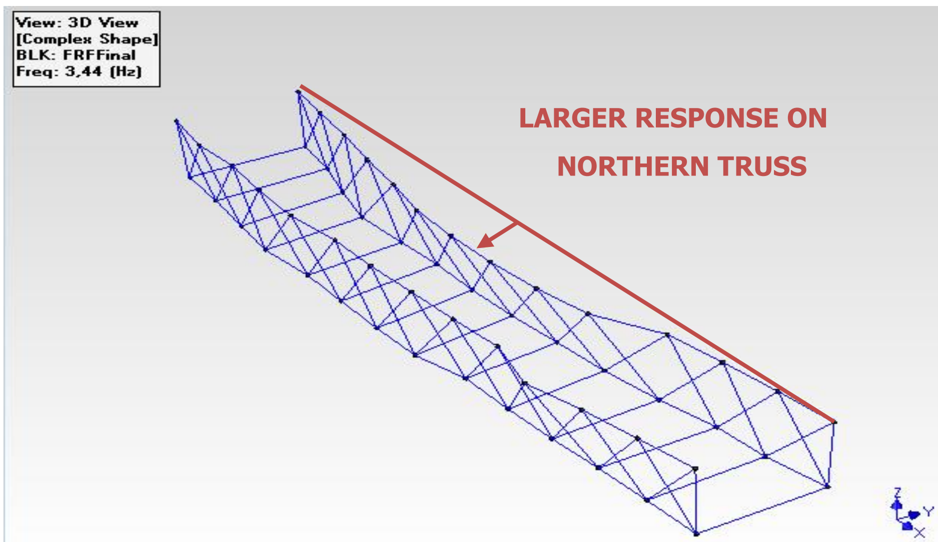


Figure 31: Comparison of the response of the Trusses in the Sway Mode Shape (3.45Hz)

The figure above demonstrates the larger lateral response from the northern truss in relation to the southern truss. This was true for all modes shapes. This would imply that the southern truss has a larger inherent stiffness of the northern truss. This was an anomaly as the trusses are of identical construction and materials. The stiffness variation could be attributed to a number of factors which would have to be examined further. One explanation could be that there the walkway on the eastern side of the bridge provides additional stiffness to the truss. The walkway consists of number of plates welded to the cross beams right next to the truss. Another reason could be attributed to the rigidity of the connections or support conditions. It is possible that the rigidity of all 4 bridge supports could vary which would contribute to the stiffness of the trusses. Additionally, as noted in the initial assessment, the cross beam connection bolts were identified to be loose and variation in the rigidity of these connections could affect the stiffness of the truss. Understanding these factors would contribute to reducing the uncertainties in the model thus these possibilities must be considered.

8.2 Finite Element Modelling Outcomes

8.2.1 Model Calibration

The adjusted support parameters in order to correlate the model's modal parameters with the real structure's modal parameters are summarised in table 8 below:

Table 8: Adjusted Spring stiffness of supports for FEM calibration

Bridge Stiffness				
Central Abutment Support	**Translational Stiffness**		**Rotational Stiffness**	
Global z	2,02E+08 (Imposed Constraint)	N/m	4,00E+07	N.m/rad
Global y	2,00E+08	N/m	3,00E+07	N.m/rad
Global x	5,00E+07	N/m	5,00E+07	N.m/rad
Approach Abutment Support	**Translational Stiffness**		**Rotational Stiffness**	
Global z	Infinity	N/m	4,00E+07	N.m/rad
Global y	Infinity	N/m	3,00E+07	N.m/rad
Global x	5,00E+07	N/m	5,00E+07	N.m/rad

The process for calibrating the model requires an acceptable correlation between the measured modal parameters and FEM modal parameters. The comparison between the measured modal frequencies and FEM modal frequencies are summarised in table 9.

Table 9: Comparison between measured and theoretical modal frequencies

Mode Reference	**Mode Shape Description**	**Measured Frequency**	**Theoretical Frequency**	**% Deviation**
1	1st Lateral Sway	3,44	3,48	0,9%
2	Complex	5,97		
3	Vertical Bending	6,47	6,22	4.0%
4	Vertical Bending	6.88		
5	1st Torsional	7.25	7,81	7.8%
6	2nd Lateral Sway	8.00	7,92	1.0%

The FEM mode shapes correlated with the identified measured mode shapes assessed in MEScopeVES. The FEM mode shapes are available in Annexure E.

An important comparison was also made between the Original FEM's modal parameters and the Calibrated FEM's modal parameters to identify the effect of the change on the structure's modal parameters. Table 10 illustrates the effects of the adjustment structure's physical parameters on its modal parameters. Hidden modes not picked up in the FEM but picked up by the measured data were excluded in the comparison.

Table 10: Comparison between Original FEM's modal parameters and the Calibrated FEM's modal parameters

	Original FEM		**Calibrated FEM**	
Mode Reference	**Mode Shape Description**	**Natural Frequency (Hz)**	**Mode Shape Description**	**Natural Frequency (Hz)**
1	1st Lateral Sway	3.65	1st Lateral Sway	3,48
3	Vertical Bending	6.80	Vertical Bending	6,22
5	2nd Lateral Sway	7.87	1st Torsional	7.81
6	1st Torsional	9.25	2nd Lateral Sway	7.92

The parameters on the original FEM were all considered "fixed" which implied that these support conditions translated all moments and forces to its supporting structure. As these parameters were tweaked to form partially fixed supports, theoretically the overall stiffness of the structure would be reduced thus lowering the structures natural frequencies. On observation of the results in table 10, this correlation can be seen. Additionally, it was also noted that the mode shapes for the mode 5 & 6 in the original FEM switched positions in the calibrated FEM. This may be attributed to the fact that the torsional motion is a combination of vertical and horizontal movement, and the fact that the modes are identified as being close together along the frequency spectrum based on the overlaid FRF's produced from the measured data.

8.2.2 Damping

Sofistik CAE uses Rayleigh Damping in the calculation of damping for dynamic analysis. The outcome of the damping calculation is illustrated in Figure 32.

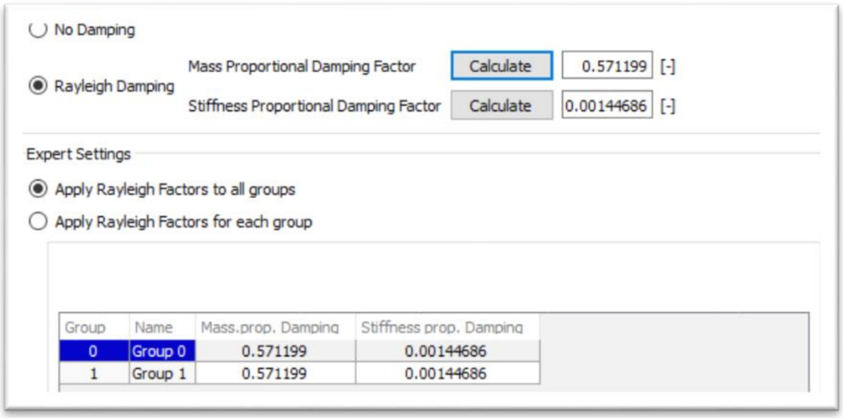


Figure 32: Sofistik CAE Rayleigh Damping Factors

8.2.3 Structural Response to Loading

The actual response of the structure is a major component in assessing its performance. For assurance that the structural performance is acceptable, it must conform to the serviceability limit state requirements. In this case the serviceability limit state, as stipulated in the Eurocode is to ensure that deflection is controlled. The maximum allowable deflection of a bridge of this kind is limited to its span/1000. As the bridge span is 32m the allowable deflection is 32mm. For the purposes of this study, the deflection criterion was assessed, however the FEM can be used for a full analysis of the bridge. The deflection response of the structure at the mid-span of the bridge (on the truss) due to the worst load cases are summarised in table 10 below:

Table 11: Deflection at Bridge Mid-Span due to static and running load cases

Load Cases (Static)	Truss Mid-Span Deflection (mm)
Load Case 0 - Dead Load	8,2
Load case 3: Locomotive at the end of the Bridge pulling 16.5 ton/axle wagon	27,0
Load case 6: Locomotive at the end of the Bridge pulling 22.5 ton/axle wagon	29,8

Load Cases (Running Load)	Truss Mid-Span Deflection (mm)
Load Case 0 - Dead Load	8,2
Locomotive pulling 16.5 ton/axle wagon	30,6
Locomotive pulling 22.5 ton/axle wagon	33,8

In the case of the 16.5-ton axle wagon, the deflection is within the defined serviceability limits for all load cases. This implies that the bridge satisfies the serviceability limit requirements and the overall structure is safe under its current loading. When comparing the results to the worst-case outcomes of (Moyo & Tait, 2010) it was found that the deflections somewhat correlated with the previous results.

Table 12: Deflection at Bridge Mid-Span sourced from (Moyo & Tait, 2010) *Study*

Load Cases	**Truss Mid-Span Deflection (mm)**
Locomotive pulling 16.5 ton/axle wagon	26,3
Locomotive pulling 22.5 ton/axle wagon	36,1

In the case of the 22.5-ton axle wagon, the deflection of the static case was within the defined serviceability limits. The running load case however falls outside of the serviceability limits. It is within 6% of compliance of this requirement. Based on the serviceability requirements the bridge in this state is unsafe and requires strengthening if it is decided to accept the 22.5-ton axle wagons. This could be attributed to differences in certain assumption or the different methodologies or software utilised for the loading on the structure.

Table 13: Deflection at Bridge Mid-Span due to static load based on the Eurocode

Load Cases (Static Eurocode)	**Truss Mid-Span Deflection (mm)**
Model 71	27,0

To identify another comparison for assessment, the Eurocode Model 71 load configuration was applied in the model. This produced a deflection at the mid-span of 27mm which is close to the static load results for the 16 ton/axle case.

9 CONCLUSIONS

The objective of this study was to propose a systematic procedure for the condition assessment of railway bridges. This should yield a better understanding of the performance and condition of the bridge.

A visual inspection is a good starting point for a condition assessment as it provides a qualitative understanding of the defects on the structure. The inspector can identify potential defects that have manifested on the surface of a structure which could in the long term preserve the bridge.
The visual assessment on the Kalbaskraal bridge identified the following concerns:

- Loose bolts on the connection between the cross-beams and the main truss
- Specific sleepers have deteriorated and are even hollow in some cases, this is a major safety concern as they support the rail directly
- There are localised corrosion points on most of the members specifically at connections

Ambient Vibration testing was conducted on the bridge as part of the Operational Modal Analysis to acquire the modal parameters of the bridge. The method was selected based on the equipment available and the ease of implementation. The test results were processed in ME Scope. The results acquired from the field work contained some anomalies which affected the interpretation of the mode shapes. The data was discarded and the test was carried out again. The FRF's and modes shapes acquired from the second test were acceptable. The primary mode shapes were interpreted as follows:

1) Sway Mode Shape at 3.44Hz
2) Vertical Mode Shape at 6.47Hz
3) Torsional Mode Shape at 7.24Hz
4) 2nd Sway Mode Shape at 8.00Hz

It was noted from further evaluation of the mode shapes that the southern bridge truss was stiffer than the northern bridge truss. This was attributed to either the effect of the walkway, the support conditions or the rigidity of the connections.

The finite element model (FEM) was developed based on the existing drawings, onsite measurement and assumed material properties. The purpose of the FEM was to effectively calibrate the model using its modal parameters to provide a better simulation of the real life structure. The parameters selected for calibration were the bridge supports which were modelled as springs. The spring stiffness could then be adjusted to tune the modal parameters of the FEM to align with the modal parameters of the real structure, thus providing a more effective model. The calibration was achieved within the acceptable tolerances.

Static and running loads were imposed on the structure to determine whether it was performing within its serviceability limit states. The governing factor for serviceability in this case was deflection. Based on the response of the structural model, it could be seen that under the current 16 ton/axle loading condition, the structure can perform to its requirements, barring any localised failures. The localised problems should be identified in the visual assessment and rectifying any defects or deterioration can only stiffen the structure. The proposed increase to a 22.5 ton/axle loading still produces deflections at the mid-span within the serviceability limits under static load cases. The 22.5-ton axle running load however fails under serviceability requirements, thus caution should be taken in recommending that the loads can be increased, as there is a large variability in this case from previous studies. This may require some further assessment using the FEM in the analysis of individual structural members as the salient structural response for this study was deflection (ie; for an overall assessment of the structure). Consideration to strengthen the structure must be taken if the loading on the structure is to be increased. Consideration must also be taken that fatigue and measured material factors were not taken into account in this study.

In the bridge management process, the assessment of the condition and performance of the bridge provides so much value to the decision making process. The accuracy of the information provided from the assessment is thus important. The vibration based assessment provides that assurance in that the uncertainty in the behaviour of the structure is reduced. One can see from the process and the conclusions above that this would provide value and validation to the bridge owners.

10 RECOMMENDATIONS

10.1 Future Studies

It is believed that the study has provided merit in a vibration based condition assessment process as the deductions drawn from the systematic procedures required for OMA and FEM updating & analysis, produces a better understanding of the overall performance of the structure and a qualitative understanding of the localised condition of the structure. The following should be considered when implementing future studies:

- Additional material and fatigue analysis would improve the methodology and provide a more rounded assessment of the structure.
- Further study should be conducted to investigate the interaction of the rail-sleeper system and its fixity in relation to the bridge. There are many uncertainties in the fixity conditions which results in additional assumptions in the modelling process. Clarifying these uncertainties would result in a more accurate FEM.
- The irregularities in the analysis data were of particular concern during this study as it was impractical to move back onto site. In the implementation of the fieldwork for OMA, the development of a strict quality control procedure should be put in place. This should include controls for each reading and the initial assessment of the result. Upon leaving the site there is rarely an opportunity to go back and re-test the structure if irregularities or errors are found in the analysis process. Although a quality control procedure was in place for this study, there were still some gaps. As a rule of thumb, an initial rudimentary analysis of the data should be conducted in ME Scope whilst on site; as there is a lot of time available while the data is being collected. Such a duty could be designated to a team member.

11 ANNEXURE

11.1.1 Annexure A: Visual Assessment

Transnet Freight Rail			**BRIDGE Field Inspection Sheet**	**No.**	
				Name	KalbasKraal Railway Bridge
Inspection Type		**Inspector**	**Firm**	**Date**	**Route/Section**	Saldanha
Current	**PR**	Hendricks	UCT	05-May-16	**Route km**	N/A
Last Principle	**PR**				**N Route Over/Under**	River
Last Monitoring	**MO**				**Feature Name**	
Last Maintenance	**MA**					
Last Verification	**VE**				**Feature Road No.**	N/A
Bridge Type		Simply Supported	**No. of Spans**	2	**Direction of River Flow**	
Year Constructed		1932	**Overall Length**	32 x 2		
Bridge Orientation						
Time (Hours)			**Inspection**	1		

Inspection Item	**D**	**E**	**R**	**U**	**Picture (If Available)**	**Comments**
1. Appoach Embankment	U					
2. Guardrail	N/A					
3. Waterway	1	4	1	R		Waterway is dry
4. Appr. Emb/ Prot. Works	N/A					
5. Abutment Foundations	U					
6. Abutments	1	1	4	0		The abutments are in good condition. No deterioration. Water and rust staining due to lack of drainage at the abutment
7, Wing/Retaining Walls	N/A					
8. Surfacing	N/A					
9. Superstructure Drainage	N/A					
10. Kerbs/Sidewalks	N/A					
11. Parapets	N/A					
Supports						
12. Pier Protection works	N/A					
13. Pier Foundations	U					

14.Piers/Columns	1	2	4	0		The abutments are in good condition. No deterioration. Water and rust staining due to lack of drainage at the abutment
15. Bearings	2	3	3	1		Bearing show sign of corrosion and minor cracks stemming from stress concentration. Bolts are in tact
16. Support Drainage	N/A					
17. Expansion Joints	N/A					
Spans						
18. Longitudinal Members	2	2	4	1	 	Generally in good condition. Rivets are tight. Localised corrosion. Channel filled with debris. Minimal drainage from channel
19. Transverse Members	3	2	4	3	 	Transverse members are in fair condition. Localised corrosion. Some bolts at the supports are loose which will affect the stiffness of the overall bridge. Bracing members are in good condition. No major deterioration
20. Decks and Slabs	N/A					
21. Misc. Items	3	3	3	4		Rail shows localised plastic deflection. A few timber sleepers are deteriorated and has hollow sections. Could prove dangerous as it becomes more brittle

11.1.2 Annexure B: Equivalent Bridge Sections

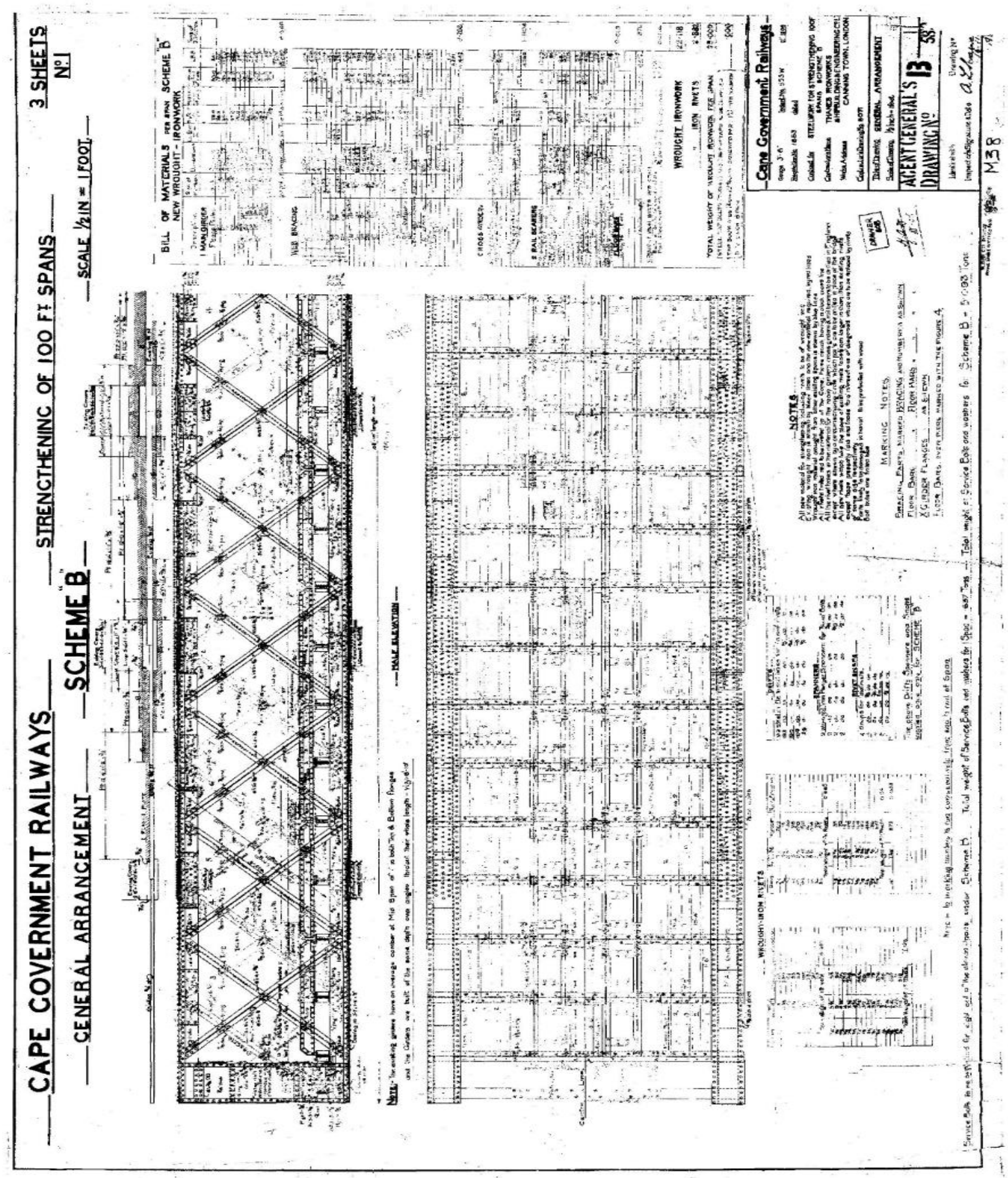
Cross-girder with varying section properties						
As-build section properties	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Unequal angle thickness (mm)	12,7	12,7	12,7	12,7	12,7	12,7
Web plate thickness (mm)	6,4	6,4	6,4	6,4	6,4	6,4
Web plate depth (mm)	219,3	219,3	376,9	419,8	438,6	453,7
Angle horizontal (mm)	101,6	101,6	101,6	101,6	101,6	101,6
Angle vertical (mm)	88,9	88,9	88,9	88,9	88,9	88,9
Flange plate thickness (mm)			11,1	11,1	11,1	11,1
Flange plate width (mm)			209,6	209,6	209,6	209,6
CA top (mm^3)	1,14E+06	1,14E+06	2,30E+06	2,60E+06	2,74E+06	2,85E+06
CA bot (mm^2)	10435,8	10435,8	13771,0	14045,5	14165,8	14262,5
CA final (mm)	109,7	109,7	166,8	185,3	193,5	200,0
Ixx (mm^3)	7,53E+07	7,53E+07	2,62E+08	3,35E+08	3,70E+08	4,00E+08
Iyy (mm^3)	1,99E+07	1,99E+07	2,84E+07	2,84E+07	2,84E+07	2,84E+07
Equivalent sections						
Flange thickness (mm)	16,5	16,5	16,0	16,0	16,0	16,0
Web thickness (mm)	13,0	13,0	13,0	13,0	13,0	13,0
Flange width (mm)	193,5	193,5	220,0	220,0	220,0	220,0
Total depth (mm)	220,0	220,0	370,0	410,0	430,0	444,0
CA top (mm^3)	1,04E+06	1,04E+06	2,12E+06	2,45E+06	2,63E+06	2,75E+06
CA bot (mm^2)	9471,0	9471,0	11434,0	11954,0	12214,0	12396,0
CA final (mm)	110,0	110,0	185,0	205,0	215,0	222,0
Ixx (mm^3)	7,52E+07	7,52E+07	2,63E+08	3,32E+08	3,70E+08	3,98E+08
Iyy (mm^3)	2,00E+07	2,00E+07	2,85E+07	2,85E+07	2,85E+07	2,85E+07

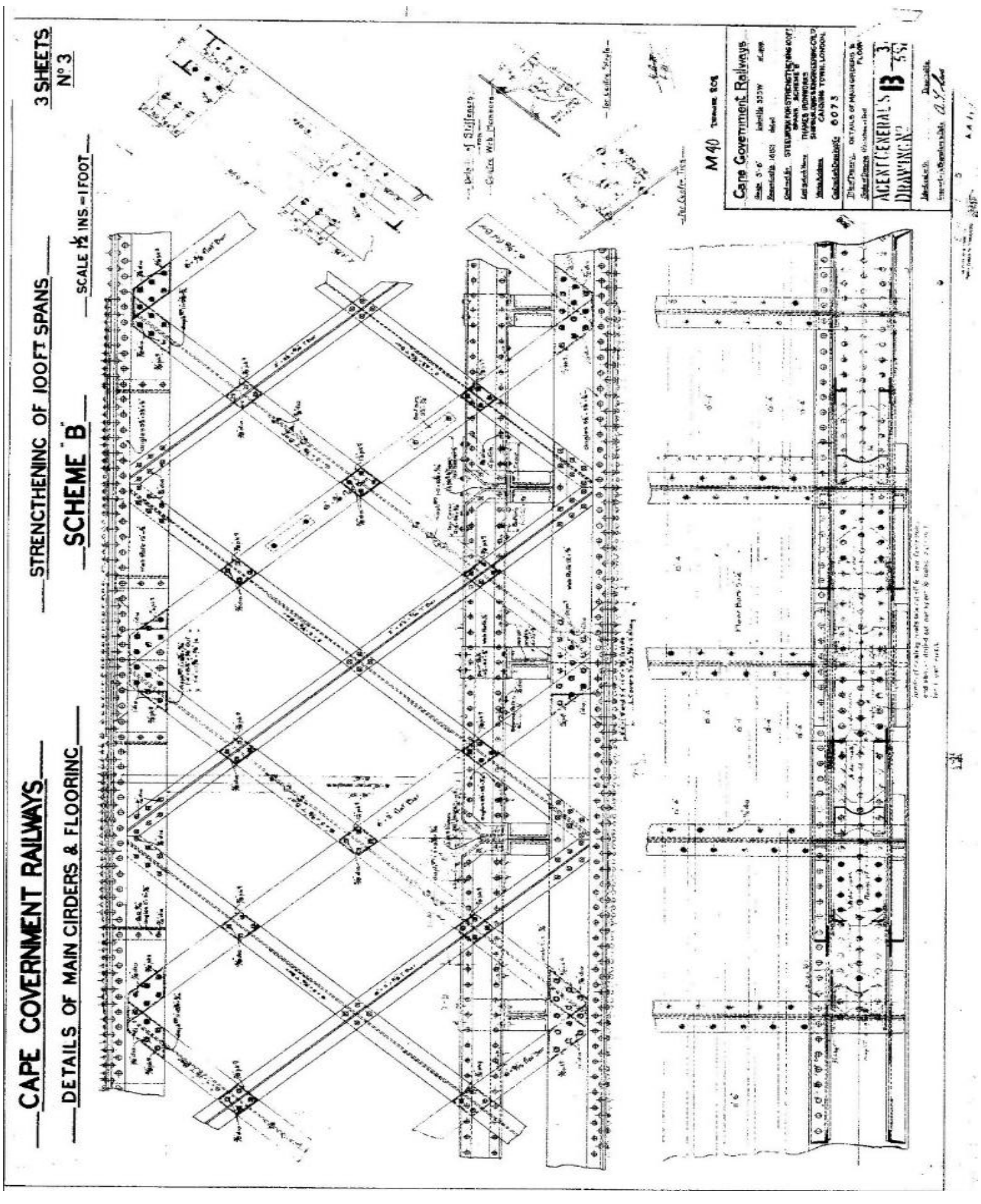
Main girder with varying section properties

Section number	1	2	3	4	5	6	7	8	9	10	11	12	13	14
As-build														
Equal angle (mm)	88,9	88,9	88,9	88,9	88,9	88,9	88,9	88,9	88,9	88,9	88,9	88,9	88,9	88,9
Angle thickness (mm)	12,7	12,7	12,7	12,7	12,7	12,7	12,7	12,7	12,7	12,7	12,7	12,7	12,7	12,7
Flange width (mm)	304,8	304,8	304,8	304,8	304,8	304,8	304,8	304,8	304,8	304,8	304,8	304,8	304,8	304,8
Flange plate thickness (mm)	9,5	9,5	9,5	9,5	9,5	9,5	9,5	9,5	9,5	9,5	9,5	9,5	9,5	9,5
Web plate depth (mm)	533,4	533,4	533,4	533,4	533,4	533,4	533,4	533,4	533,4	533,4	533,4	533,4	533,4	533,4
Web plate thickness (mm)	7,9	7,9	7,9	11,1	11,1	11,1	11,1	17,5	17,5	17,5	17,5	19,1	19,1	19,1
Web plate 1 depth (mm)				533,4	355,6	355,6	355,6	355,6	355,6	355,6	355,6	533,4	355,6	355,6
Web plate 1 thickness (mm)				9,5	9,5	9,5	9,5	9,5	12,7	12,7	12,7	19,1	19,1	19,1
Web plate 2 depth (mm)				254,0		330,2	330,2	330,2	330,2	330,2	330,2	330,2	330,2	330,2
Web plate 2 thickness (mm)				9,5		9,5	9,5	1,6	9,5	9,5	9,5	1,6	11,1	11,1
Web plate 3 depth (mm)							330,2	330,2	279,4		330,2	330,2	279,4	
Web plate 3 thickness (mm)							9,5	9,5	9,5		11,1	11,1	9,5	
Web plate 4 depth (mm)							279,4	304,8			279,4	304,8		
Web plate 4 thickness (mm)							9,5	9,5			9,5	12,7		
Web plate 5 depth (mm)								279,4				304,8		
Web plate 5 thickness (mm)								12,7				9,5		
Web plate 5 depth (mm)												279,4		
Web plate 5 thickness (mm)												12,7		
CA top (mm³)	1,09E+06	1,09E+06	1,09E+06	1,38E+06	1,31E+06	1,51E+06	1,85E+06	2,09E+06	1,88E+06	1,77E+06	2,23E+06	3,12E+06	2,19E+06	2,05E+06
CA bot (mm²)	9227,6	9227,6	9227,6	18417,0	14302,0	17448,8	23258,3	27816,0	24631,1	21968,5	28296,4	42441,1	28255,7	25593,0
CA final (mm)	118,3	118,3	118,3	74,7	91,4	86,3	79,7	75,3	76,5	80,4	79,0	73,4	77,6	80,1
Iyy (mm³)	1,14E+08	1,14E+08	1,14E+08	1,47E+08	1,35E+08	1,52E+08	1,63E+08	1,60E+08	1,66E+08	1,64E+08	1,74E+08	2,01E+08	1,73E+08	1,72E+08
Average Iyy (mm³)	1,14E+08	1,14E+08	1,31E+08	1,41E+08	1,44E+08	1,58E+08	1,61E+08	1,63E+08	1,65E+08	1,69E+08	1,87E+08	1,87E+08	1,72E+08	
Ixx (mm³)	3,96E+08	3,96E+08	3,96E+08	5,69E+08	4,72E+08	5,00E+08	5,46E+08	6,31E+08	6,10E+08	5,93E+08	6,43E+08	8,91E+08	6,58E+08	6,41E+08
Average Ixx (mm³)	3,96E+08	3,96E+08	4,83E+08	5,21E+08	4,86E+08	5,23E+08	5,89E+08	6,20E+08	6,01E+08	6,18E+08	7,67E+08	7,75E+08	6,50E+08	6,50E+08
Equivalent sections														
Web thickness (mm	18,0	18,0	18,0	19,0	23,0	28,0	18,5	18,9	19,5	20,3	24,8	24,4	20,7	20,7
Web depth (mm)	418,0	418,0	418,0	423,0	417,0	416,1	420,0	421,0	414,0	417,5	433,2	435,5	425,6	425,6
Flange thickness (mm)	13,0	13,0	18,0	19,0	16,5	17,0	23,0	25,0	25,0	25,0	30,0	30,0	25,0	25
Flange width (mm)	285,0	285,0	276,0	278,0	286,0	290,0	280,0	275,0	276,0	277,0	270,0	270,0	278,0	278
CA top (mm³)	1,12E+06	1,12E+06	1,43E+06	1,54E+06	1,45E+06	1,58E+06	1,87E+06	1,96E+06	1,97E+06	1,99E+06	2,30E+06	2,30E+06	2,01E+06	2,01E+06
CA bot (mm²)	14466,0	14466,0	16812,0	17879,0	18243,7	20558,8	19799,0	20761,9	20898,0	21310,3	25455,4	25362,2	21674,9	21674,9
CA final (mm)	77,4	77,4	85,2	86,0	79,3	76,8	94,3	94,3	94,4	93,6	90,4	90,6	92,9	92,9
Iyy (mm³)	1,15E+08	1,15E+08	1,31E+08	1,41E+08	1,43E+08	1,58E+08	1,61E+08	1,63E+08	1,65E+08	1,69E+08	1,87E+08	1,87E+08	1,72E+08	1,72E+08
Ixx (mm³)	3,94E+08	3,94E+08	4,81E+08	5,22E+08	4,86E+08	5,23E+08	5,89E+08	6,20E+08	6,01E+08	6,18E+08	7,67E+08	7,75E+08	6,50E+08	6,50E+08

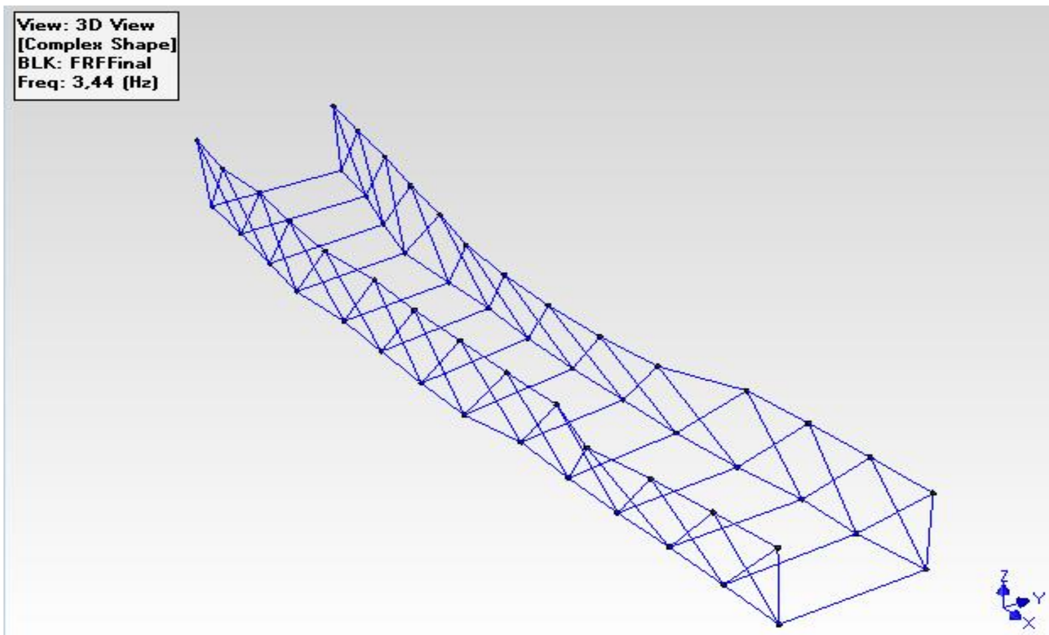
Vertical end beam	
As-build section	
Flange plate thickness (mm)	9,5
Flange plate width (mm)	533,4
Equal angle (mm)	76,2
Angle thickness (mm)	9,5
Web plate thickness (mm)	11,1
Web plate depth (mm)	795,0
CA top (mm³)	9,45E+06
CA bot (mm²)	28178,5
CA final (mm)	335,3
Iyy (mm³)	3,11E+09
Equivalent section	
Web thickness	30,0
Web depth	533,0
Flange thickness	25,0
Flange width	750,0
CA top (mm³)	1,43E+07
CA bot (mm²)	51990,0
CA final (mm)	274,7
Iyy (mm³)	3,11E+09

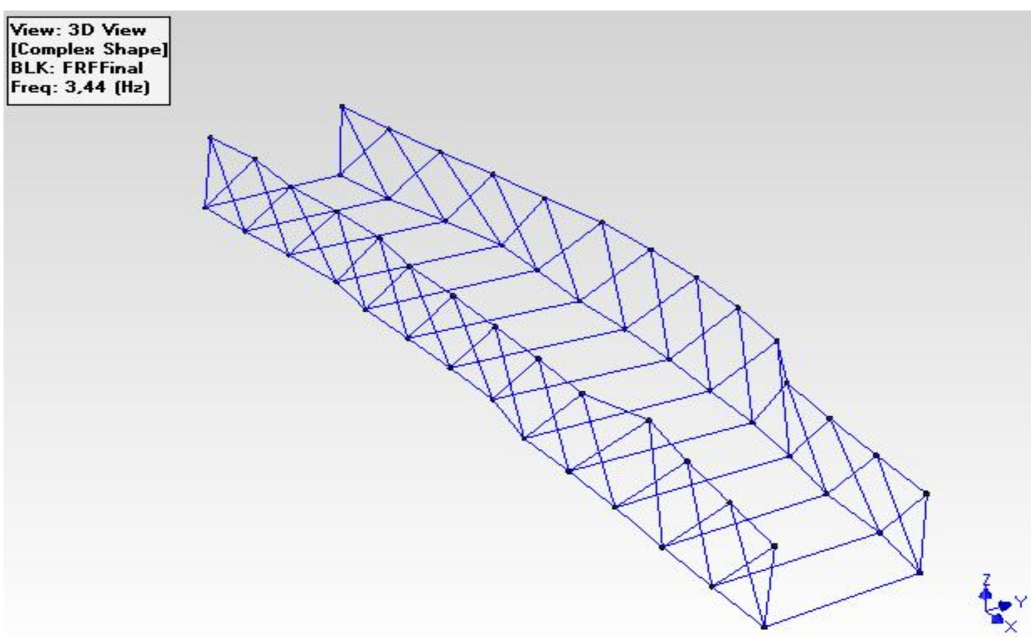
11.1.3 Annexure C: Available Drawings



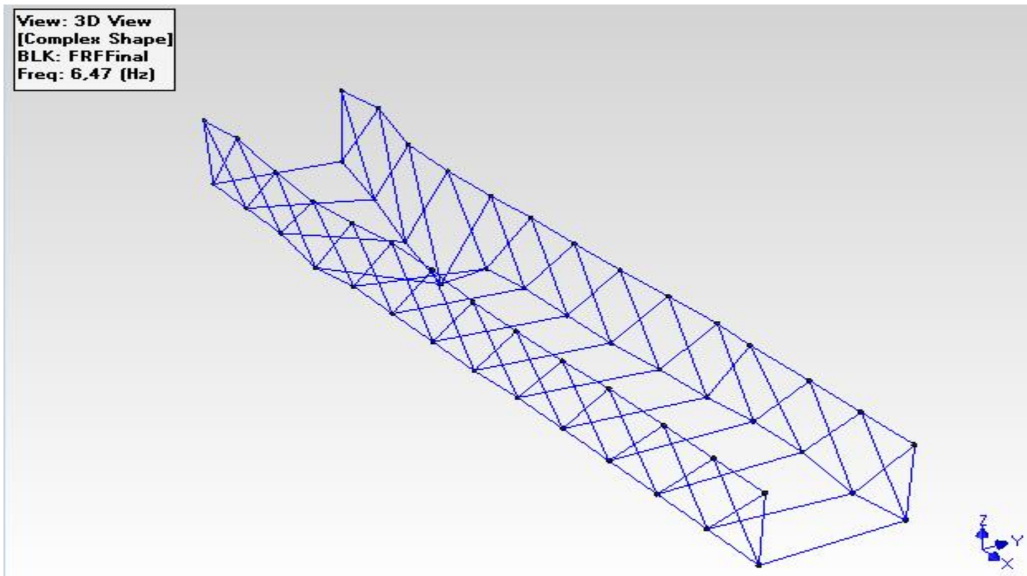

CAPE GOVERNMENT RAILWAYS
DETAILS OF MAIN GIRDERS & FLOORING
STRENGTHENING OF 100FT SPANS
SCHEME 'B'
SCALE 1½ INS = 1 FOOT
3 SHEETS
No 3
M40
Cape Government Railways
AGENT GENERAL'S
DRAWING No
B 3/35

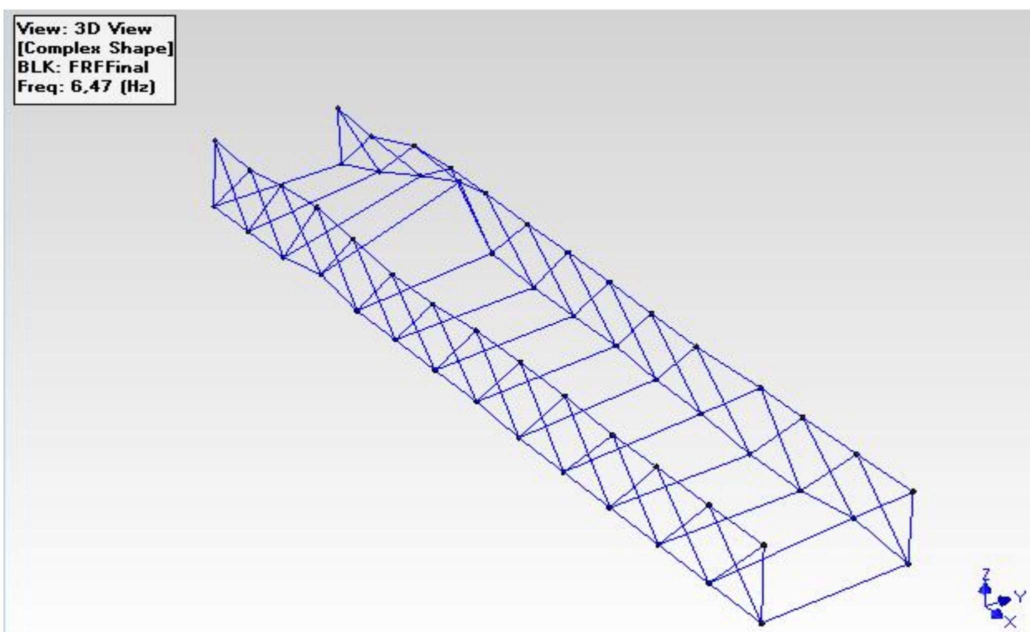
11.1.4 Annexure D: ME Scope Results



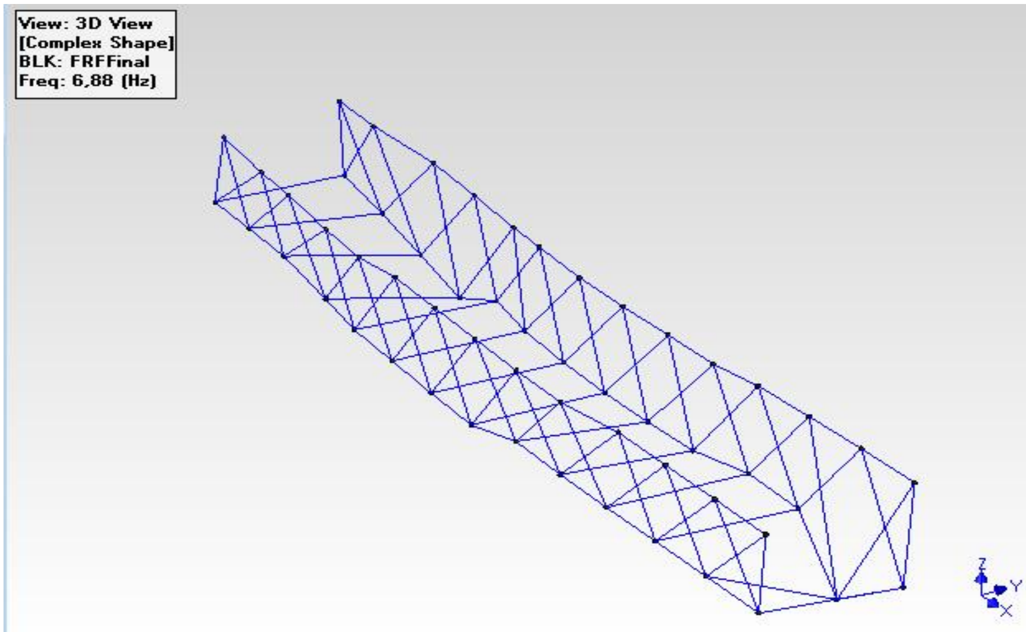


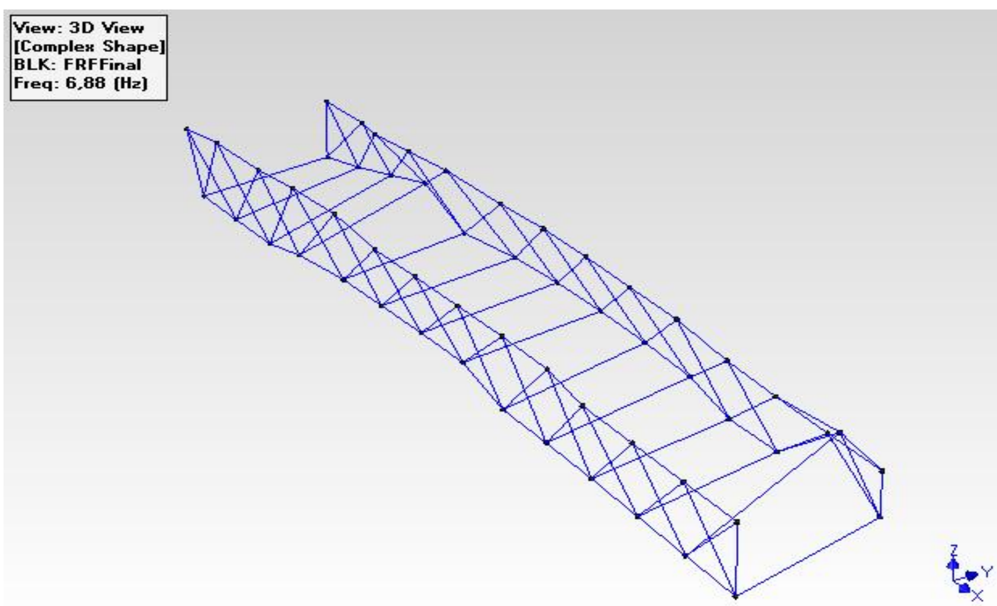
MEScopeVES 1st Sway Mode at 3.44hz



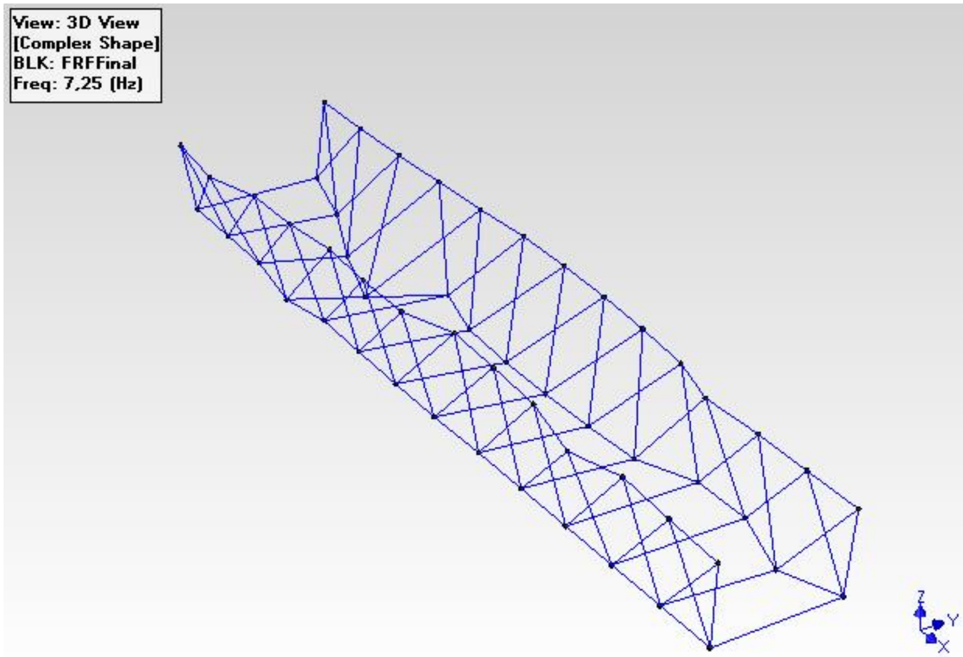


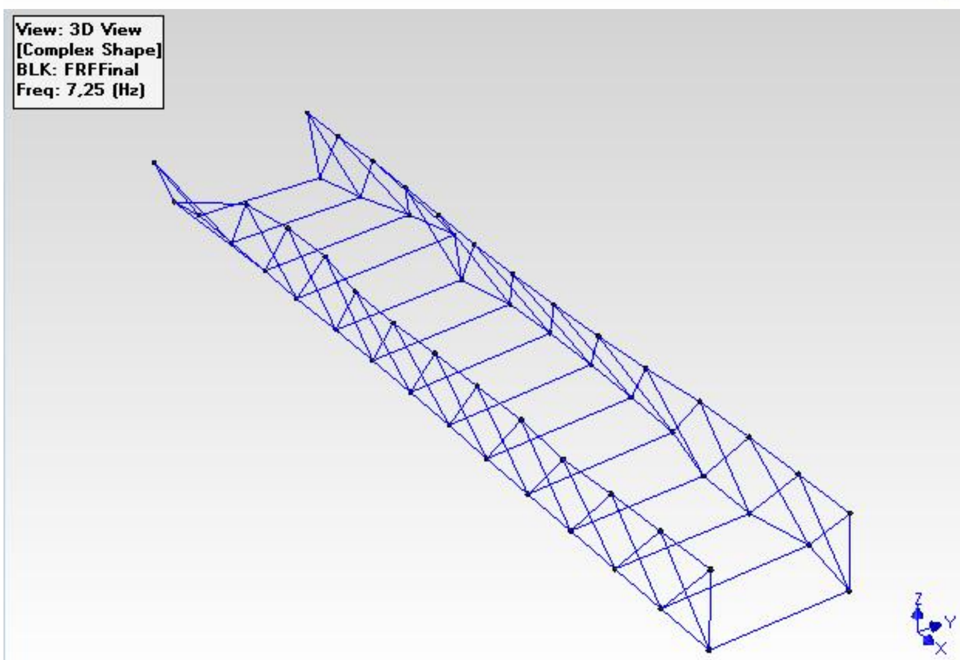
MEScopeVES Vertical Mode at 6.20

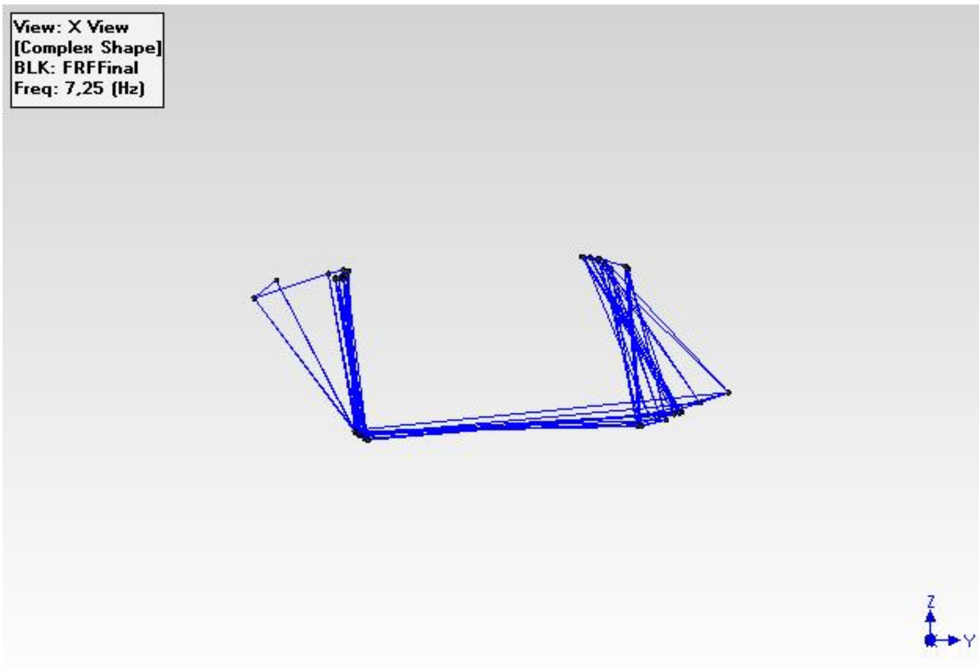


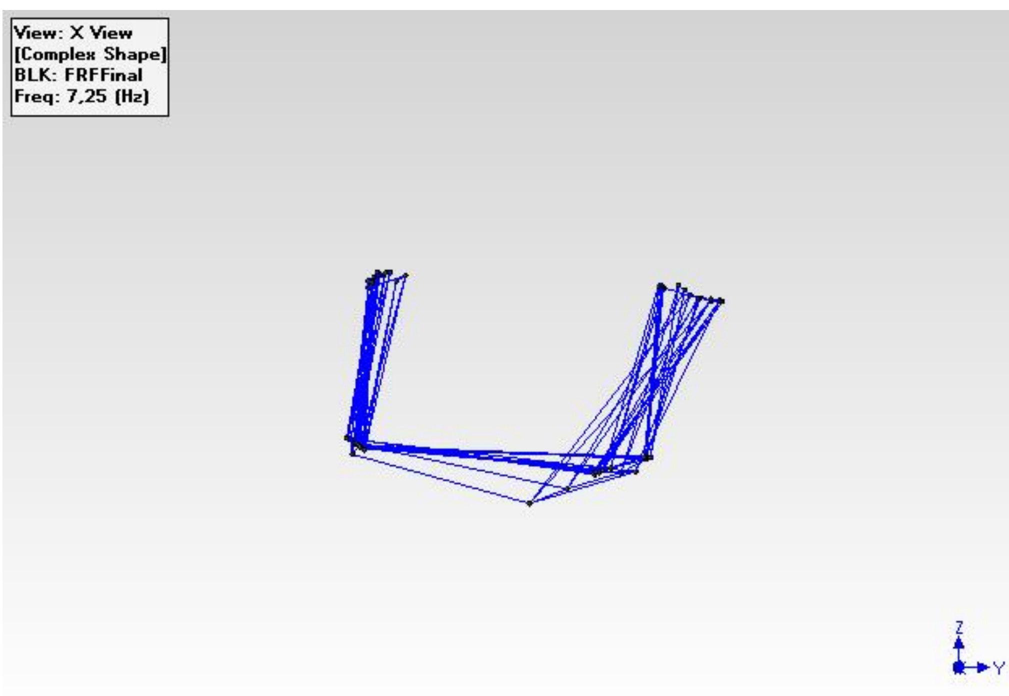


MEScopeVES Vertical Mode at 6.88

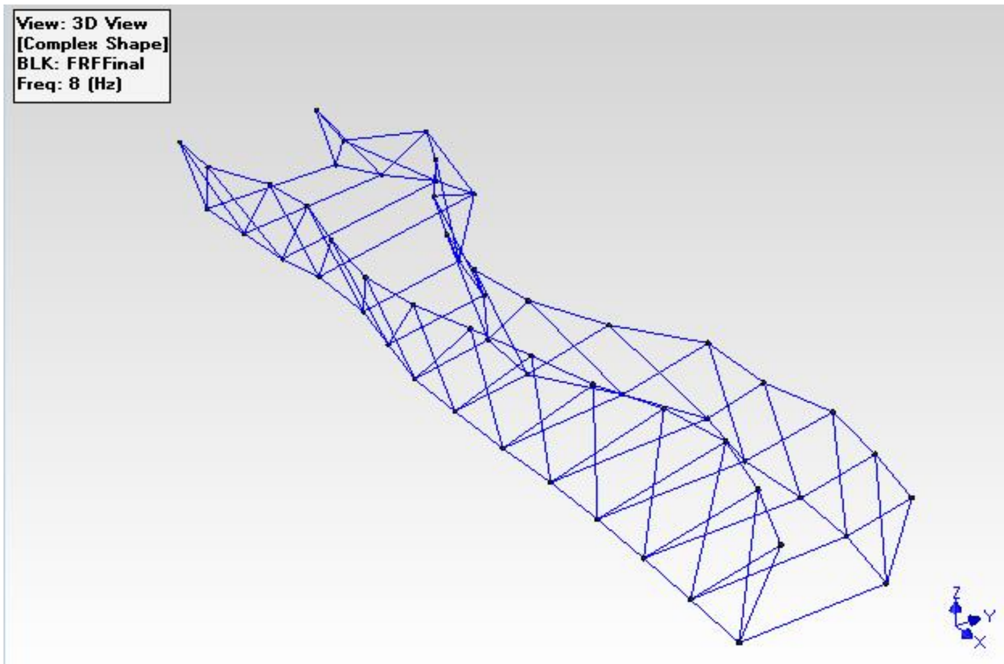

View: 3D View
[Complex Shape]
BLK: FRFFinal
Freq: 7,25 (Hz)
Z
Y
X

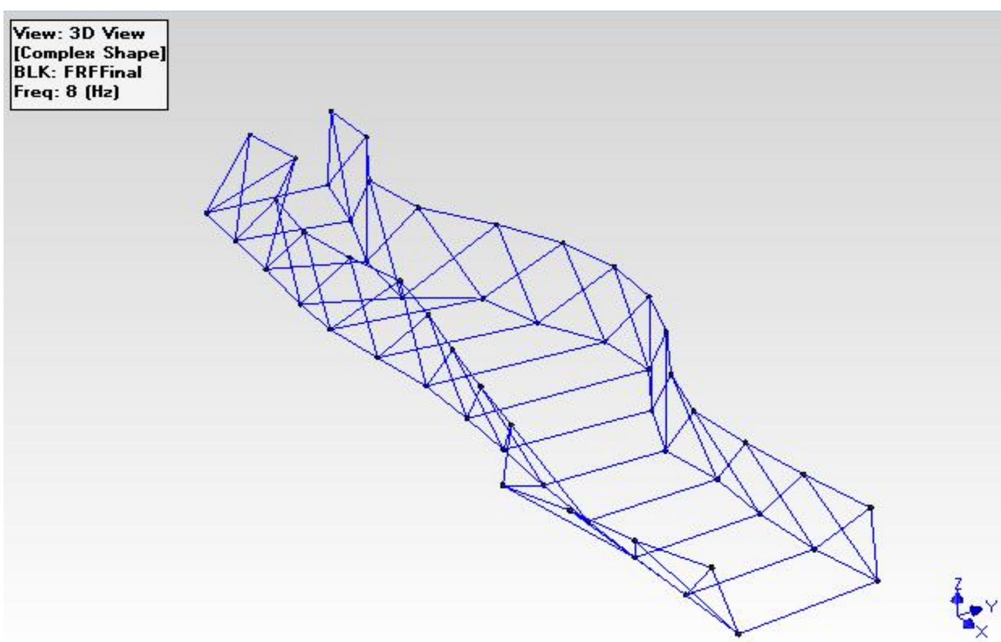

View: 3D View
[Complex Shape]
BLK: FRFFinal
Freq: 7,25 (Hz)
Z
Y
X





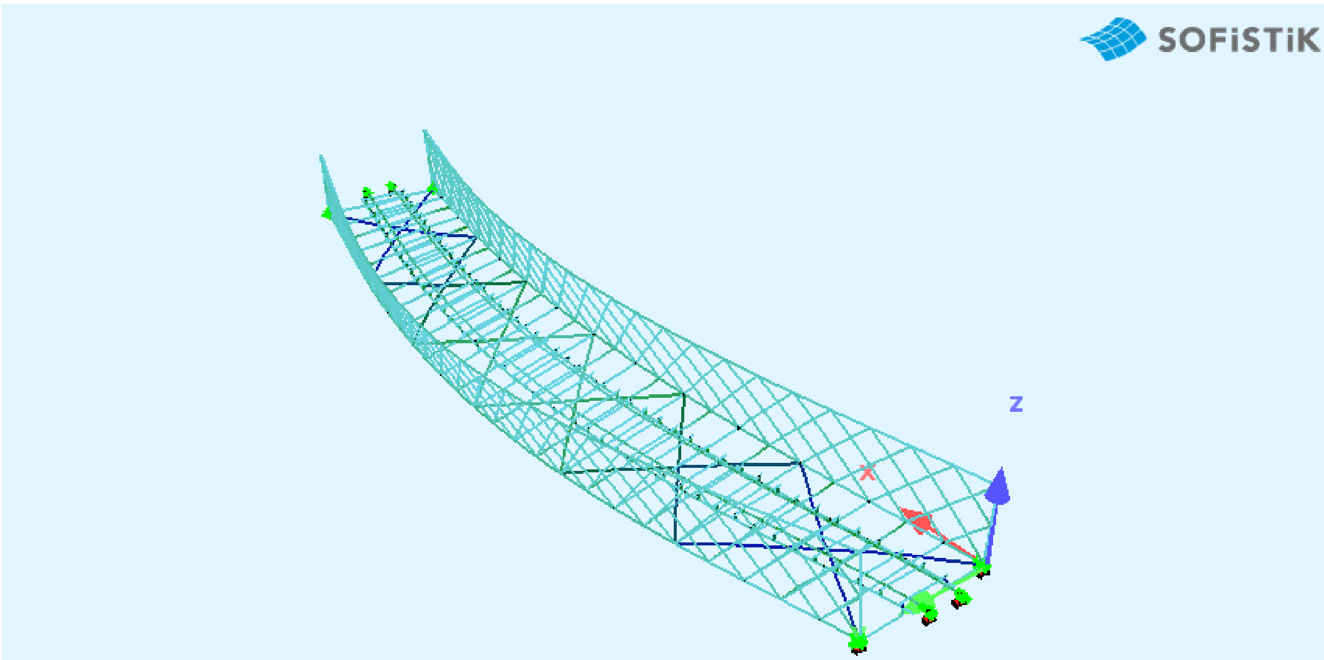
MEScopeVES Torsional Mode at 7.25Hz



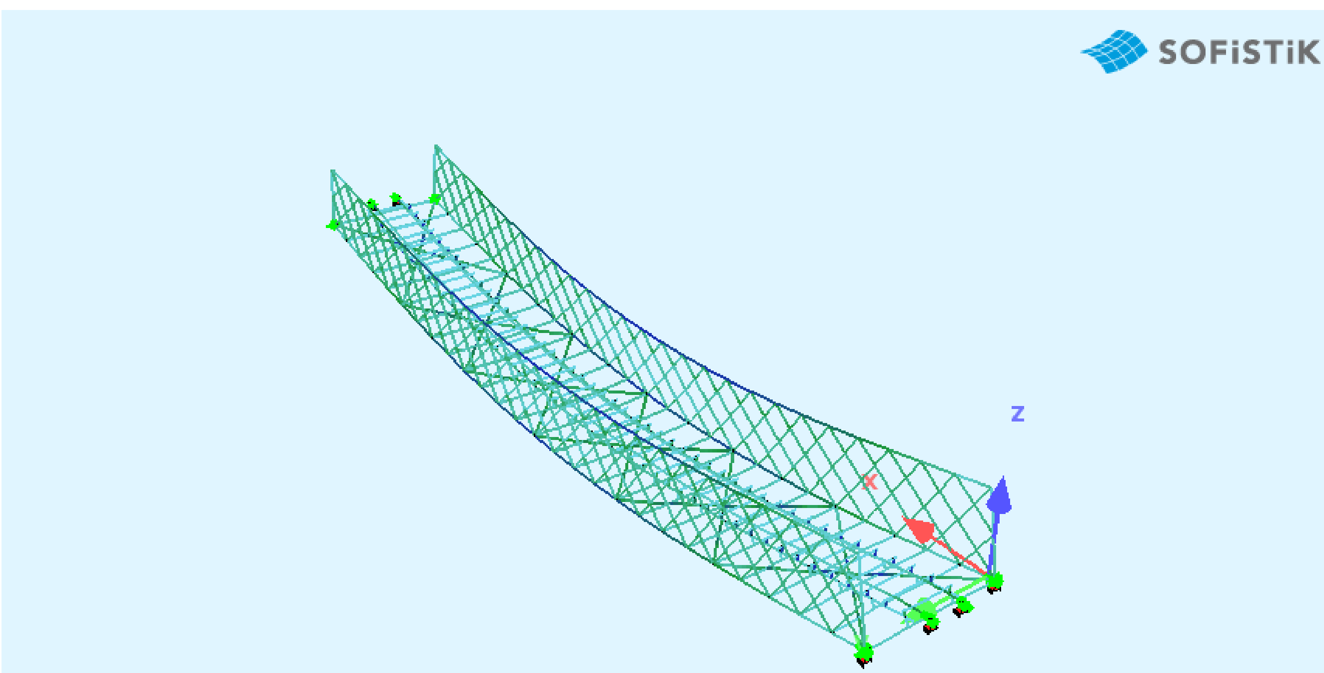


MEScopeVES 2nd Sway Mode at 7.92Hz

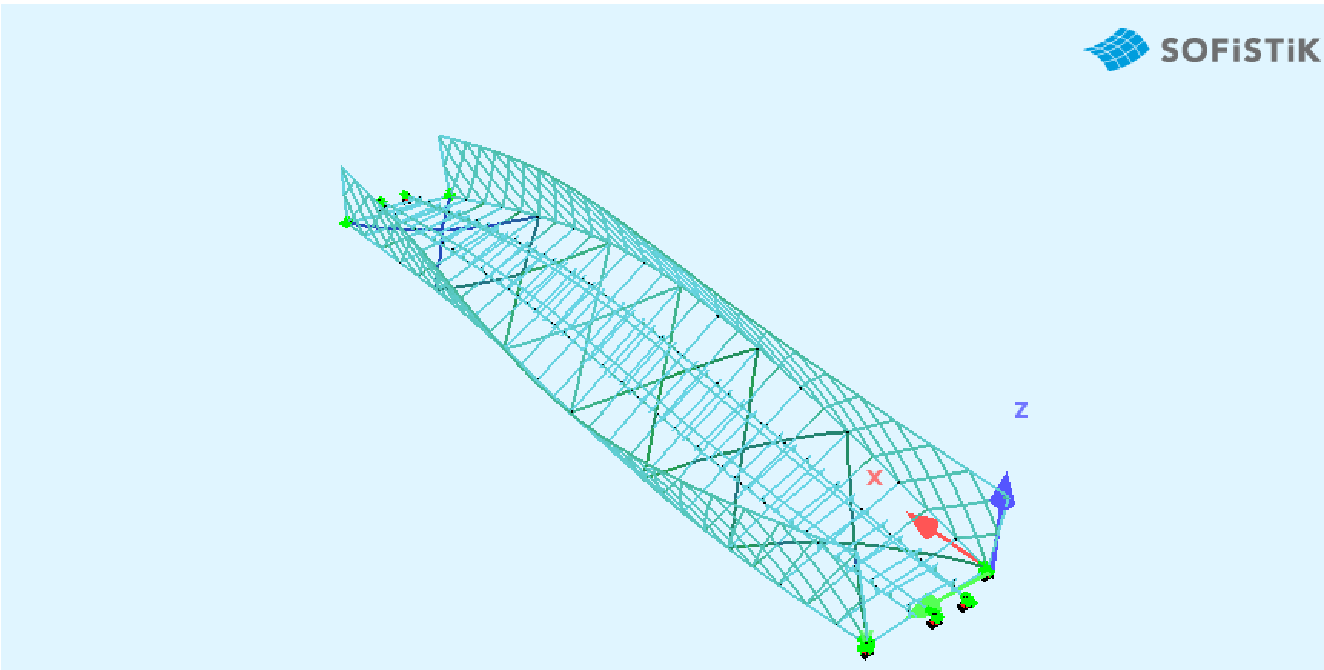
11.1.5 Annexure E: FEM Mode Shapes



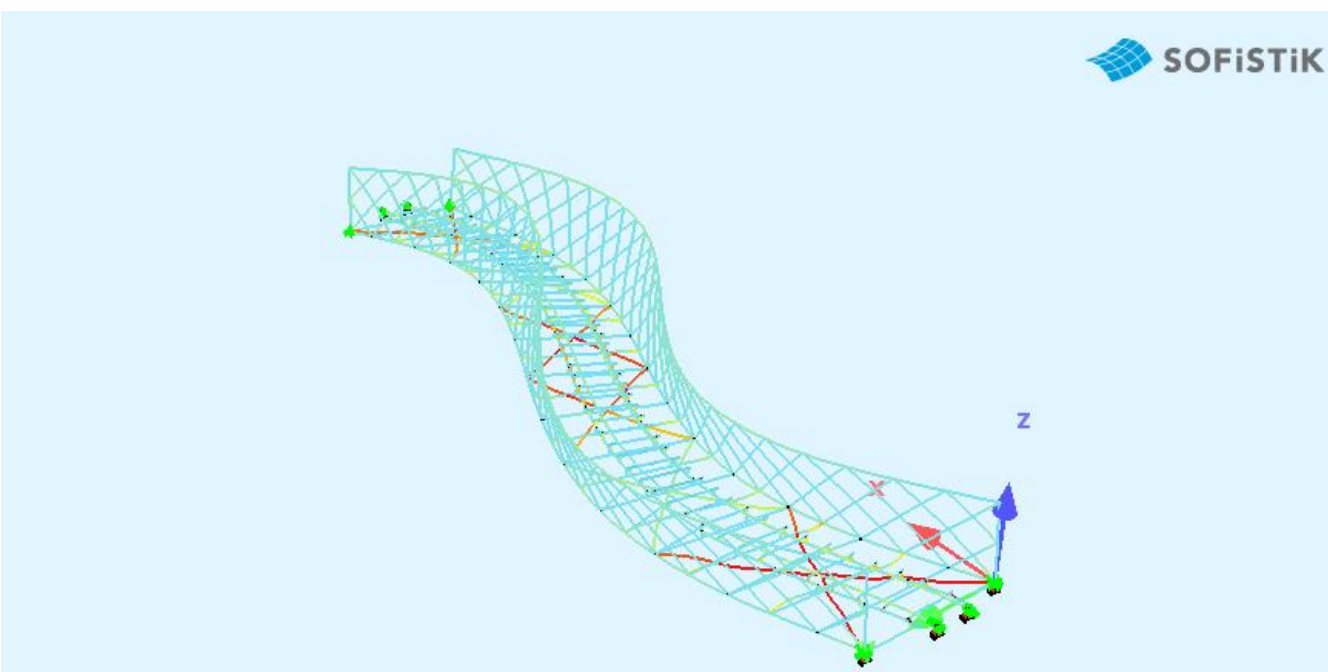
Sofistik CAE 1st Sway Mode at 3.44



Sofistik CAE 1st Vertical Mode at 6.20

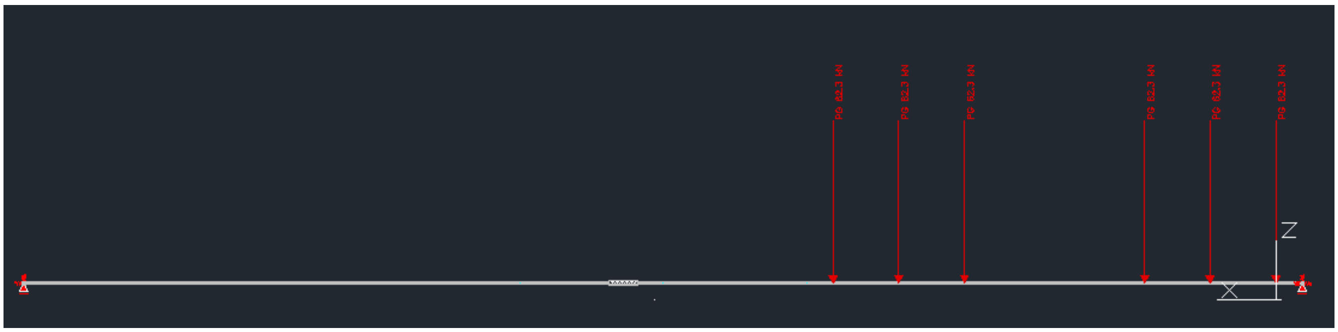


Sofistik CAE 1st Torsional Mode at 7.81

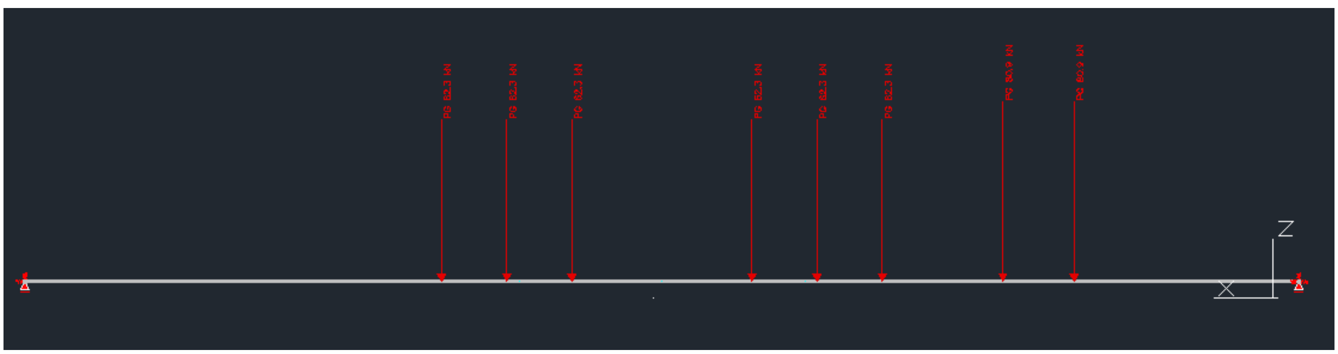


Sofistik CAE 2nd Sway Mode at 7.92

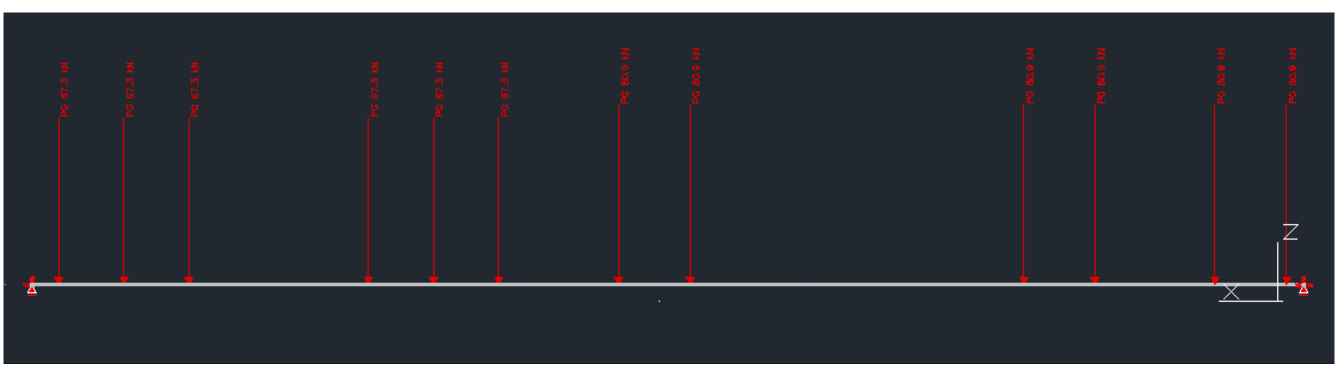
11.1.6 Annexure F: Static Load Cases



- Load case 1: Locomotive at the start of the Bridge



- Load case 2: Locomotive at the mid-span of the Bridge pulling 16.5 ton/axle wagon
- Load case 5: Locomotive at the mid-span of the Bridge pulling 22.5 ton/axle wagon



- Load case 3: Locomotive at the end of the Bridge pulling 16.5 ton/axle wagon
- Load case 6: Locomotive at the end of the Bridge pulling 22.5 ton/axle wagon

12 REFERENCES

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