SERVICEABILITY ASSESSMENT OF PEDESTRIAN BRIDGES

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ABSTRACT

Frequency analysis of existing structures is a realistic and cost effective manner in which to conduct serviceability assessments. This type of analysis uses vibration testing and dynamic analysis to identify any change in a structure's natural frequencies, thereby highlighting any degradation invisible to routine inspection. This report aims to analyse a train bridge in Dublin, Ireland and pedestrian bridge on Blanchard Street, Thunder Bay using vibration data. The railway bridge will be analysed using frequency analysis to identify the different types of trains operating on the bridge whilst taking their respective speeds into account. The pedestrian bridge was exposed to a multitude of excitations while the vibrational response was recorded. A finite element model was created modelling the existing Blanchard Street bridge using the S-Frame program. The frequency results from the vibrational data were then compared to the 3D model.

It was determined by using each trains physical characteristics (i.e. weight, number of cars, placements of axles, etc.) and unique frequency plots created from the vibrational data, that one could match a train and its resulting plot accurately. In regards to the pedestrian bridge, it was established that using the 3D model created with S-Frame to compare with the vibrational data allowed for accurate and feasible serviceability assessment of structure.

I.INTRODUCTION

HERE were two major objectives for this project. First is to analyse a train bridge in Dublin Ireland using vibrational, and subsequently, frequency analysis. The second objective of this project was to conduct a serviceability assessment using frequency analysis on a pedestrian bridge located in Thunder Bay, Ontario.

The Irish railway bridge was analysed by recording the acceleration data at the mid-span of the bridge under different dynamic loads created by the TGV and the 201LOCO trains. The trains which cross the bridge at speeds ranging from 40 km/hr to 300 km/hr, and have different physical characteristics such as mass, weight distribution and number of carriages. Each train causes a unique vibrational response in the bridge's structure. These responses were recorded and were analysed with respect to frequency, with the objective of matching each unique frequency response to the train responsible for the creation of that unique response.

The pedestrian bridge in Thunder Bay was exposed to a multitude of excitations while the vibrational responses were recorded. Blue prints provided by the City of Thunder Bay were used to create an accurate model in S-Frame using finite element method (FEM). The recorded data was transformed to frequency domain and compared with frequency results generated in the S-

Frame program. Discrepancies between the two would indicate degradation in the real bridge's structure, perhaps section loss, due to a factor such as extreme oxidization.

1.1FEM Software Evaluation

A minor objective of the project is to evaluate the use of S-Frame, particularly the dynamic analysis with respect to frequency to conduct the serviceability assessments. This will be conducted simultaneously with the pedestrian bridge serviceability assessment, by comparing the frequency results from the dynamic analysis in S-Frame with the frequency results obtained from the accelerometers during the trials on the actual bridge. Due to the simple design of the pedestrian bridge utilizing visual inspection, it was determined that there was no major section loss in the bridge's substructure. Therefore, the vibrational data generated during the trials should be the bridges natural frequencies or modes (baseline). Ideally the comparison between frequency results should be similar, if not match, proving the adequacy of software such as S-Frame to be utilized in part during a serviceability assessment using frequency analysis

1.2Frequency Analysis and Structure Integrity

Frequency analysis has many different applications ranging from economic data analysis to inverse synthetic aperture radar. In civil engineering, it can be used to perform serviceability assessments on existing structures. This is particularly useful because all structures deteriorate over time without preventative maintenance and in some cases rehabilitation. This decay in the structure's components diminishes capacity, reduces overall strength and if left unmitigated can result in failure. The extent and type of deterioration is usually dependent on the environment in which the structure is located and the loads it is subjected to. While some degradation is visible to the naked eye and easily diagnosed, other types are more difficult to identify. Also, at which point does the deterioration becomes significant enough to warrant a preventative or restorative response. Fig.1 clearly demonstrates how deterioration through oxidization can have a detrimental effect on structural components as well as how severe damage can become if left unattended.

Fig. 1. Deteriorated Girder

Currently, conventional methods of monitoring structures' overall health rely heavily upon regular inspections by certified individuals. While this has proven to be effective, it is time consuming, costly, and somewhat impeded by visual limitations. This impairment reduces the accuracy and effectiveness of visual inspections and can perpetuate unidentified problems sometimes resulting in catastrophic failure as in the case of the Latchford Bridge located in Ontario on Highway 11 in 2003. Frequency analysis is an inexpensive and effective method to completely monitor a structure's health, even in areas invisible to the naked eye, making it a viable method to be used in conjunction with physical inspections. The analysis requires the placement of accelerometers in strategic positions on the structure, which measure the acceleration of the structure when it is exposed to dynamic loads. The vibrational data is then transformed using Fast Fourier Transform (FFT) from a time domain into a frequency domain. This data is then used to determine the natural frequencies or modes of the structure. Over the course of the structure's life, frequency analysis is performed regularly with the results being compared to the original natural frequencies or baseline frequencies. Any discrepancies between the two results would indicate a deterioration of the structure or component(s) of the structure.

This method of serviceability assessment can be simplified to comparing frequency data on a monitor which is more accurate and precise than undertaking a complete inspection. The amount of time spent, as well as the financial savings are both favourable for frequency analysis assessment when compared with standard inspection assessments. Also, if baseline frequencies have not been recorded initially, the use of dynamic analysis on the structure through the utilization of finite element modelling software (S-Frame for this project) is acceptable. In this case study frequency results from the FEM are compared with the vibration data recorded from the structure. Similar to the baseline comparison, any discrepancies would indicate deterioration in the structure. In addition, it should be noted that not only is frequency analysis able to determine if deterioration has occurred, but can also estimate the extent of the damage to the substructure.

1.3Lateral Displacement

As bridges become longer and high slender, lateral sway as opposed to just vertical displacement is becoming an issue which requires attention in the design phase. This is a relatively new phenomenon which affects the serviceability of the structure, while not necessarily affecting structural integrity. If lateral displacement occurs, even a couple of feet which may not be detrimental, could render the bridge unpassable to pedestrians. This reduction in serviceability would not only be expensive and time consuming to remedy, the bridge itself would be closed for the entire extent of the repairs.

Lateral displacement in bridges is caused by external excitation which include, but are not limited to dynamic loading caused by people or traffic moving across or by forces such as the wind striking the side of the structure. Regardless of the cause, if the lateral displacement is not dampened and oscillation continues unhindered, there is a chance that frequency of the oscillations may match the bridge’s natural frequency. If this were to happen, the entire structure could fail resulting in the loss of structural integrity. A famous example of this was the Tacoma Narrows Bridge in Washington in 1940. Wind striking the side of the bridge caused severe oscillations which matched the natural frequency of the structure. The bridge ultimately collapsed due to the resonance it was experiencing.

1.4Structural Dynamics

While the acceleration or amplitude of a structure is dependent upon external dynamic loading or excitation, the frequency response of said structure is independent. Said response is dependent upon the nature of the structure itself in terms of stiffness and mass. During excitation, a structure will vibrate at specific modes or frequencies unique to that structure. Frequency, or in this case natural frequency, is related to stiffness and mass by the equation

$$f = \frac{1}{2\pi}\sqrt{\frac{k}{m}}$$

Where M is Mass, K is Stiffness

The amplitude when the structure vibrates can be calculated by:

$$A = 2\pi f$$

Where A is Amplitude, F is Frequency.

For a system with a single degree of freedom, the system has one mass and vibrates at only one frequency or mode. This corresponding frequency is known as the resonance or natural frequency. For more complicated systems with additional degrees of freedom there are specific frequencies, corresponding to the different degrees of freedom. In bridges, where mass is usually uniformly distributed along the entire length, there are an infinite number of degrees of freedom, and consequently, an infinite number of natural frequencies and equivalent modes. There are only a few resonance frequencies that oscillate with significant amplitude, and it is only these frequencies which are of importance. Modes are defined as the frequency pattern at which the structure vibrates. It should be noted that the first few modes are comprised of the majority of the energy applied to the structure. The figure below illustrates natural modes of a rectangular bar.

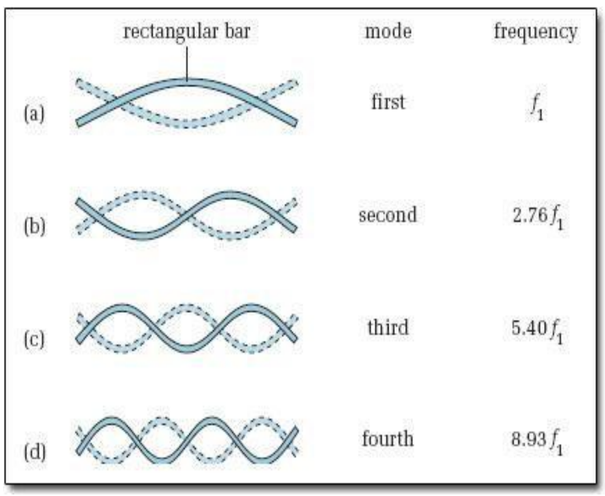

Fig 2. Example of Vibrational Modes

Resonance frequencies can be discovered through a process called Modal Analysis or through testing. This process utilizes the general equation of motion which is expressed by the following second order differential equation

$$[m]\ddot{x} + [c]\dot{x} + [k]x = F(t)$$

Where [M] is mass matrix, [C] is damping matrix, [K] is stiffness matrix, $\ddot{x}$ is acceleration vector, $\dot{x}$ is velocity vector, x is displacement vector.

Equation 2 is represented in the following diagram for a two degree of freedom system.

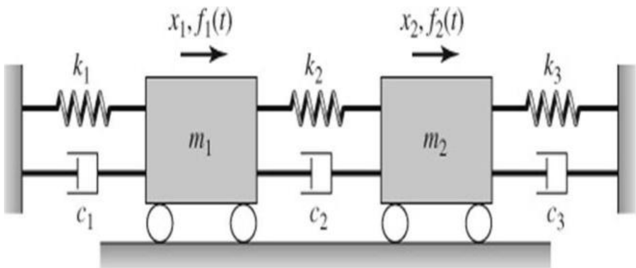

Fig 3. Two Degree of Freedom System

The objective of Modal analysis is to realize the frequencies and natural mode shapes of a structure during free vibration. By ignoring the dampening term in the equation of motion and assuming discrete values for mass and stiffness one can arrive at the m and k matrices.

$$[m]\ddot{x} + [k]x = F(t)$$

The frequency is found by transforming the resulting equation into an Eigensystem with λ as the eigenvalue which has the units of s^{-2}. λ is determined by using zero as the equal of the determinant of the Eigensystem

$$det[k - \lambda m] = 0$$

then solving for λ

$$f = \frac{1}{2\pi}\sqrt{\lambda}$$

Where

$$\lambda = \frac{k}{m}$$

Each mode shape corresponds to an eigenvalue which can be found by solving the eigenmatrix as a system of linear equations equal to zero. The resulting values are the displacements at the distinct locations of mass and can be combined to give the mode shape for that frequency. The larger the number of distinct points, the greater the accuracy of the model.

1.5Dampening

While not a significant portion of this project, damping plays an important role in the motion of a structure. It can be quite difficult to determine the damping of a structure; however, on a pedestrian bridge, it is primarily provided by the friction of the interaction with other structural components. Damping is crucial to the role of dissipating the vibrational energy in a structure and without it vibrations would not dissipate and continue indefinitely. All structures are subject to damping, however, as construction materials and techniques advance coupled with improvements to technology, damping appears to decrease. This decrease while unintentional, is creating a situation where frequencies are oscillating with higher amplitudes whereas previously said vibrations would dissipate more quickly.

Damping can be divided into 3 categories: critical damping, overdamping and under-damping. From Newton's second law motion equation:

$$ma + cv + kx = 0$$

The general equation of motion is given by the second order differential equation seen below

$$[m]\ddot{x} + [c]\dot{x} + [k]x = F(t)$$

Substituting the second order differential equation to an auxiliary equation for λ:

$$m\lambda^2 + c\lambda + k = 0$$

The roots of the quadratic auxiliary equation are

$$\lambda = \frac{-c \pm \sqrt{c^2 - 4mk}}{2m}$$

All three cases are illustrated in the figures below

$$c^2 - 4mk > 0$$

Overdamped

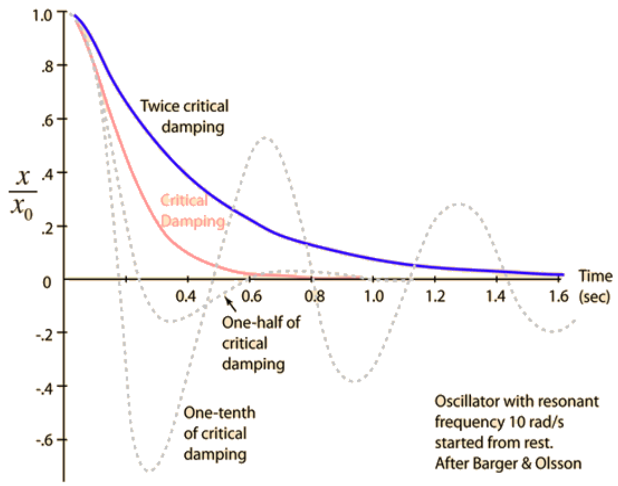

Fig 4. Overdamped Oscillator

Critical damping

$$c^2 - 4mk = 0$$

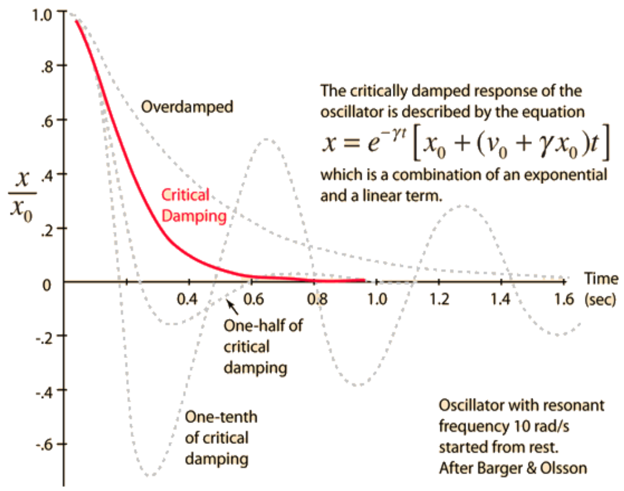

Fig 5. Critical Damping

Underdamping

$$c^2 - 4mk < 0$$

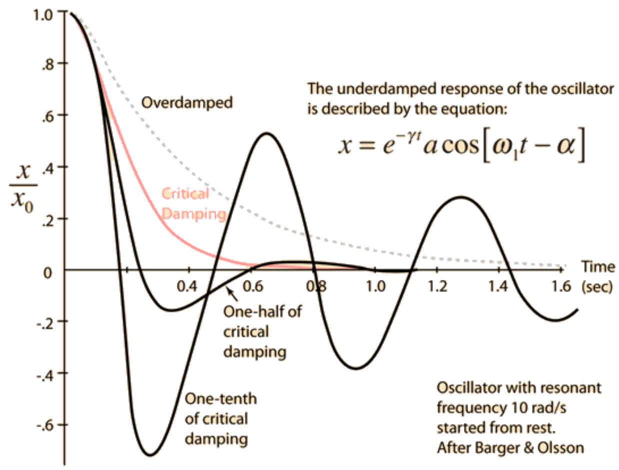

Fig 6. Underdamped Oscillator

Damping ratio expressed as ζ, is dependent on the viscous damping coefficient and critical damping. The damping ratio equation is noted as

$$\zeta = \frac{c}{2\sqrt{km}}$$

When the damping ratio is less than 1, vibrations disperse slowly, theoretically never completely dissipating. When the damping ratio is equal to 1, vibrations dissipate rapidly. This can be explained by the damper being strong enough to avoid the mass vibrating. The mass will return directly back to its original position. When the damping is greater than 1, the same phenomenon occurs as critically damped albeit at a slower pace.

Damping is particularly important when the dynamic load or excitation creates a frequency which is relatively similar to that of the natural frequency of the structure. When this occurs, structural damage is possible through resonance. Damping helps to prevent this resonance vibration, thereby potentially alleviating structural damage.

1.6Time and Frequency Domain

When the raw data was recorded for the pedestrian bridge on Blanchard Street in Thunder Bay, it was in the domain of time. This is due to the accelerometers which were used to collect the data, the base units were cm/s^2 with respect to acceleration, and time was recorded in seconds. As the project focused on frequency, a transformation from time domain to frequency domain was needed. This transformation converts acceleration to amplitude and time to frequency, allowing for frequency analysis. The Fast Fourier Transform is an algorithm used to convert time domain data into frequency domain data. It allows for frequency and mode shapes information without requiring mass, damping ratio, or stiffness however, it should be noted, this method is applicable for finite signal only. Any vibration pattern can be taken as a series of sinusoidal waves with different amplitudes and frequencies combined. Any periodic signal can be expressed as superposition of infinite harmonics using Fourier Series. For Aperiodic signals such as the vibration data obtained from the bridges, Fourier Integral can be used to express the signals. The Fourier integral can be expressed as

$$x(t) = 2\int_0^\infty A(\omega)\, cos\ \omega t\ d\omega + 2\int_0^\infty B(\omega)\, sin\ \omega t\ d\omega$$

While the Fourier Transform, coefficients are

$$(\omega) = \frac{1}{2\pi}\int_{-\infty}^{\infty} x(t)$$

$$(\omega) = \frac{1}{2\pi}\int_{-\infty}^{\infty} x(t)$$

Where $x(\omega)$ is Fourier Transform of $x(t)$

$$x(\omega) = A(\omega) + iB(\omega)$$

Fast Fourier Transform can separate the time-domain data into a series of harmonics, Time-domain data is then expressed in frequency-domain and each peak in FFT plots represent the significance of time-domain data at certain frequency. The following figure illustrates the difference between time and frequency domain.

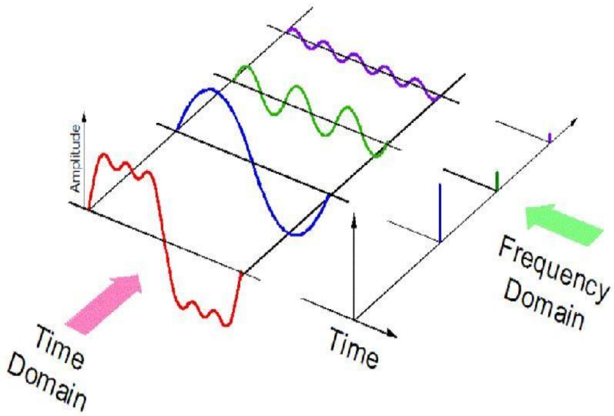

Fig 7. Time and Frequency Domain

In both instances, for the pedestrian bridge and the railway bridge, the Fast Fourier Transform was employed in MATLAB to present the recorded vibration data in the frequency domain. In the frequency domain, the major peaks from the signal corresponding to the natural frequencies of vibration of the bridge.

II.PEDESTRAIN BRIDGE

The pedestrian bridge used for this test is located in Thunder Bay, Ontario off Blanchard Street over McVicars Creek. The footbridge bridge met the criteria for the project and was selected for analysis with the City of Thunder Bay's permission. The bridge was composed of two I-beams fixed at both ends, steel struts spaced along the length of the bridge and a concrete deck. The pedestrian bridge spans 34 feet in length and approximately 6 feet wide. The figures shown below are three views of the pedestrian bridge.

Fig 8. Front view

The purpose of this experiment was to cause external excitations on the pedestrian bridge such as walking, running, and walking in synchronization to obtain vibration data. These activities are described in the following section. From the vibration data, dynamic analysis was performed to determine the structure's frequency and mode shapes. Secondly an S-Frame model of the existing pedestrian bridge was created to accurately predict the natural frequencies of the structure. The comparison between the predicted natural frequencies from S-Frame and those recorded from experimental data will govern the serviceability, as well as the accuracy of the finite element model.

Fig 9. Side View

2.1Vibration Testing

The vibration testing data was collected with the use of piezometer sensors and DAQ (Data Acquisition) collector. A total of six piezometer sensors were used for this experiment with three evenly spaced along the length of the bridge on either side, at 8.5 ft. The piezometer sensors are used to measure the acceleration of the bridge under loading. All six sensors were attached by a cable to a DAQ which was located on a stand near the end of the bridge, but not on it. The collector measures the dynamic signal acquisition module that can be utilized for either noise or vibration measurements. The collected vibration data is then converted into digital data for analysis in MATLAB.

Fig 10. Piezometer Sensor

The vibration data was measured (sampling frequency) every 5 milliseconds which is dictated by the Nyquist Theorem. According to this theorem sampling frequency is equal to one over double the natural frequency of the structure being monitored. Civil structures are generally understood to have a frequency of 100 Hz, therefore, the sampling frequency was set to 200 Hz. Figure 13 shown below consist of the layout of the sensors for testing and the dimensions of the footbridge. It is important to notice sensor three and four are located at the centre of the bridge. The importance of this will be discussed in the analysis of results section.

Fig 11. Data Acquisition Equipment

The pedestrian bridge used for testing is located in a residential area and consequently, is mostly subjected to light traffic. The different excitations the bridge was exposed to during the test were intended to represent the normal conditions under which the bridge is subject to. Each individual's unique characteristics (weight, height,) were recorded and incorporated as part of the analysis. During the testing, all individuals were required to count the number of steps they took to cross the bridge. This information was used to determine the stepping frequency and must be accurate because miscounting steps could have a noticeable change in the stepping frequency and may not match with S-Frame results.

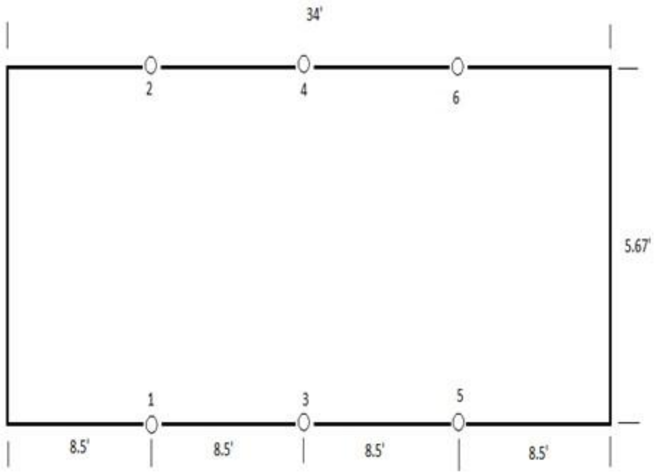

Fig 12. Pedestrian Bridge Sensor Layout

While the tests were being conducted, a stopwatch was used to manually record the time it took to complete each trial over the bridge. By relating the time and steps that were recorded, it is possible to obtain the stepping frequency. This is calculated by dividing the number of steps by the time (in seconds) required to travel across the bridge deck. A total of 26 tests were conducted on the pedestrian bridge. The description of the different tests conducted are described in the following table.

Table 1: Description of Excitations

Tests	**Description**
Test 1: Single walk	A total of ten tests were recorded in which a single person walked at a relaxed pace from one end to the other.
Test 2: Single running	A total of ten tests were recorded in which a single person was running at a fast pace from one end to the other.
Test 3: Group walking	Two tests were recorded in which a group of six people walked at a relaxed pace.
Test 4: Side by side in sync	A test was recorded in which two people walked in sync across the bridge.
Test 5: Side by side no sync	A test was recorded in which two people walked out of sync across the bridge.
Test 6: Single jumping	A total of two test were conducted in which a single person located at the center of the bridge jumped.

III.RESULTS

The vibration data was recorded in time domain and then converted into frequency-domain using the Fast Fourier Transform (FFT) algorithm through MATLAB. By using MATLAB, the acceleration data from each node was organized into vectors and displayed in frequency-domain. Amplitude (cm) is measured on the y-axis and the frequency (Hz) on the x-axis. These plots are shown below.

Fig 13. Walking Excitation

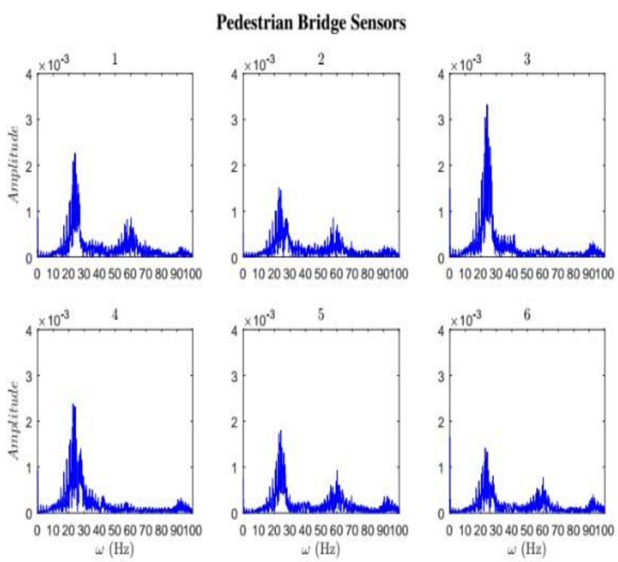

Fig 14. Walking Excitation Response

Fig 15. Single Running Excitation

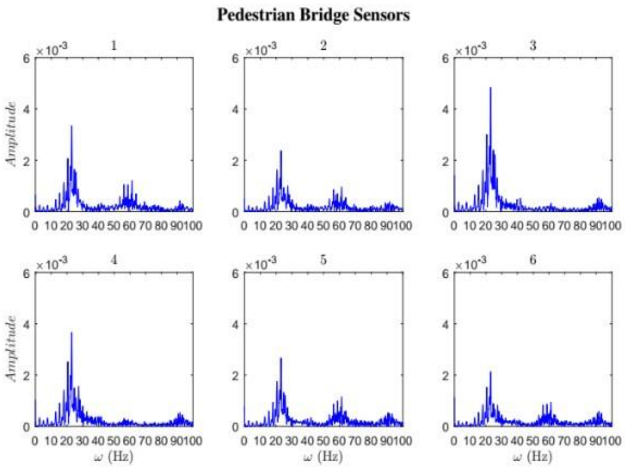

Fig 16. Single Running Excitation Response

Fig17. Group of Six Walking Excitation

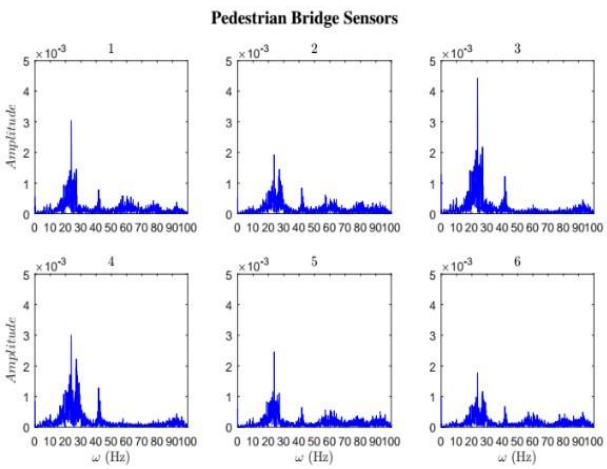

Fig 18. Group of Six Walking Excitation Response

Fig 19. Group of Two Synchronized Walking Excitation

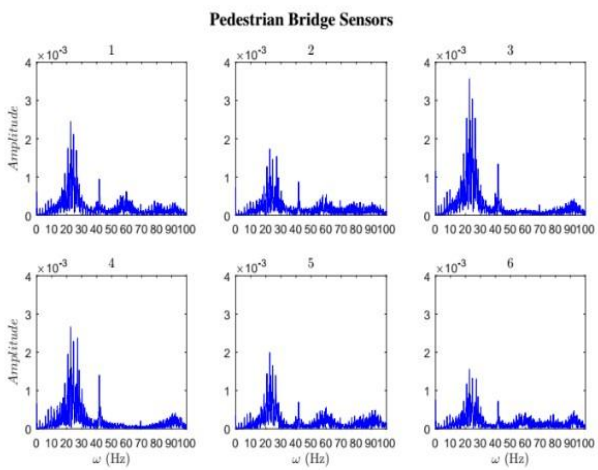

Fig 20. Group of Two Synchronized Walking Excitation Response

Fig 21. Group of Two Unsynchronized Walking Excitation

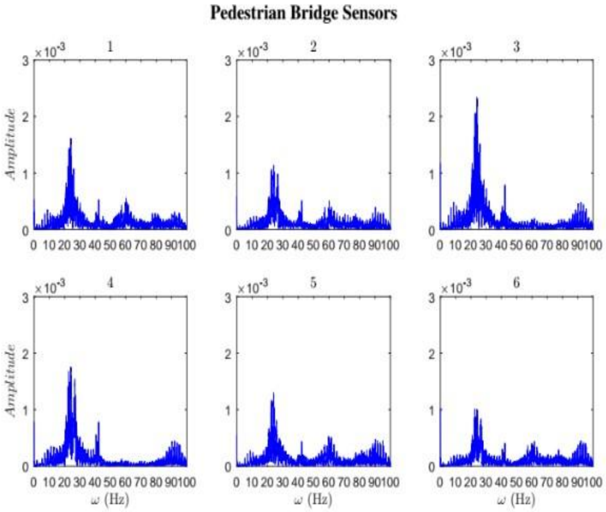

Fig 22. Group of Two Unsynchronized Walking Excitation Response

Fig 23: Jumping Excitation

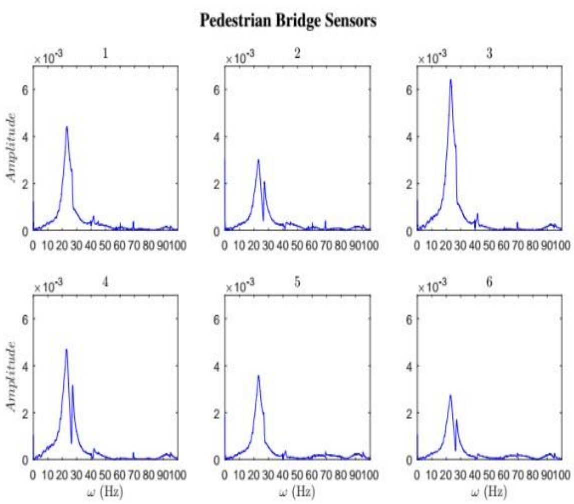

Fig 24. Jumping Excitation Response

IV.ANALYSIS OF RESULTS

All of these plots are examples of the frequency data obtained from the three forms of excitations (walking, running and jumping). Each figure has six subplots corresponding to the six piezometer sensors used for testing. By analysing the results of each plot, the highest amplitude happens at the first mode. This can be explained by the first mode having the highest energy on the structure. It is also important to notice that sensor three and four, have the highest amplitudes because they are located at the mid-span of the bridge. This can be explained by sensor three and four being the farthest away from the supports. When comparing sensor three and four amplitudes it must be noted that sensor three has a slightly higher amplitude throughout the tests, some reasons this could have occurred are the sensors may have been placed at different distances from the edge of the bridge or from the exact mid-span of the bridge in reference to length. Another reason might be the excitation being applied to the bridge was closer to sensor three. The difference between sensor three and four could also imply that there is damage or inconsistency of the bridge at the mid-span.

From the frequency-domain plots, the frequencies of the bridge were determined by analysing the different peaks where the energy was the highest. The peak frequencies at each mode were analysed in MATLAB using the "data cursor" tool which allows for exact results. Each test was analysed and recorded in order to compare with the other excitations for an accurate representation of the structure's frequencies under different excitations into Table 2.

4.1Frequency Analysis

Table 2: Frequency Readings for Each Test (Hz)

	in Hertz					steps/time	
test	ω1	ω2	ω3	ω4	Stepping Frequency FFT	Stepping Frequency	
1	24.85	41.51	60.20	91.15	2.15	2.41	
2	23.89	41.74	58.83	91.25	1.95	2.17	
3	23.11	41.43	57.44	90.49	1.97	1.90	
4	23.38	41.59	59.58	91.47	1.98	1.93	
5	22.67	40.12	59.99	92.49	2.16	2.26	walking
6	23.77	41.91	59.26	92.81	2.00	1.96	
7	23.47	41.33	59.34	91.97	1.95	1.90	
8	22.67	41.82	57.13	92.57	2.10	2.02	
9	23.48	41.43	58.65	92.12	2.06	1.83	
10	25.97	41.35	59.79	92.64	1.95	1.75	
11	22.13	40.05	58.76	90.99	2.81	2.05	
12	21.98	41.04	61.37	92.18	2.65	1.81	
13	23.14	41.87	57.55	90.54	2.58	2.01	
14	23.46	38.53	58.28	92.22	2.64	1.86	
15	22.65	41.36	59.32	91.87	2.88	2.22	
16	22.90	40.43	57.96	91.34	2.74	2.28	Running
17	23.10	40.34	60.51	91.52	2.60	2.32	
18	22.08	41.47	58.69	91.70	3.20	3.36	
19	22.10	41.59	60.11	92.04	2.83	2.62	
20	23.20	41.18	56.57	91.34	2.69	2.34	
21	23.81	41.71	57.70	92.82	1.84	1.49	Group
22	22.78	41.68	59.63	91.27	1.90	1.25	
23	22.65	41.66	59.61	92.13	1.97	1.82	In sync
24	23.77	42.28	60.25	91.42	1.82	1.16	No sync
25	23.03	42.40	60.34	92.46	-	-	
26	24.57	41.12	57.88	91.07	-	-	Jumping

When analysing the results from each excitation it must be noted that the first four modes were located at very similar frequencies. When comparing the modes of the 26 different excitations and all the recorded data from the six different sensors, an executive decision was made to take the average value of each set of sensors to create a structured table of all the data sets. From the tests applied to the pedestrian bridge ω1 through ω4 represent the first four modes that occur. The different colours in Table 2 represent the different excitations. The last two columns represent the stepping frequency recorded from the plots and the calculated stepping frequency from the experiment.

The first mode of the pedestrian bridge shows that there is a similar frequency that ranges between 22 - 25 Hz, but when looking at Table 2, you can see that the amplitude substantially varies based on the excitation applied to the bridge. Comparing the average amplitude at the mid-span of the first mode, the 10 walking excitation tests have an amplitude range of 0.00077 and 0.00432 cm. There are many contributing factors that can cause this range of amplitude. For example, the weight and stepping frequency of each individual will vary. This variance will also affect the stepping frequency of the 10 trials of people running the span of the bridge. The range of amplitude that occurs during the running excitation is from 0.00136 to 0.00805 cm. This shows that using the same weight but applying a higher stepping frequency will result in a larger vibration within the structure. It was observed that the difference between the walking and running excitation for the first mode is almost a 100% increase in amplitude.

The second mode shows a range of frequency between 38 - 42 Hz for the tests conducted, while the walking excitation range of amplitude is now 0.00022 to 0.00115 cm, which has drastically decreased from the first mode. Also, the running excitation has decreased at a similar rate based on the stepping frequency remaining the same. The change between the modes is due to the amount of energy provided at that specific frequency and it is more efficient to have a decrease in mode peak compared to an increase.

Comparing the trend of the modes as they appear in the frequency analysis, it should be noted that the peak values of the following modes also continue on a downward slope. Mode 3 has an average frequency at 59 Hz, has peak values that range between 0.00017 to 0.00091 cm with respect to amplitude for walking, and 0.00024 to 0.00194 cm for running. The first three modes of the structure consist of the majority of the energy created from these tests.

From the experiment, the walking frequencies range from 1.75 to 2.41 Hz whereas the running frequencies range from 2 to 3.36Hz. Generally, individuals walk with similar step frequencies due to similar physical characteristics with slight variances. Stepping frequencies are influenced by the purpose of the movement and the traffic intensity upon the structure. Theoretically, the stepping frequencies ranges from 1.25 to 2.3 Hz according to Human Induced Vibrations on Footbridges. Our observed frequencies fall almost entirely within this range.

The vibration data will be compared with the predicted frequency from the finite element model in order to determine the accuracy of bothRMS frequencies. This will be detailed in a following section (Finite Element Modeling).

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4.2RMS Data

The Root Mean Square (RMS) is one methodology that may be used when doing analysis on a vibration signal, it measures the peaks of a harmonic wave and converts them to a positive average value. It is particularly useful when performing data analysis and detect the trend of data to collect meaningful findings of the vibration characteristics. If the signal was being determined based on the average of these values, the positive/negative wave peak values would cancel each other out creating an average value of zero, RMS method can avoid this problem and help the designer with the data analysis.

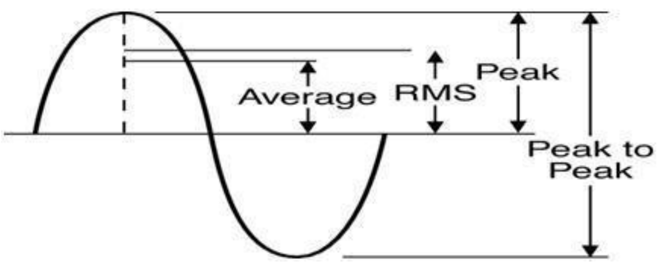

Fig 25. Theory of RMS

In order to calculate the RMS, the vibration data values of the wavelengths must be squared and these squared values are averaged over a certain time period. This time period interval must be at least one period of the wave in order to arrive at the correct value. Once this single number is calculated, the square root of this number is the RMS value of the signal.

$$= \sqrt{\left[\frac{1}{n}\left({x_1}^2 + {x_2}^2 \dots + {x_n}^2\right)\right]}$$

where *n* is the number of steps in time.

The RMS value is the most significant measure of amplitude because it takes into account the time history of the wavelength and gives an amplitude value. The RMS amplitude value indicates the vibration energy on the structure. A higher RMS amplitude value corresponds to increased vibrations on the bridge, while a lower RMS value will indicate less vibration in the structure during excitation. With this observation, we can perform data analysis for different pedestrian bridge tests summarized below.

4.3RMS vs Stepping Frequency

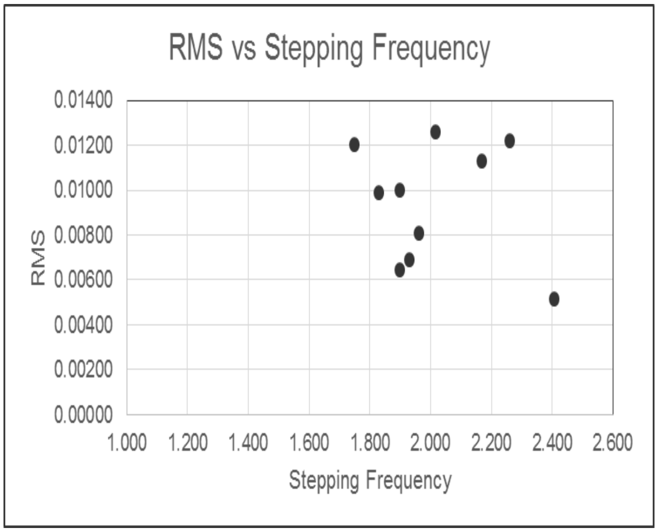

Fig 26. RMS vs Stepping Frequency

Referring to Figure 27, these values are based on the average RMS at the mid-span where the amplitude is most significant (sensors three and four), the range of RMS for the amplitude is between 0.005 and 0.013 for the 10 walking excitations. These values were quite low, and were attributed to the weight and speed of the pedestrians walking across the bridge applying diminutive vibration to the structure. According to Equation, the frequency of the structure changes according to the mass of the structure along with the pedestrians. Since the mass is the denominator of the equation, greater the mass will lead to a lower frequency. For this experiment, the mass of the pedestrians is very small compared to the mass of the footbridge. Thurs, the frequencies from the pedestrian bridge test are low and the same for the amplitude based on Equation 2. Typically, as the stepping frequency increases, the dynamic loading being applied to the bridge will intensify, the highest stepping frequency was recorded to have the highest RMS value. There is one discrepancy with this data set at 2.4 Hz which created an outlier. It was assumed an error occurred either in the participant's number of steps recorded or with the time logged on the stop watch and would explain the unanticipated result.

4.4RMS vs Running Frequency

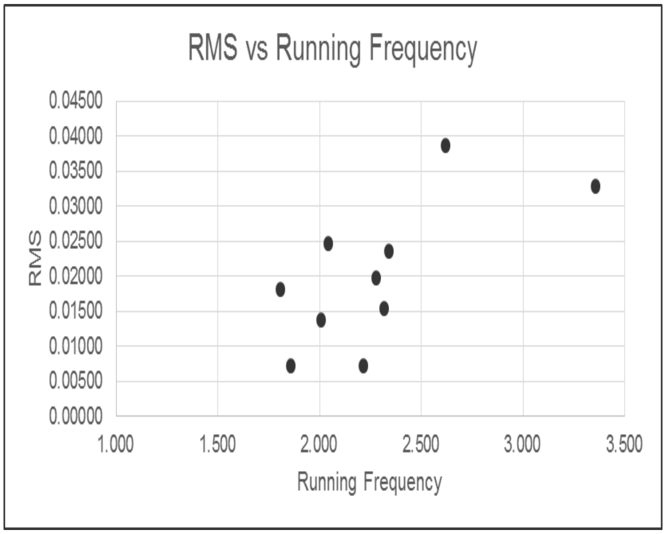

Fig 27. RMS vs Running Frequency

The RMS values regarding the amplitude from the running excitations are shown in Figure 28. The RMS is between 0.007 and 0.038 for the 10 running excitations. The RMS values have a positive trend-line when compared to the running frequency, and is an accurate representation of what should transpire between RMS and running frequency. Higher the frequency, greater amount of energy is exerted on the bridge resulting a more significant forced vibration. When comparing the stepping frequency with the running frequency, it should be noted that the running frequency will have a larger peak in RMS. This is not entirely unexpected as running excitation is accompanied with increased velocity and creates vibrations with greater amplitude on the pedestrian bridge. There is an outlier for this experiment plot at the running frequency of 3.4 Hz, the RMS at this point is lower than the one at 2.6 Hz. The 3.4 Hz is the highest frequency among all of the bridge test participants and the highest standard variation compared to the rest of the results which indicate a potential error. The high running frequency results in more steps at a given time which means high running speed; with the higher speed, the time for the dynamic loading on the structure is reduced which can explain the lower amplitude RMS.

4.5RMS vs Weight

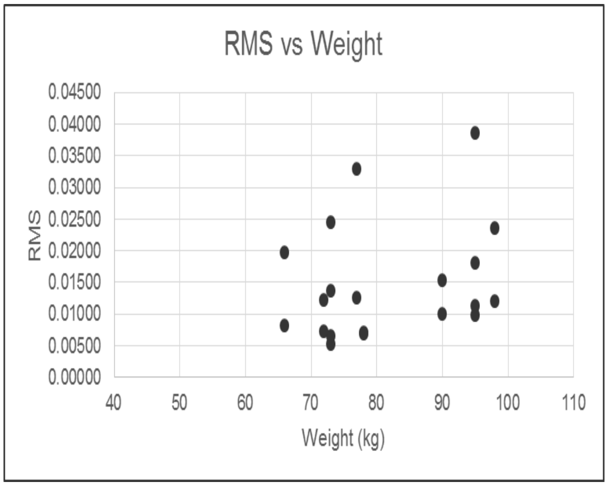

Fig 28. RMS vs Weight (kg)

The weight and height of each individual participating in the data acquisition for the different excitations were recorded and utilized to create this plot. The weight of the individual ranges from 65 to 98 kg. It can be noted that the greater the weight of individual participant the greater the amplitude the structure vibrates since the body weight from the pedestrians constitutes the major part of the dynamic loading on the bridge. Thus, the energy for the dynamic vibration of the bridge is directly related to the weight of the pedestrians. By analyzing the graph, the trend is upward creating a positive linear slope. Therefore, with an increase in weight cause an increase in RMS.

4.6Finite Element Modeling

To accurately predict the vibration data and compare and verify the vibration characteristics of the pedestrian bridge, a Finite Element Model (FEM) is required. The most accurate and convenient way to conduct FEM modelling is to construct a bridge model in S-Frame. This powerful software can not only perform static linear analysis but also dynamic analysis which is beneficial for this project.

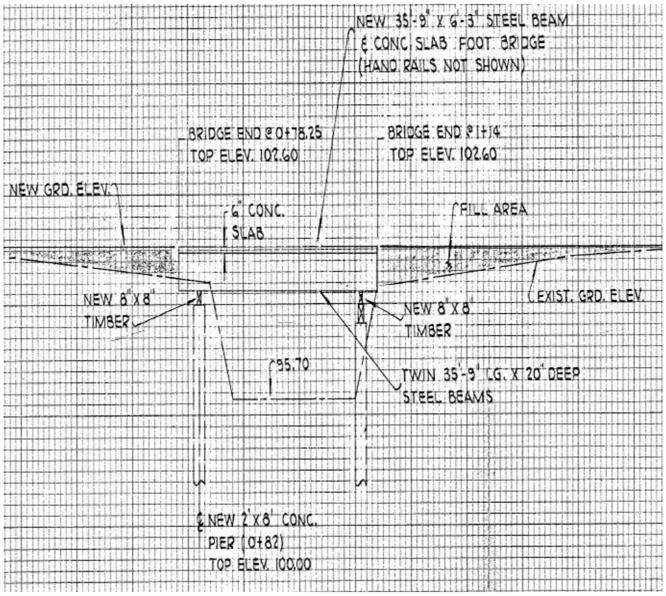

Figure 29: Existing Drawing (Elevation View)

Before the finite element model can be built, the structure needs to be idealized and simplified to reflect the nature of this bridge. This bridge is a slab-on-girder structure, with the girders sitting on timber bearings which allow the bridge to move horizontally. The superstructure rests on two concrete piers on each side of the bridge acting as abutments and foundations. Due to the timber bearing, the superstructure is not integral with the substructure, thus the bridge would not be considered a rigid frame structure. From the drawing, the main structural component is comprised of twin steel I girders, each with a depth of 508mm (20"). The bridge deck is cast integral with the steel girders, and concrete was cast flush with the bottom of the girder flange. According to onsite inspection, the concrete deck is supported by a thin steel joist deck between the two girder lines. The thickness of the steel deck is estimated as 10 mm. The finite element model for the vibration analysis can be visualized as two steel girders supporting a composite bridge deck which consist of 150 mm concrete deck and 10 mm steel plate.

As the bridge deck section is not a standard section as defined in S-Frame, custom design for the deck section was required to properly model the bridge. Using S-Calc, the project team managed to customize design the section to represent the existing condition of the bridge. It is assumed by the project team that the 150mm concrete deck was cast integral with the steel joist deck thus creating a composite section of concrete and steel materials. The property data for the section is then imported to the S-Frame model to form the bridge deck as part of the structure.

Fig 30. Modelled Bridge Deck Section

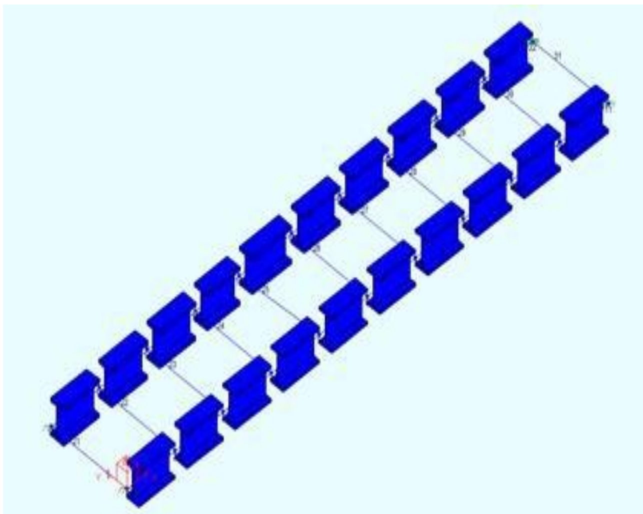

Fig 31. Overview of the Bridge (Finite Element Model)

From the CISC standard steel section, W460X260 is chosen to model the existing pedestrian bridge as this section has a depth of 508 mm, close to what is depicted in the existing drawing. Other dimensions such as flange width and web depth from the field measurement are similar to the specification of W460X260 as well, thus the team was confident about the accuracy for the girder modelling.

The Finite Element Model was constructed as 10 segments of sections in S-Frame to achieve the most accurate results, with each segment dimensioned as 1728 mm X 1090 mm, the entire model length came to be 10.9 m. All the segments are connected continuously to represent the infinite degrees of freedom of the bridge. From the onsite inspection, the girders and bridge deck are rested against both abutments and earthwork which restrains the superstructure from moving in x-axis direction independently, it is noted that the bridge can move freely with respect with the pier foundations which are designed to reduce the bending moment on the piers. For the dynamic analysis purpose for the project, the supports are considered as fully rigid for the superstructure.

With the pedestrian bridge replicated in S-Frame based on the above-mentioned assumptions and analysis, the model is analysed under Unstressed Vibration analysis with an Eigenvalue of 50 to generate the structure's mode shapes and natural frequencies. The Unstressed Vibration analysis is suitable to analyse light bridge types such as pedestrian bridges and temporary truss bridges. The size, weight, load and member composition are factors considered when selecting the appropriate analysis method. For this bridge, the size is small with only 10.9 m and 1.7 m wide while the structure itself is rigid enough to withstand wind load from the horizontal direction due to the two large steel I girders. Also, the horizontal sway for this structure can be considered as minimum. The self-weight of the bridge is relatively large compared to timber deck pedestrian bridges, and the stress load applied on the deck is snow load and pedestrian traffic live load, which are generally short term and small compared to the self-weight. From the existing drawing and field inspection, the structural members are all linear components as there is no proof for the existence of structural damper systems or any other non-linear structural elements. Thus, the selection of using Unstressed Vibration is verified for this particular structure.

From the dynamic analysis of the Finite Element Model, three major and one minor natural frequencies is predicted from the first 100 Hz of the pedestrian bridge. The detailed results are summarized below

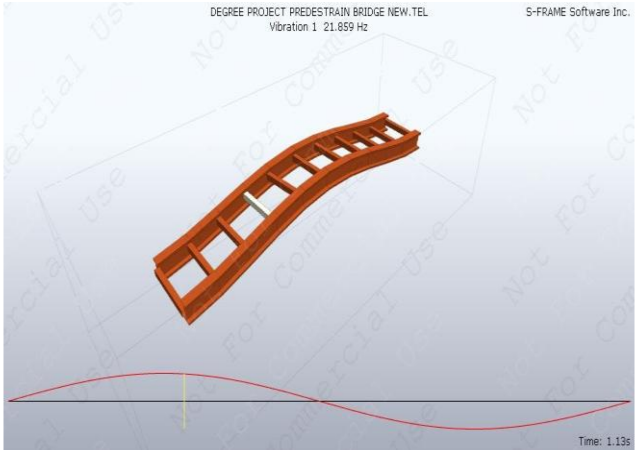

Fig 32. First Mode Shape (Vertical), at 21.86 Hz with energy of 77.34%

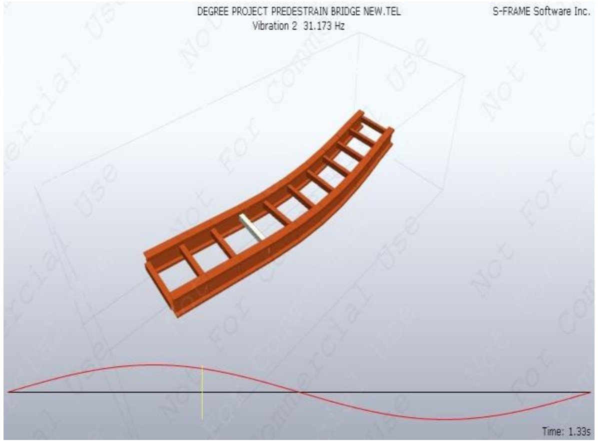

Fig 33. Second Mode Shape (Horizontal), at 31.17 Hz with energy of 85.5%

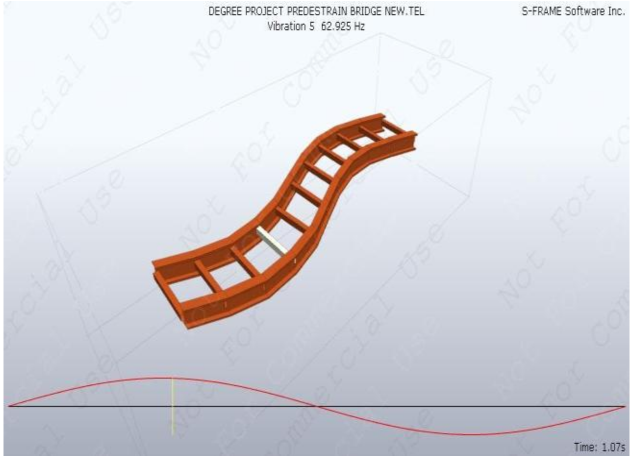

Fig 34: Third Mode Shape (Torsional), at 62.92 Hz with minimum energy

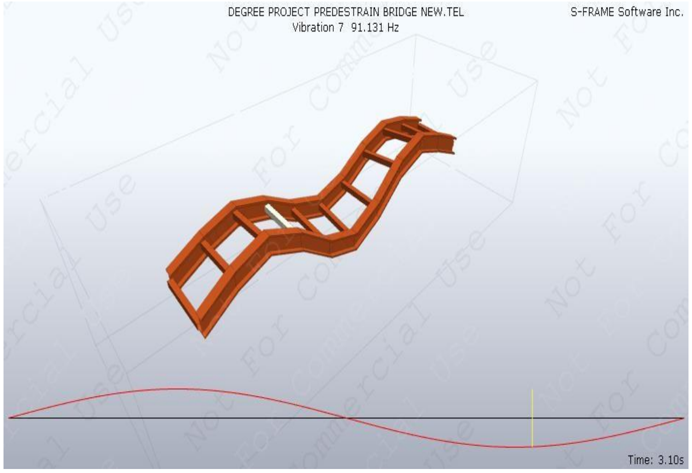

Figure 35: Fourth Mode Shape (Vertical), at 91.13 Hz with energy of 15.34%

The results illustrate the natural frequencies and mode shapes with different directions. From the results, it can be observed that the most significant vibrating frequency with the highest energy and deflection amplitude is the first natural frequency. The first natural frequency generally has the lowest frequency and highest mass percentage. For this bridge, the first mode has a frequency of 21.86 Hz oscillating with 77.34% mass vertically. There is a significant drop in terms of the percentage mass on the fourth mode which has only 15.34% which proves that in order to vibrate at a higher node a higher energy is required to initiate the forced vibration.

Compared with the results from the physical test summarized in Table 2, one can see that the natural frequencies and mode shapes predicted from the FEM agree with the vibration analysis data from the onsite testing:

Table 3: Average Frequencies from Test with FEM Results

Avg. frequency	ω^1	ω^2	ω^3	ω^4
Walking	23.73	41.42	59.02	91.90
Running	22.68	40.79	58.91	91.57
Jumping	23.80	41.76	59.11	91.77
FEM	21.86	31.17	62.92	91.13

The above table summarizes the average natural frequencies from the vibration testing and results predicted from the model. In this table, it is easy to identify how similar the predicted results are with the actual results from the experiment. For the First, Third and Fourth natural frequencies, the differences are negligible.

The difference between the predicted results from the actual test results can be ignoredas the individual data and the trend of the analysis is agreeable. The only significant difference from the FEM model is the horizontal mode shape, the natural frequency for the third mode shape is 31.17

Hz from the FEM while for the pedestrian bridge test it is 40 Hz. The contributing factors for the difference can be: First the peak identifying from the FFT plots from the walking and running test, some of the plots have peaks at around 30 Hz while all the FFT plots have natural frequencies as 40 Hz thus the second major natural frequency is identified as 40 Hz for the structure. Second, from the onsite inspection, there are 3 small diameter steel struts braced between the two girder webs which are not shown on the existing drawing. The additional steel struts can affect the horizontal stiffness of the structure making the structure more rigid in horizontal direction. With the higher horizontal stiffness, the natural frequencies for the structure will become higher which explains the lower horizontal frequency predicted from the Finite Element Model.

V.CONCLUSION

The method of comparing the FFT plots for the railway bridge has proven to be a feasible technique to determine the train type causing the recorded vibrations. With the recorded vibration data, it is a reliable resource for researchers to identify more information other than train types. The information can be used to analyze the weight of the trains and with proper modification or more time lapse of the vibration data, the dynamic wheel loads of the trains can be studied as well. The vibration data collected from the railway bridge can be used to monitor the health of the structure, and the data collected each year is a sound source with which to check and confirm the deterioration of structure members.

For thepedestrian bridge, using Finite Element Model to predict the mode shape of the structure was deemed successful, and mirrored the results acquired from the physical tests. When designing a new bridge, designers usually build finite element models to perform structural analysis. Dynamic analysis to obtain the natural frequencies and mode shapes of the structure arealso performed,recorded for later structural health monitoring purposes and future rehabilitation resolutions. Whenever a bridge inspection is performed, a simple vibration test, similar to the tests performed during this degree project, can be utilized to obtain current vibration data. This would allow engineers to determine the amount of the stiffness reduction and section loss that has occurred.

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