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Structural Health Monitoring by Identification Dynamic Properties and Load Rating Factor at Multi-span Prestressed Concrete Girder Bridge

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Abstract – It is crucial to perform routine bridge maintenance in order to evaluate the structure's current state. As a result, it is possible to guarantee that the bridge structure can offer services that are both comfortable and secure. The bridge structure being able to reach the service life as planned is another goal that can be accomplished. Visual inspection or the use of some currently popular sensors can be used to monitor the condition of the bridge. The dynamic properties of a structure including modal frequency and mode shape will be used to determine the structure's present and potential future conditions. Using a velocitymeter, vibration data collection is conducted as the first step. The next step is analyzing data to determine natural frequency. The fundamental frequency of the Tugu Suharto bridge structure in Semarang was determined to be 3.995 Hz. Future bridge structure condition monitoring can be done using frequency data and finite element model. The condition of bridge infrastructure in the future for one city is an important thing that must be considered. Some bridges are classified as structurally deficient, and many bridges are nearing the end of their design lives. The next generation of Semarang highway bridges is currently being designed and built, but existing bridges still need to be maintained through proper inspection and load rating. In order to incorporate structural modeling, instrumentation, and nondestructive testing into the design, construction, and management of bridges, this study proposes an objective load rating protocol. Using information gathered from structural health monitoring (SHM), a baseline structural model is developed and verified. The load rating factors of the bridge are then determined using the structural model under both real-condition and simulated damaged conditions.

Keywords: girder bridge, vibration measurement, frequency, mode shape, FE model

Introduction

The use of vibration methods in data collection is still required for non-destructive evaluation of structures and the detection of damage that occurs in a structure. The distribution of the structure's stiffness will change permanently as a result of damage. By keeping an eye on the structure's vibrations, these changes can be found. Numerous studies have used this dynamic property to observe the structural conditions because stiffness and mass have a relationship with natural frequency, mode shape, and damping ratio. The vibration method is a structural study that is currently under development and can help in identifying and locating structural damage. The results are complementary because vibration data has shown to be very useful when used in conjunction with other monitoring systems.

Numerous vibration-based studies on structural conditions have been conducted, including those on railroads (Kusumawardani *et al.*, 2019; 2022) and cable-stayed bridges (Wu *et al.*, 2022; Chen *et al.*, 2023. Numerous studies

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relating to algorithms have been conducted for operational modal analysis in monitoring bridge structures. A new SSI methodology was also developed by Wu (2016a) based on a hierarchical sifting procedure and an alternative stabilization diagram. It has been demonstrated that using this improved SSI algorithm, precise modal parameters of cable-stayed bridges (Wu *et al*., 2017; Wu *et al*., 2016b; Wu *et al*., 2017b; Wu *et al*., 2019) and office buildings (Wu *et al*., 2017a; Wu *et a*l., 2019) can be steadily determined from on-site or long-term vibration measurements to carry out relevant investigations for SHM. A faithful FE model can typically be developed first with design or construction drawings, followed by careful calibration, using the reliable modal parameters obtained from OMA. For instance, a complex mechanism consisting of two tie-down devices, three rubber bearings, two shear keys, and twelve longitudinal cables for the connection between the bridge girder and the abutment at one end was taken into account by adding a linear spring and a rotational spring in the study for foundation scour evaluation of a cable-stayed bridge (Chien *et al*., 2014).

Several methods have been developed and successfully applied in the last decades. In the frequency domain approach, several methods are known, such as peak-picking, frequency domain decomposition, and maximum likelihood identification. In the time domain such as eigen system realization algorithm, autoregressive moving average model, and stochastic subspace identification (SSI). Due to its solid mathematical foundation and excellent applicability in various linear systems, SSI is attracting more and more attention thereby increasing its popularity among all these methods. With its advantages in mathematical accuracy and numerical convergence, the application of SSI techniques to large-scale civil structures still encounters several obstacles. The most frequently encountered problem is that process noise and measurement noise from structures generally do not meet all the zero-mean, white-noise, and stationary assumptions which are the basis for SSI reduction. To overcome this difficulty, SSI techniques are usually associated with stabilization diagrams to perform practical analysis of measured signals from civil structures. So this research uses the Stochastic subspace identification (SSI) method which is equipped with a stabilization diagram and hierarchical shifting process.

Assumptions (such as those regarding the support condition of the components being rated) and documented data (such as design drawings) are needed for the analytical approach of load rating. Physical load testing is desired to provide the necessary information and to produce a trustworthy load rating when the assumptions are subject to verification or the information is not readily available. Here, we use the term "diagnostic load testing" to describe such physical load testing. Researchers have conducted a number of load rating studies and published the results in a number of papers. By using field-measured data instead of specific information or the underlying assumptions required for analytical rating, diagnostic load testing can lead to more accurate load ratings for bridges. To get more accurate load ratings for the fascia beams, diagnostic load testing was used (Fu et al., 1997). A posted, three-span, steel-girder and slab bridge is put through an experimental load test. The bridge was intended to have three straightforward supported spans, each of which was made up of nine non-composite steel girders in cross section. The bridge's measured response was used to create a finite-element model of the main span. When the outcomes of analyses using the numerical model were compared to the results of the measurements, it was discovered that they were remarkably similar (Chajes and Mertz, 1997)

Typically, a bridge's condition is assessed based on visual inspections, and bridge-load ratings are computed using relatively straightforward analytical techniques without the use of site-specific, live-load, bridge-response data. This load testing, however, is only given priority for bridges that are thought to require immediate repairs due to financial limitations. Based on this circumstance, Chages and Shenton (1969) devise techniques for using nondestructive evaluation methods to increase the accuracy of bridge capacity evaluation.

The study by Akgul *et al.* (2004) presents the findings of an investigation into the relationship between the rating and dependability of a group of 14 bridges in an existing network of bridges. This investigation is based on cutting-edge techniques to assess each network bridge's dependability. Based on the type of bridge, rating factors for various bridge groups are identified, and the mean group rating factor is used to compare bridge groups. In this study, Estes and Frangopol (2005), the same highway bridge is subjected to both a load rating analysis and a reliability analysis. The results are compared, and each approach's benefits and drawbacks are discussed.

It is explained in the research by David *et a*l. (2012) how finite-element (FE) analysis software was created specifically for the load rating of flat slab bridges. By comparing results from commercial FE software predictions made under believable loading scenarios, the formulation and convergence of the FE software were confirmed. A new paradigm in bridge design, construction, and management that incorporates structural modeling,

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instrumentation, and nondestructive testing is exploited in the paper by Bell *et al.* (2013) to propose an objective load rating protocol. Using information gathered from a controlled static load test known as structural health monitoring (SHM), a baseline structural model is developed and verified. The load rating factors of the looks at how modern technology is applied to bridge load ratings. With the development of data acquisition and sensor technology, diagnostic testing to assess bridge load ratings has gained popularity. To quickly load test and raise the load rating of bridges, wireless data acquisition and primarily magnetic and reusable strain gauges will be used.

A method by Khateeb *et al.* (2018) has been developed to compute continuous rating factors over the course of a bridge's life by combining data from SHM systems with traditional load rating equations. The SHM-based ratings take into account both the actual in-service response caused by live loads as well as slowly varying changes caused by thermal effects and deterioration. A nondestructive method for load rating reinforced concrete (RC) slab bridges without structural plans is suggested in the paper by Bagheria (2018). In the suggested method, numerous finite element analyses were carried out to describe the modal characteristics of a sizable population of bridges with various geometric features. An artificial neural network model that forecasts the flexural rigidity of a bridge based on the measured modal frequencies derived from vibration testing was then developed using the findings and geometric inputs.

Materials and Methods

Description of Bridge

Tugu Suharto Bridge spans the Kaligarang River (Figure 1) and is situated at 7° 01' 01.57" S and 110° 23' 16.36" E in the Gunungpati district, connecting a number of important roads in Semarang and a number subdistricts. With the strategic importance of the bridge, maintenance is obviously required to extend the bridge's life while continuing to provide expected traffic services. The bridge is precast concrete girder type I, height 1.7 meter with four tendons each containing seven wire strand type prestress cables and a parabolic tendon model. The floor slab's concrete is made with K-350 quality and U-32 quality reinforcing steel. It has three spans, each measuring 5.3 meters in width and 87.4 meters in length overall. With a bridge height of 9.5 meters, the distance from pillar to pillar is 31.8 meters. Due to its narrow traffic width of only 4 meters, the bridge only has 1 lane for 2 directions. Figures 2 and 3 provide for top view and side view, respectively. Figure 4 is shown the cross section of the bridge.

Figure 1. Location of study area

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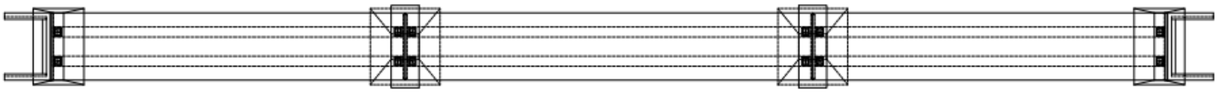

Figure 2. Top View

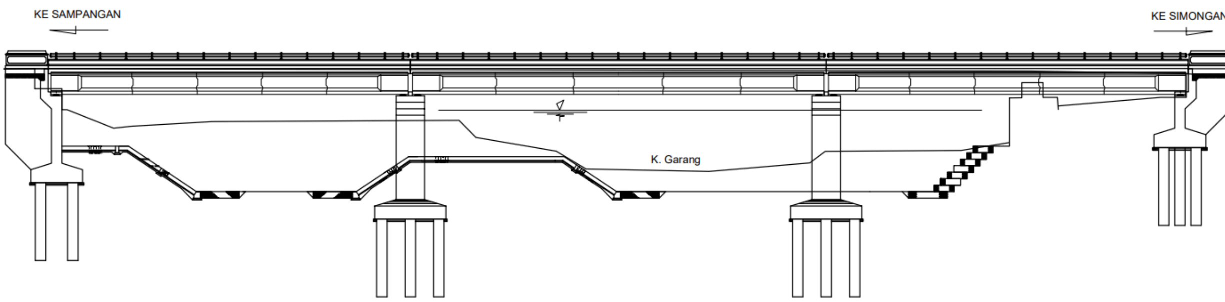

Figure 3. Side view

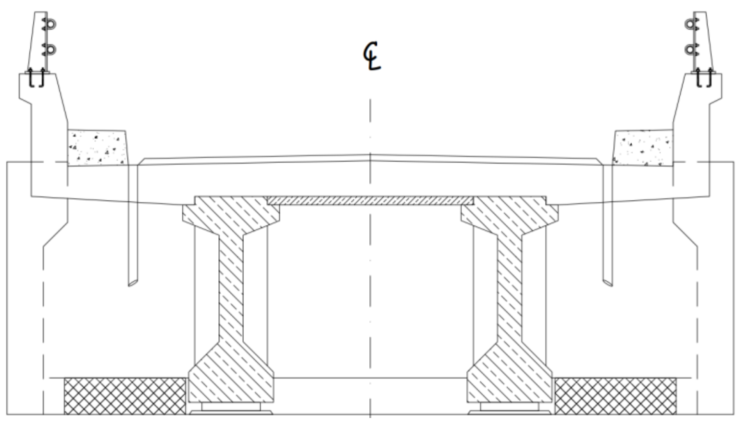

Figure 4. Cross section

FE Model of Bridge

The finite element model of bridge (Figure 5) can be constructed refer to information from as built drawing. The floor plate of the bridge is made of K-350 concrete and measures 5.3 m in width by 0.25 m in thickness. D16-200 reinforcement is used in the longitudinal direction and D16-100 reinforcement in the transverse direction at the top of the plate. D16-200 reinforcement is used in the longitudinal direction and D16-150 reinforcement in the transverse direction at the bottom of the plate. Two I-shaped girders (Figure 4) with dimensions of 0.8 m in width and 1.7 m in height are used for the girders and are made of K-500 concrete quality. There are four tendons with a parabola-shaped path inside the girder. For tendon 1, 2, 3, and 4, there are 12, 9, 7, and 7 steel wires, with a cross-sectional area of 12.7 mm2 for each steel wire. The Tugu Suharto bridge is supported by two piers, each of which measures 6 meters in width at the bottom, 6.5 meters in width at the top, and 8 meters in height. The circular pillars have a 2 m diameter and are made of K-350 concrete with U-32 steel reinforcing. Two layers, one measuring 60D32 for the first layer and the other 24D32 for the second, make up the main pillar reinforcement. D16-100 is used for shear reinforcement in the support section, while D16-150 is used in the field area.

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Figure 5. Finite Element Model

Ambient Vibration Measurement

As was mentioned in the initial explanation, ambient vibration measurement was used to collect bridge vibration data while the bridge was operating normally. Three sensors (Figure 6) of the Tokyo Sokushin brand velocity meter are placed at distances (Figure 7) of 7.95 m (1/4 span), 15.9 m (1/2 span), and 23.85 m (3/4 span). This sensor has a length of 55 mm, width 69.5 mm, height 72 mm, and weighs 270 grams. With its relatively small size and fairly light weight, the velocity sensor used is very supportive for collecting data in the field. Additionally, the low frequency range is between 0.1 Hz to 70 Hz, with a high resolution of 1.5 x 10-6 cm/s^2 to 2 x 10-6 cm/s^2. This velocity meter tool can be used for substructure surveys, micro-tremors, micro-earthquakes, and various experiments related to vibration. To collect data (data logger) use SPC52 which is equipped with several applications that can convert speed into acceleration and displacement over time. It can also do calculations to display waves versus time. This data logger is also equipped with PWave32 software which can display FFT spectrum, H/V spectrum, spectrum response, signal filtering process, etc The placement of these sensors are determined by the total number of sensors and refer to the results of the mode shape and and modal frequency from initial FE model prediction. As shown in Figure 8, a data logger using the Tokyo Sokushin SPC52 is also necessary for this measurement. Data was gathered at a sampling frequency of 200 Hz for 10 minutes.

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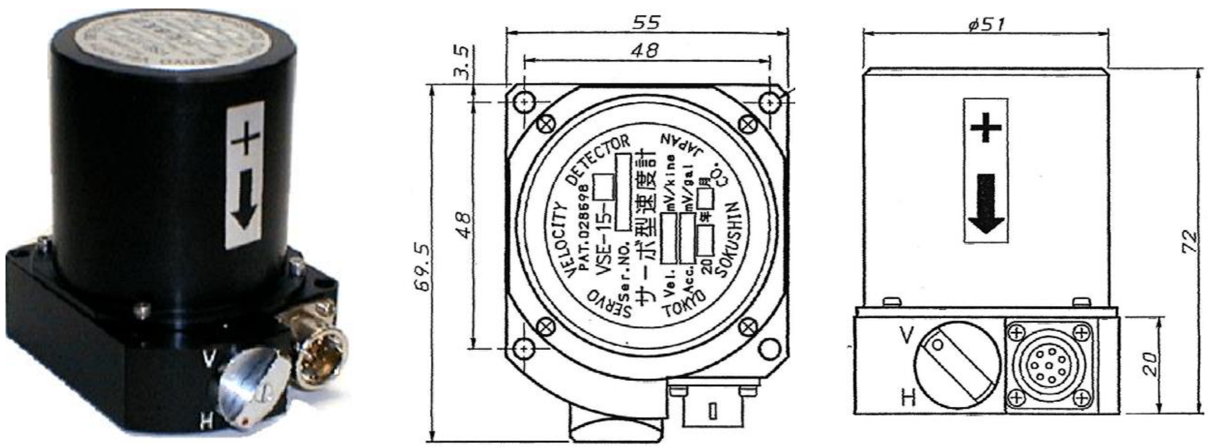

(a) Photo (b) Top View (c) Side View

Figure 6. Velocitymeter VSE-15D equipment used in the present study

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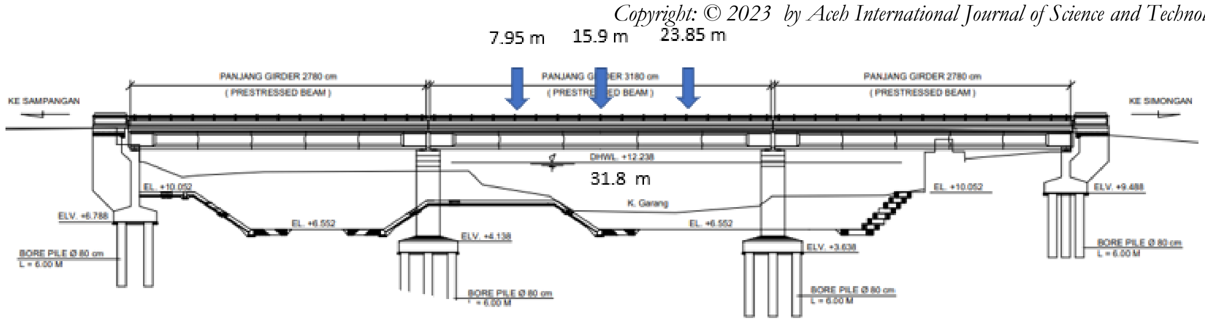

Figure 7. Placement of sensor

Figure 8. Vibration measurement

Results

Data Analysis / Identification of Modal Parameter

In this data analysis, the field measurement results are used to identify the frequency and range of vibrations. As shown in Figure 9, initial data are now acquired in the form of time data on the X axis and acceleration data on the Y axis. In order to obtain the modal frequency and mode shape using the fast Fourier transform, an analysis is conducted from the initial data obtained in the form of velocity data as shown in Figures 10 and 11.

Compare result from FE model and measurement

At this point, it will be compared how the mode shape looks and what frequency each vibration type has. The results from ambient measurements are used as a reference if there are discrepancies between the mode shape and the frequency value (Figure 12). In order to update the following FE model while still using the bridge design codes, some of the necessary parameters will need to be adjusted.

Updating Finite Element Model (Baseline FE Model)

In structural health monitoring research, differences in dynamic property values can occur between the results of measurements in the field and the FE model. Of these two values that have differences, what is used as a reference is the results of measurements in the field. The existing FE model needs to be updated so that the difference values that occur are as small as possible. The final FE model, which will be the baseline for further measurements, will be obtained from the outcomes of the process of updating the FE model. As can be seen in Figure 12 and Table 1, the final FE model obtained is thought to be able to accurately depict the state of the bridge structure in real conditions. While the three dots depict the result from ambient vibration measurements, the straight line represents the result from the FE model. It is obvious that the straight line and the three dots

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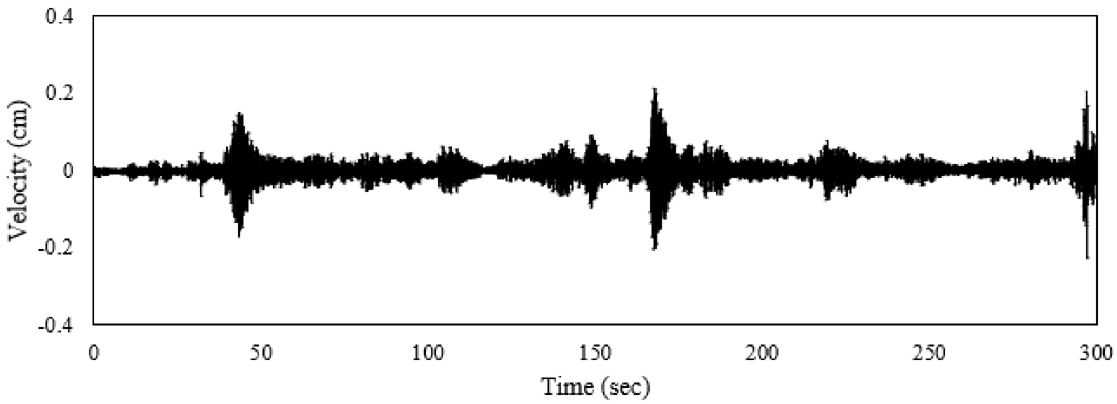
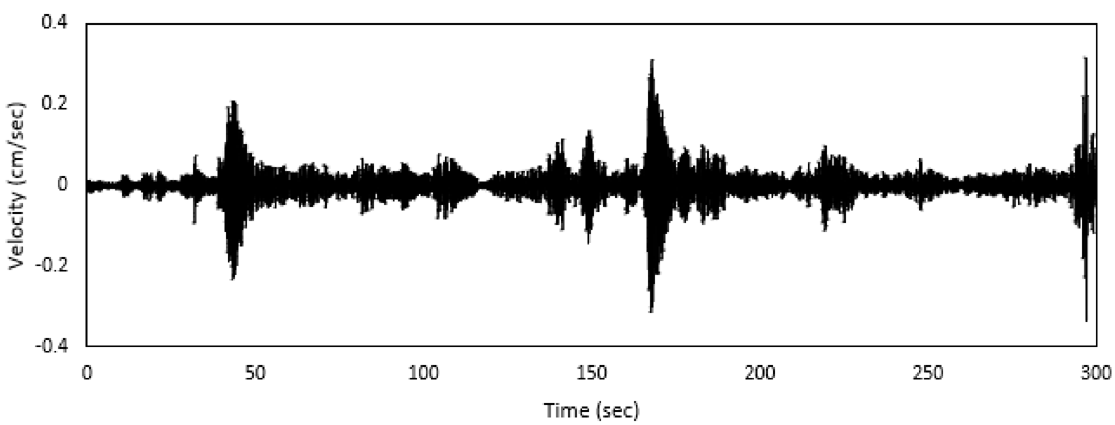
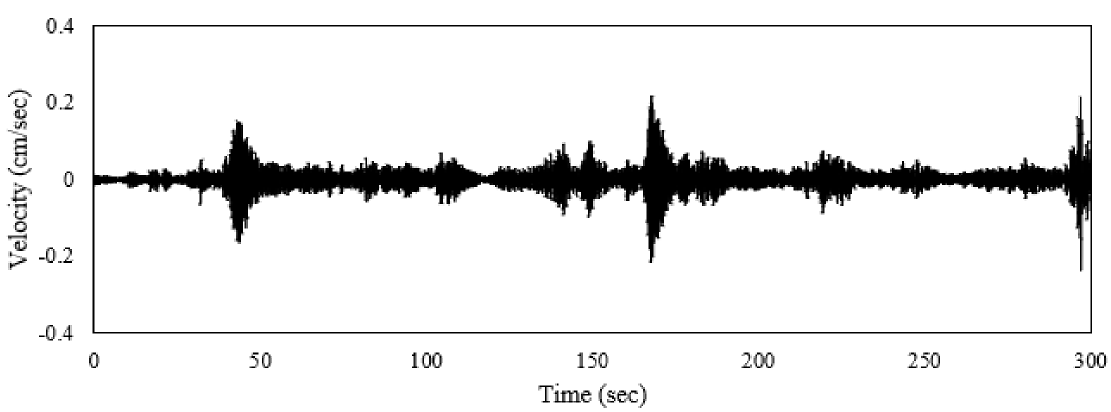
almost coincide. The measurement results' frequency value is 3.995 Hz, which is lower than the FE model's frequency value of 3.949 Hz by -1.15%. It can be inferred that the final FE can accurately depict the condition of the bridge structure based on the difference in frequency values, which is still less than 5%, and the shapes of the vibrational variations, which are nearly coincident.



(a) Sensor 1st

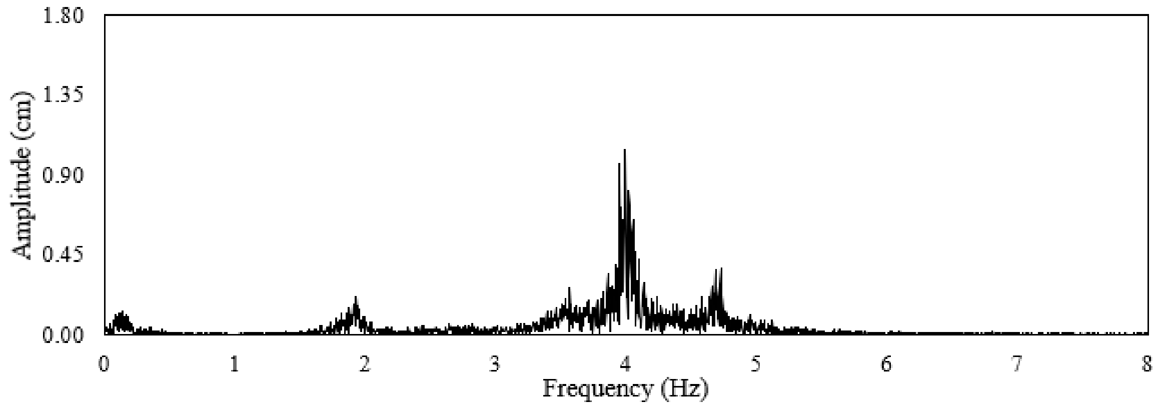


(b) Sensor 2nd

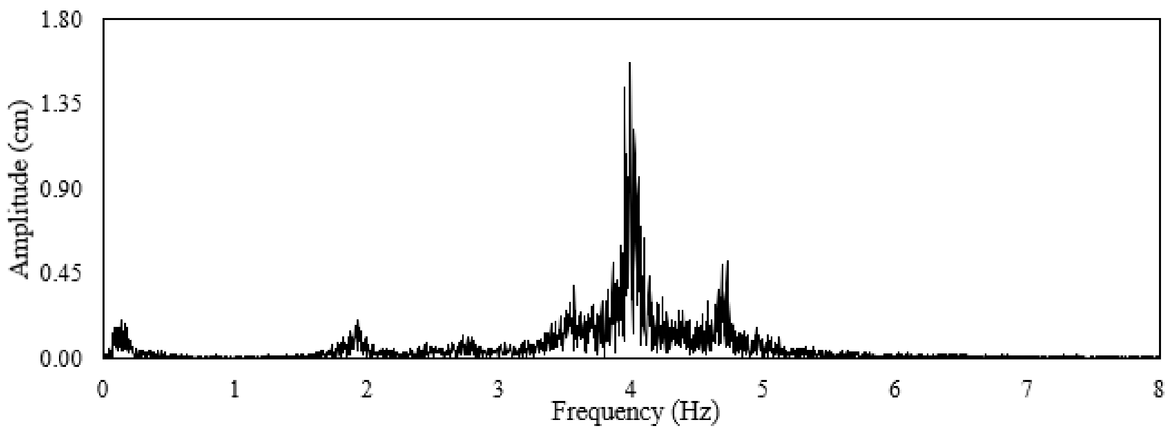


(c) Sensor 3rd

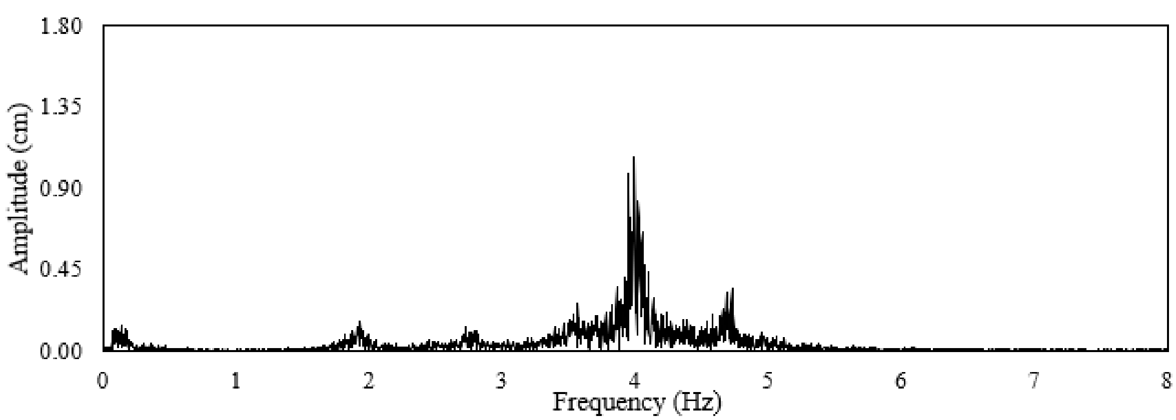
Figure 9. Velocity record from measurement



(a) Sensor 1st



(b) Sensor 2nd



(c) Sensor 3rd

Figure 10. Fast Fourier Transform

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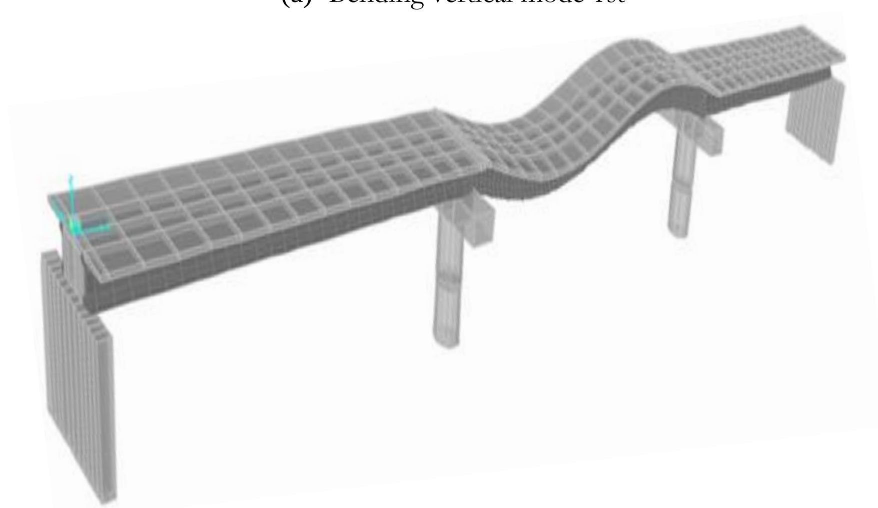
(a) Bending vertical mode 1st



(b) Bending vertical mode 2nd

Figure 11. Bending vertical mode

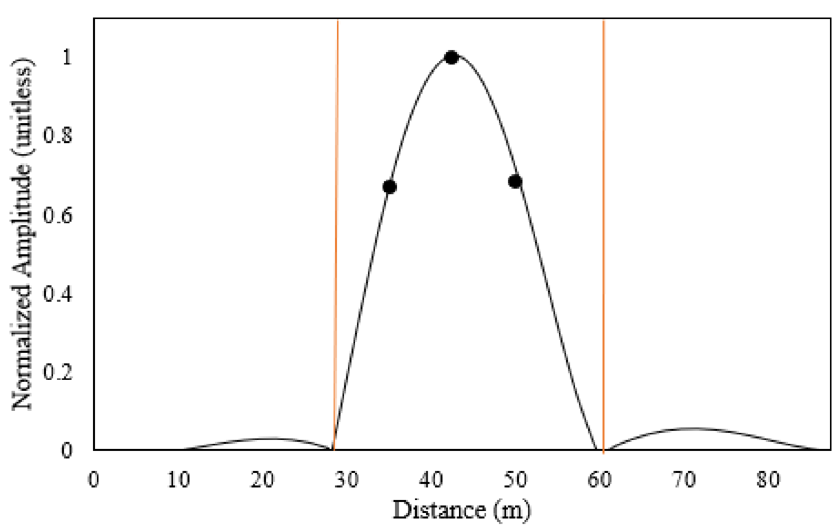


Figure 12. Comparison FE model and Measurement

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Table 1. Comparison result from FE model and measurement

Parameter	Measurement	FE Model	Deviation
Frequency (Hz)	3.995	3.949	-1.15 %

Discussions

Since live loads are one of the loads that have a significant impact on the bridge, the Factored Load Method (first-level evaluation) in the code about Guidelines for Determining Bridge Capacity Values only takes these into account. The bridge is loaded with HL-93K-type trucks in addition to this Live Load. The design of bridge ratings is the primary goal of this modeling. Bridge rating is therefore used in the design options section's analysis and design submenu.

In the code about Guidelines for Determining Bridge Capacity Values, the Factored Load Method (first-level evaluation) is used for Define Load Cases. This method only takes into account permanent loads and traffic loads that are calculated to calculate reactions or internal forces.

There is a manual formula for calculating the load rating factor in the regulations for determining the bridge load rating for existing bridges. Each of these variables has a value that can be entered in the load combination section. According to the study's findings in figure 13, the load on the girder's maximum moment and shear force are 2239 kNm and -1692 kN, respectively.

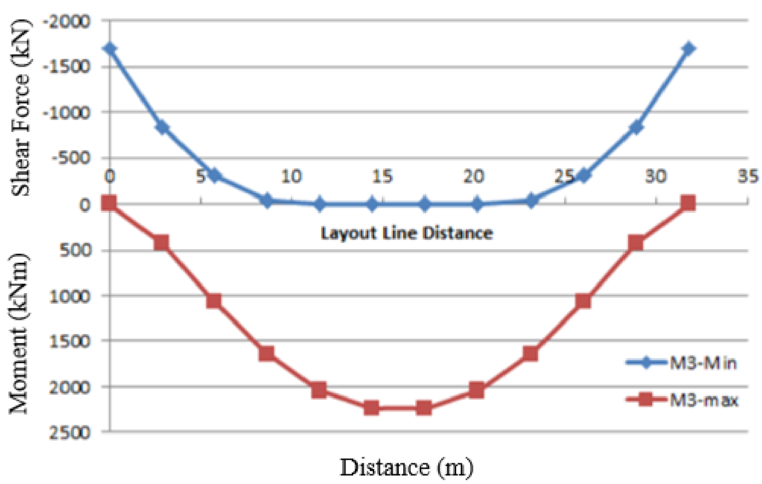

Figure 13. Moment and Shear Force

The maximum displacement that can be modeled for this bridge is 6.8x10^{-3} m, which happens in the middle of the span (Figure 14). The additional load of the HL-93K vehicle that was passing on the bridge caused the displacement of 6.8x10^{-3} m to occur.

Load Rating Factor

The standard for evaluating bridges used is the code about Guidelines for Determining Bridge Capacity Values. Load rating factor is the method used in the first level evaluation. According to the code about Guidelines for Determining Bridge Capacity Ratings, if a bridge is capable of withstanding its planned load based on the load factored method, then evaluation of the second and third level load ratings does not need to be carried out. Because in this study, all components are able to accommodate the force that occurs due to the planned load, therefore there is no need to evaluate the second level of load rating. Based on the analysis carried out using CSI:BRIDGE v21, it produces load rating factor values for the bridge girders and plates. These results become a reference for the condition of the bridge's upper structure if it experiences additional loading. Figure 15 is the condition of the bridge plate if it experiences additional loading. So the load rating factor output results are obtained in the form of Table 2

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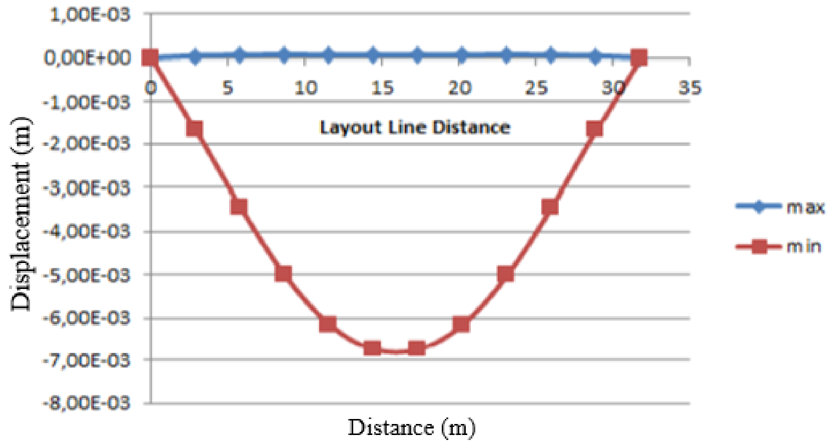

Figure 14. Displacement

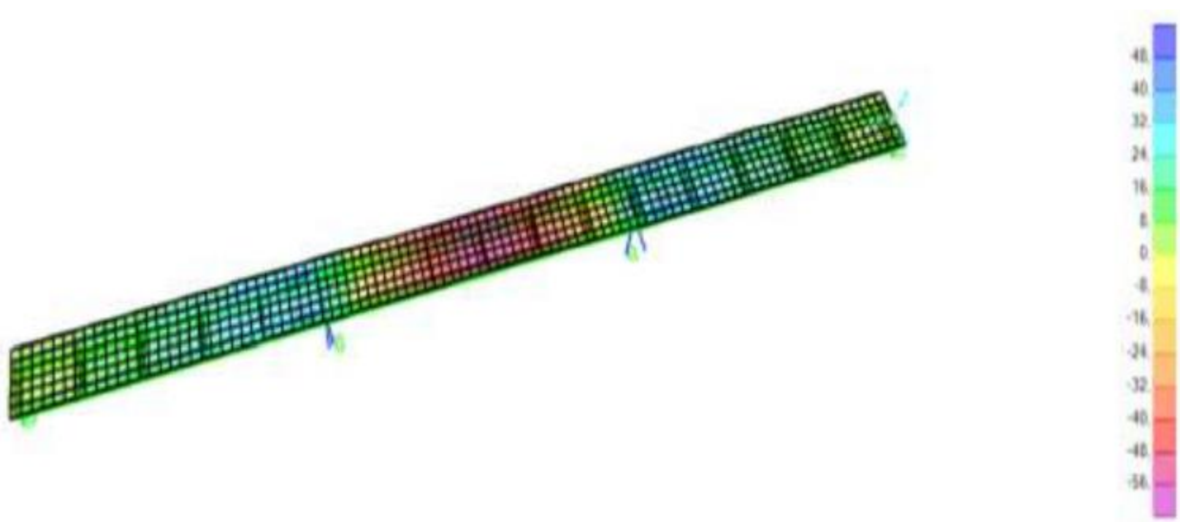

Figure 15. Condition deck due to additional loading

The results of the modeling above show that the value of the inventory load rating factor on the superstructure of the bridge has a maximum value of 3.54 at the 8 meter station and 23 meter station, then a minimum value of 2.67 at the station other than the maximum value. Because the rating factor value is larger than 1, this shows that the superstructure of the bridge is able to withstand the planned load, which is to be traversed by HL-93K-1 type trucks as shown in Figure 16 .

Conclusion

From the analysis of dynamic properties in the form of modal frequency and mode shape, it is known that the modal frequency of the FE model is 3.949 Hz and the results of vibration measurements on the bridge are 3.995 Hz. A difference of -1.15 % indicates that the FE model that has been updated is considered to be able to represent the condition of the bridge structure and can be used as a reference FE model. From this FE model, it can then be used to carry out research or studies for monitoring bridge structures in the future. Regarding the analysis of the load rating factor of the Tugu Suharto Bridge Semarang using CSI: BRIDGE v21 software, it shows the value of the inventory load rating factor with a maximum value of 3.54 at a station of 8 meters and a station of 23 meters, then a minimum value of 2.67 at a station other than the maximum value. In the Bridge Load Rating Factor Determination regulations for existing bridges, if the value of the load rating factor is larger 1, then the bridge does not require handling action (no need for load restrictions or repairs), which means that the capacity on the profile can withstand the additional design load that works, so that the feasibility of the bridge

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structure The Suharto Monument in Semarang can be declared worthy. The bridge can withstand the planned load based on the load factor method. In future research, further research can be carried out in the form of regular monitoring of the condition of the bridge structure based on current measurement data. It can assess the condition of the bridge structure at the time the measurements are taken.

Table 2. Load rating factor

Bridge Obj	Station	Girder Dist	Cond Fact	Syst Fact	Resist Fact	M3DC	M3DW	M3P	M3LUM	Mr	Rating
	m	m	unitless	unitless	unitless	KNm	KNm	KNm	KNm	KNm	unitless
BOB#1	0	0	1	1	1	0	0	0	-1756.86	-3320.14	1.8
BOB#1	0	0	1	1	1	0	0	0	-1756.86	-3320.14	1.8
BOB#1	2.89091	2.89091	1	1	1	0	0	0	-814.133	-2289.56	2.8
BOB#1	2.89091	2.89091	1	1	1	0	0	0	-814.133	-2289.56	2.8
BOB#1	2.89091	2.89091	1	1	1	0	0	0	-816.707	-2289.41	2.8
BOB#1	2.89091	2.89091	1	1	1	0	0	0	-816.707	-2289.41	2.8
BOB#1	5.78182	5.78182	1	1	1	0	0	0	-287.659	-844.10	2.9
BOB#1	5.78182	5.78182	1	1	1	0	0	0	-287.659	-844.10	2,9
BOB#1	5.78182	5.78182	1	1	1	0	0	0	-288.798	-843.95	2.9
BOB#1	5.78182	5.78182	1	1	1	0	0	0	-288.798	-843.95	2.9
BOB#1	8.67273	8.67273	1	1	1	0	0	0	1715.37	6060.24	3.5
BOB#1	8.67273	8.67273	1	1	1	0	0	0	1715.37	6060.24	3.5
BOB#1	8.67273	8.67273	1	1	1	0	0	0	1705.52	6060.38	3.5
BOB#1	8.67273	8.67273	1	1	1	0	0	0	1705.52	6060.38	3.5
BOB#1	11.5636	11.5636	1	1	1	0	0	0	-13.842	-3320.14	1.8
BOB#1	11.5636	11.5636	1	1	1	0	0	0	-13.842	-3320.14	1.8
BOB#1	11.5636	11.5636	1	1	1	0	0	0	-14.490	-2289.55	2.8
BOB#1	11.5636	11.5636	1	1	1	0	0	0	-14.490	-2289.55	2.8
BOB#1	14.4545	14.4545	1	1	1	0	0	0	-7.6023	-2289.41	2.8
BOB#1	14.4545	14.4545	1	1	1	0	0	0	-7.6023	-2289.41	2.8
BOB#1	14.4545	14.4545	1	1	1	0	0	0	-8.2722	-844.101	2.9
BOB#1	14.4545	14.4545	1	1	1	0	0	0	-8.2722	-844.101	2.9
BOB#1	17.3454	17.3454	1	1	1	0	0	0	-8.2722	-843.959	2.9
BOB#1	17.3454	17.3454	1	1	1	0	0	0	-8.2722	-843.959	2.9
BOB#1	17.3454	17.3454	1	1	1	0	0	0	-7.6023	6060.24	3.5
BOB#1	17.3454	17.3454	1	1	1	0	0	0	-7.6023	6060.24	3.5
BOB#1	20.2363	20.2363	1	1	1	0	0	0	-14.490	6060.38	3.5
BOB#1	20.2363	20.2363	1	1	1	0	0	0	-14.490	6060.38	3.5
BOB#1	20.2363	20.2363	1	1	1	0	0	0	-13.842	6060.38	3.5
BOB#1	20.2363	20.2363	1	1	1	0	0	0	-13.842	6060.38	3.5
BOB#1	23.1272	23.1272	1	1	1	0	0	0	1705.52	6060.24	3.5
BOB#1	23.1272	23.1272	1	1	1	0	0	0	1705.52	6060.24	3.5
BOB#1	23.1272	23.1272	1	1	1	0	0	0	1715.37	-843.959	2.9

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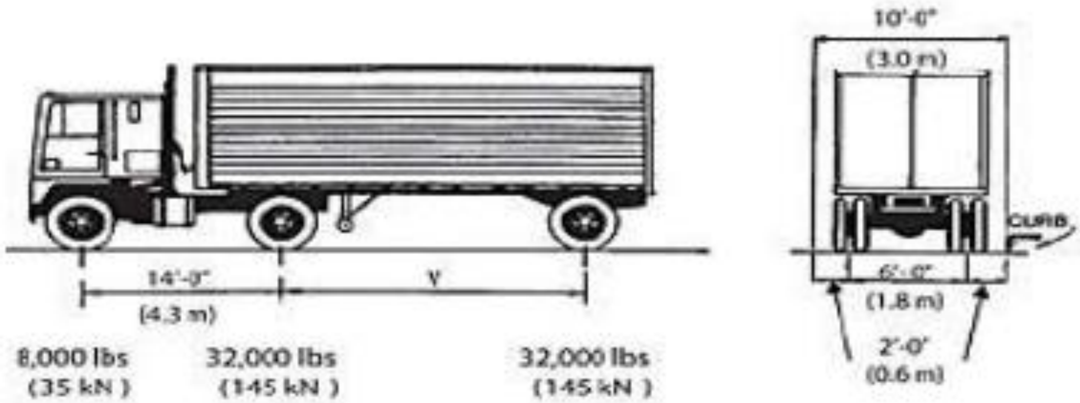

Figure 16. Truck type HL-93K-1

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