Brühwiler, 2023; Dindar et al., 2020; Setayesh et al. 2022; Porthin et al., 2020). Additionally, rating systems are employed to quantify the severity of observed defects and prioritize maintenance and repair efforts based on their impact on structural integrity (Abdal et al., 2023; Abdallah et al., 2022; Abdelkhalek and Zayed, 2020; Ye et al., 2020; Zheng et al., 2022; Kruachottikul et al., 2021).

Non-destructive testing (NDT) techniques play a vital role in the condition assessment of concrete bridges, allowing for thorough inspections without causing damage. The use of non-destructive testing (NDT) methods, whether based on embedded or surface mounted sensors, or contact or non-contact based devices, plays a critical role in providing real-time data for bridge management systems, which enables informed decisions for maintenance and repair operations (Kashif Ur Rehman et al., 2016). Additionally, NDT methods aid in developing a comprehensive understanding of structural behavior under service loads, predicting remaining bridge life, and identifying possible failure modes. On the other hand, the use of embedded sensors and other emerging technologies for real-time data collection has been identified as a valuable technique for incorporating bridge condition information into management systems (Costin et al. 2024; Jiménez Rios et al. 2023; Kaewunruen et al. 2021; Omer et al. 2021). This approach enables the continuous monitoring of structural behavior and the detection of potential issues, contributing to more informed maintenance and repair decision-making. Additionally, the development of field-ready testing equipment and ongoing data collection of bridge failure modes are essential for risk assessment and predicting remaining bridge life. These evaluation techniques, when combined, provide a comprehensive approach to the condition assessment of concrete bridges, enabling effective maintenance and ensuring structural safety and longevity (Senthilkumar et al., 2020; Anikwe et al., 2022).

Understanding the fundamentals of bridge infrastructural behaviour and performance is crucial for the selection of suitable ideal non-destructive testing and evaluation (NDT&E) techniques for condition assessment and health monitoring of in-service bridges (Zarate et al., 2022; Abdal et al., 2023). The response of bridge structures to various NDT methods needs to be thoroughly understood to accurately interpret test results. Moreover, engineers need a comprehensive understanding of the behavior of concrete under service loads to effectively interpret the readings obtained from NDT methods. Furthermore, long-term bridge behavior in response to diverse loading scenarios, particularly in relation to the effects of damage, is essential for predicting remaining bridge life and potential failure modes (Dong et al., 2020; Zarate et al., 2022; Ye et al., 2020; Luo et al., 2021; Panian and Yazdani, 2020).

More importantly, the development of an autonomous robotic system equipped with NDT&E sensors, such as ground penetrating radar (GPR), electrical resistivity (ER), and a camera, offers a cost-effective and non-disruptive solution for bridge deck inspection in urban environments. This system can operate in real-time, providing efficient data collection without the need for inspection sites to be evacuated prior to inspection, thus minimizing disruptions to urban traffic flow. Therefore, addressing the challenges of urban environments in bridge condition assessment requires a combination of technological innovation, financial support, and careful planning to ensure the effective implementation of NDT&E techniques (Ahmed et al., 2020; Dabous & Feroz, 2020; Abdelkhalek and Zayed, 2020; Zhang et al. 2023; Poorghasem & Bao, 2023). In addition, integration of artificial intelligence (AI) holds promise for enhancing the precision and efficiency of bridge assessments based on its fast data interpretation and decision-making process (Harle, 2024; Ranyal et al., 2022).

Challenges in the field of bridge condition assessment predominantly stem from the limitations of traditional inspection methods, which in addition to reasons cited earlier, is also less cost-effective due to travel expenses, shut-down times and long labour hours. On the other hand, the intensive sensor-based techniques come with huge financial constraints that hamper the decision to implement or expand the use of NDT&E techniques in bridge condition assessment. The development of methodologies to perform periodic non-destructive field inspections of concrete bridges based on the quantitative methods to ensure structural integrity and safety of bridge infrastructure. The ongoing research is essential for the development of field-ready testing equipment and the collection of data on bridge failure modes for risk assessment. These techniques, if properly applied, can produce more reliable and timely results, saving time, money, and manpower.

The primary goal of this study was to technically address the limitations of both the traditional inspection methods and the relatively very expensive emerging technologies by deploying easily affordable techniques using geomatics, global positioning system (GPS) and the Schmidt or rebound hammer testing on an existing concrete bridge over Metsimothlabe River. The NDT&E techniques employed variability of the compressive strength of bridge deck using Schmidt hammer, evaluation of the determination of deformation of the existing deformation profile and physical deterioration of the bridge using non-contact methods and dynamic of the bridge under random traffic and controlled loading using GPS technology.

MATERIALS AND METHODS

Management of bridge infrastructure in Botswana

Botswana has a large number of bridge structures located mainly from south to north to north-east. Some of these bridges form part of the heavily trafficked north-south trade corridor route from Ramatlabama to Kazungula, which make tremendous contribution to the country's economy. Department of Roads (Botswana) has 208 structures comprising 28 major culverts and 180 Bridges in their inventory. The estimated average age of majority of bridges (64.7%) is 38 years. Over 180 bridges in Botswana, with a total bridging length of about 7600 m, provide passage over river and rail crossings on the highway network. The dominant bridge construction material in Botswana is concrete and it accounts for approximately 81% of the bridges. The remaining structures are composite, steel and earth filled.

The trend of road maintenance budget has been generally on the increase in the last ten years. However, despite the increase in budgetary allocation, several bridge infrastructures still lack adequate maintenance due to inadequate funding, and this situation could trigger avoidable decline in structural performance due to accelerated deterioration. If this trend continues, it will have a negative impact on the bridge infrastructure condition in the long term.

The Roads Department has adopted a systematic approach to objectively assess the condition of bridges under their control by adopting the Bridge Management System for fund allocation and prioritization. Visual inspection is the default bridge inspection methodology employed by the Bridge Authority in Botswana.

Description of Metsimothlabe bridge

Metsimotlhabe Bridge, constructed in 1979, is located at approximately 18 km from Gaborone on the Gaborone–Molepolole Road (A12 route) in the Kweneng District of Botswana. The bridge provides a passable river crossing at all times over the Metsimotlhabe River. The road is a strategic route from the Gaborone, the country's capital and ultimately connects with the A2 road which is a trade corridor passing through Botswana and connecting South Africa and Namibia. The road carries a substantial amount of traffic, but financial constraints have delayed the upgrading to a dual carriageway. The estimated annual average daily traffic in 2020 was 9508 vehicles.

The bridge consists of two spans simply supported RC beam and slab supported on a wall type solid pier. The total span is 42.6 m and total width is 12.1 m. There are 1.5 m wide walkways on either side of the roadway with steel railing parapet. Each span length is 21.3 m comprising five girders spaced at 2.1 m centers. The width and overall depth of the girders are 400 mm and 1500 mm respectively. There are two wall type abutments and RC wall pier. Deck construction method is cast in-situ on stationary false work and the bearing type is elastomeric, teftlon and stainless-steel plates. The vertical alignment of the bridge is at a constant gradient of 0 to 2%, horizontal alignment is straight, camber/cross fall is 2 to 4%, and the angle of skewness is zero degree. Minimum vertical clearance is 6.5 m and the bridge altitude is at 900 m above mean sea level. The surface on deck and approaches is asphalt. Figure 1 illustrates the bridge elevation, underside and profile section.

Figure 1. The elevation, the underside view and cross-section of Metsimotlhabe Bridge

Methodology for assessment of Metsimotlhabe bridge

1) Determination of compressive strength of the superstructure

The effectiveness of Schmidt hammer in determining concrete compressive strength of the bridge superstructure was implemented in accordance with EN 13791 (2007) and EN 12504-2 (2021). The impulse test hammer (rebound hammer) is a simple, quick and inexpensive to use and was calibrated with a total of 36 precast concrete cubes of different predetermined compressive strengths between 20 and 35 N/mm^2. The N Type MH-75 model Concrete Test Hammer (Schmidt Hammer) was used as shown in Figure 2 together with the on-site test process. Prior to field test, the rebound hammer was calibrated to establish the relationship between compressive strength and rebound number.


a)


b)

Figure 2. (a) N Type concrete test hammer and (b) on-site test process

2) Monitoring of flexural and torsional deformation using geomatic survey

A Hi-target ZTS-420R total station with absolute encoding measurement method was used on Metsimotlhabe River Bridge to determine the bridge elevation in order to quantify the permanent vertical displacement measurements of the deck. For on-site deformation monitoring, 10 monitoring points, A1 to A5 and B1 to B5, spaced at 5.325 m at the northern and southern sides respectively were marked on the external bridge girders as shown in Figure 3. Two control points were also established and fixed on stable ground on either side and away from the bridge. Three observations of data for each monitoring point were undertaken to minimize measurement errors.

Figure 3. Bridge data collection using the Hi-Target total station

3) Monitoring of bridge dynamic response to ambient traffic loading

Real-time kinematic (RTK) dynamic displacement data of the bridge girder under random traffic loading was monitored using GPS survey equipment at a sampling rate of 10 Hz. Five rover observation stations were each considered along the center-line of the bridge as depicted in Figure 4. Each rover station was observed for about 15 minutes for each cycle. On the observing point, the GPS antenna was placed vertically on the bridge surface, which reflects the movement of the bridge completely. Three sets of data were collected and pre-processed using GPS-Trimble software in the analysis. The output of the GPS software was the time series of the instantaneous Cartesian coordinates of the rover receiver in the World Geodetic System (WGS84) coordinate system.

Figure 4. Field data capturing using GPS receivers

4) Dynamic response of Metsimothlabe Bridge using numerical analysis

Finite element analysis (FEA) of Metsimotlhabe Bridge was performed using OptiStruct® software to analyse the dynamic response of the bridge to an interlink loaded truck of maximum axle length of 19 m and permissible maximum combination load 549.36 kN using acceleration, displacement and strain measurement data collected at a sampling rate of 100 Hz.

RESULTS AND DISCUSSION

Correlation between rebound hammer and compressive strength of concrete

The Schmidt hammer calibrated graph was plotted from the concrete compressive results and Schmidt rebound numbers (RN) measured from the test specimens in the laboratory and are as shown in Figure 5.

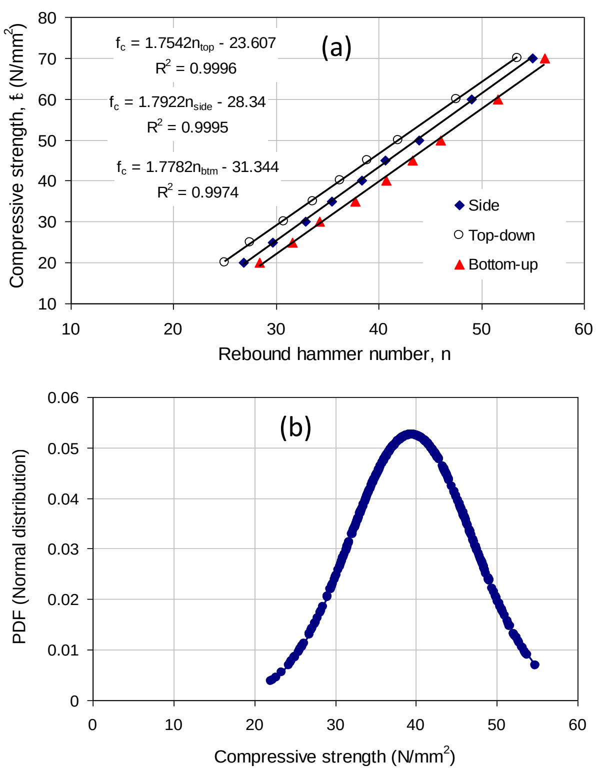

Figure 5: (a) Relation between compressive strength of concrete and rebound, and (b) normal distribution of compressive strength of the bridge superstructure.

The relationship of the Rebound numbers and compressive strength of data was established and used to extrapolate data to obtain relative values between 35 and 70 MPa. Figure 5 was then used to predict the equivalent concrete strength for each Rebound Number measured on the bridge deck as underscored by Hannachi and Guetteche (2014) that compressive strength of concrete can be determined by testing of molded specimens or by core specimens drilled from existing structures. The high R^2 value of 0.9995 confirms a more relative predictive power of this model for the estimation of concrete strength results as emphasized by Mahmoudipour (2009). These plots were made to calibrate Schmidt hammer device for non-destructive testing of the bridge concrete girders in the field. Table 1 shows that perfect correlation exists between the compressive strength obtained from different directions on concrete surface namely bottom-up, sideways and top-down.

The statistical parameters of the compressive strength distribution at 205 different points of the superstructure are summarized in Table 2.

Table 1. Correlation among measured compressive strengths at bottom-down, sideways and top-down

	Bottom-top	Sideway	Top-down
Bottom-top	1	0.9985	0.9982
Sideway	0.9985	1	0.9998
Top-down	0.9982	0.9998	1

Table 2. Statistical parameters of the compressive strength

Parameter	Rebound number (n)	Compressive strength, f_c (N/mm^2)
Mean	37.70	39.48
Mode	37.79	39.12
Median	37.79	39.12
CoV (%)	11.33	19.22
Skewness	-0.04	0.06
Kurtosis	0.03	0.04

The consistency of the compressive strength is evident from the mean, median and mode values obtained from the field measurements. The mean of compressive strength calculated is 39.48 N/mm^2. The calculated compressive strength when compared with the concrete compressive strength obtained from records at construction stage which is 30 MPa shows an increase in strength of about 31.6% and these results are, to a reasonable extent, in confirmation to findings by Ellingwood et al. (2009) who indicated that concrete strength can increase by as much as 150% beyond the 28-day standard basis due to continued hydration. This study demonstrated that the CoV value (19.2%) of rebound results are on lower side in the case of earlier constructed structure than that of old structures with age ranging between 30 to 40 years.

Importantly, damage in the form of cracking or deterioration of concrete is an indication of loss of stiffness which is directly influenced by decline in the modulus of elasticity of concrete. Even more importantly, there is a direct relationship between the compressive strength and the modulus of elasticity of concrete. Hence, the consistency of the compressive strength and the values of the CoV, skewness and kurtosis are clear evidence that the superstructure had no structural defects. Hence, it can be concluded that there were neither any cracks nor deterioration of concrete superstructure.

Evaluation of flexural and torsional deformation of concrete girders

The reliability of the data of the bridge profile was ascertained by a set of random data analyzed to determine its variability where the traditional statistical functions, mean, standard deviation and coefficient of variation were estimated. In addition, as shown in Figure 6, the results show that the north and south profile data is normally distributed. The bridge levels data depict similar results.

It is evident from the bridge level readings at the northern and southern edges that slight torsional deformation exists on the girder-deck structural system. The southern edge is slightly higher from levels 998.62 m to 998.67 m, while the northern edge is higher from levels 998.67 m and above. The percentage difference between the average levels of the northern and southern edges of 0.0014% was numerically very negligible. Further assessment of the levels of the northern and southern edges at common chainages verified the tendency for torsional deformation.

Bridge data results measured at five different locations on the bridge girder namely the two supports, two quarter spans and the mid span as shown in Figure 7. The measurement data were taken on the southern and southern sides or edges of the bridge.

It was observed that the levels of the girder/deck connections on the northern and southern edges revealed differential displacement which was a clear indication of torsional deformation of the bridge superstructure. It can also be observed that the northern edge was relatively highest at chainage zero with a differential of 84 mm and decreased to 39 mm at one-quarter span point. However, there was a progressive twist on the southern edges which increased from 19 mm at the midspan, 25 mm at the three-quarters span and highest at the support at a differential level of 105 mm. These results had confirmed the initial observation and suspicion of torsional deformation. Bridge deck is usually under flexural stress due to traffic loading and possible differential settlement of supports. The bridge insignificantly slopes from the embankment towards the center pier support with the calculated slope of 0.1%.

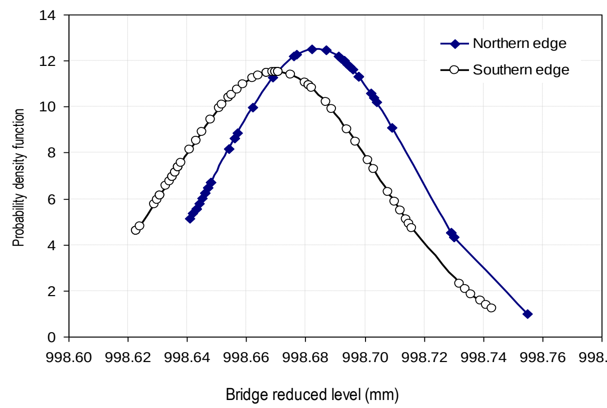

Figure 6. Normal distribution graph for north and south edges data.

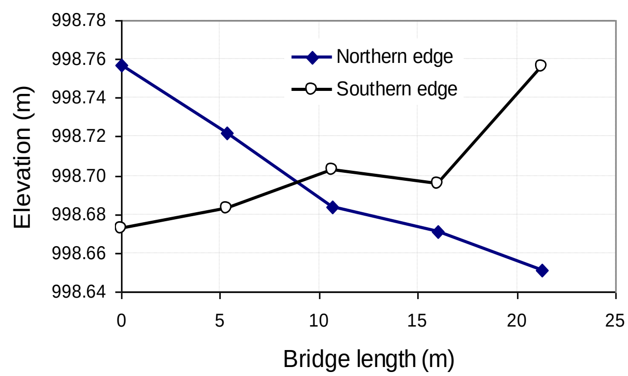

Figure 7. Bridge level at the northern and southern edges of bridge length

Field monitoring of bridge dynamic response to random ambient traffic loading

The results of the displacement of the bridge in the longitudinal (X) and lateral (Y) directions as simultaneously observed showed no movement. In the time series analysis, the long-term periodical movements of the bridge were observed as a trend in short-term observations. The displacement response of the bridge using GPS technology for 15 minutes due to real-time random traffic excitation under ambient condition at the left and right quarter spans and mid-spans is presented in Figure 7. Technically, significant transverse (or vertical) displacements are often expected in the region of the middle of bridge deck/girder.

Typical bridge response under random traffic excitation at the midspan is shown in Figure 7(a). The response data showed that the average deformation varied between -21.5 mm and +7.3 mm leaving the bottom fibre of the girder in tension under random traffic loading comprising heavy and light vehicles. The maximum dynamic flexural displacement in tension was 21.5 mm.

The average real-time dynamic displacement response of the left and right-quarter spans of the girder under the same traffic loading is presented in Figure 7 (b) & (c). The maximum displacement response varied from -12.2 to +3.6 mm (for point B1) and -11.4 to 4.8 mm (for point B3).

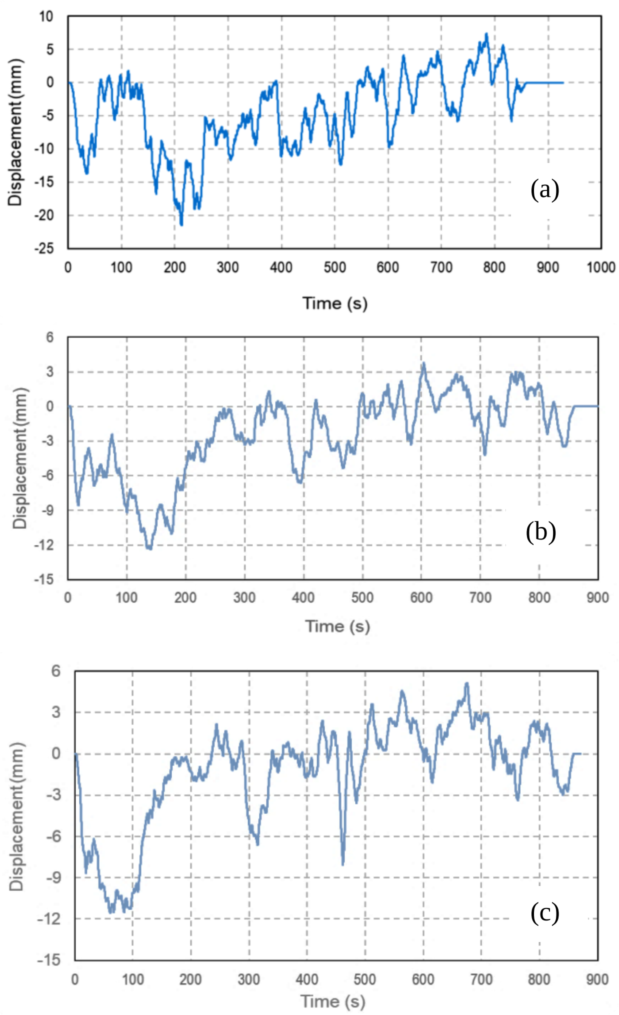

Figure 7. Bridge response under random traffic excitation at (a) midspan, (b) quarter-span (B1), and (c) quarter-span (B3).

Preliminary assessment of the dynamic displacement response monitored at B1 and B3 were comparable with perfect correlation. However, for random uncontrolled traffic movement from either way, slight variation was not unexpected. Hence, the maximum downward displacement in the bottom fibre is similar for B1 and B3 with numerical value of 12.2 mm and 11.4 mm respectively. The mean displacement measurement data at B1 and B3 were 7.39 mm (standard deviation, sd = 5.61 mm) and 7.94 mm (sd = 4.47 mm), respectively. It can be noticed that the quarter span movements demonstrate great similarity and occur at the same magnitude and direction as expected.

The primary purpose of the time-series response of the bridge girder was to determine the instantaneous movement of the deck in the transverse z-direction due to the loading and vibration of the moving loads under uncontrolled ambient and operational conditions. It is worth noting that other repeated measurements taken at the supporting ends of the girders under various traffic conditions consistently gave zero response which was a clear indication that there was no movement. Hence, the results implied stability and good condition of the elastomeric bearings.

Dynamic analysis of Metsimothlabe bridge

The bridge was modelled based on the field data for compressive strength, modulus of elasticity, and the flexural and torsional deformation of the bridge girders to determine the current dynamic conditions of the bridge. The first twelve natural modes of the bridge were extracted from modal analysis. It was found that only three were flexural modes, while the remaining modes were torsional.

A bridge system is usually subjected to a wide range of loading conditions. The critical type of loading condition that causes deformations and stresses on the bridge concrete deck is the dynamic transient wheel force from the vehicles. For the transient analysis, data were collected at a sampling rate of 100 Hz to capture the first three flexural modes of the bridge. The transverse displacement of the bridge evaluated due to vehicular movement was evaluated at the speed of 60 km/h which is the maximum allowable speed on the bridge.

The frequency spectra of the displacement time series for the bridge are shown in Figure 8 for the forced vibration under the moving truck and in Figure 9 for the free vibration of the bridge after the vehicle had left the bridge. It is evident from Figure 8 that the frequency spectra were masked with noise because of interference of the vibrations of both the bridge and the moving vehicle thereby producing many peaks around the natural frequencies of the bridge structure. On the other hand, Figure 9 shows the unambiguous frequencies of the bridge without any interference with the frequencies of the moving vehicle. In addition, only the flexural frequencies were conspicuously identifiable in the frequency spectra.

Hence, periodic monitoring of the bridge and similar road infrastructure systems could provide information on both the global properties like modal frequencies and mode shapes, decline of which is suggestive of structural deficiencies or defect that could affect the performance of the infrastructure.

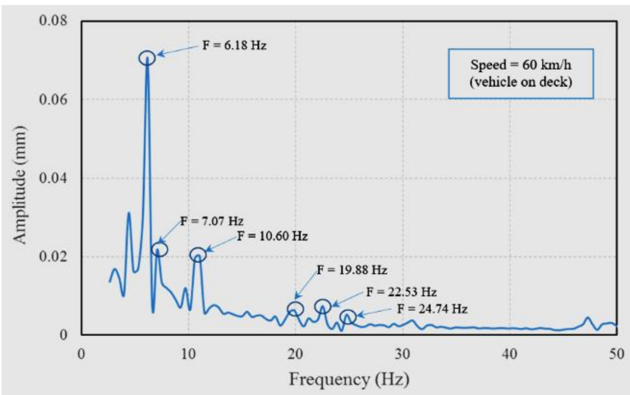

Figure 8. Frequency spectrum for the loaded bridge under moving vehicle at 60 km/h (F = 6.18 Hz)

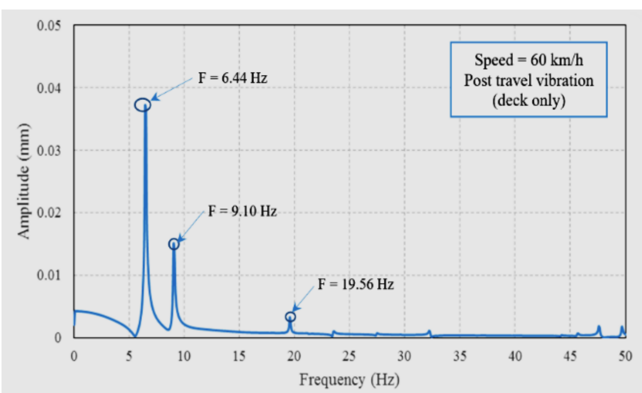

Figure 9. Frequency spectrum for free vibration (white noise) at 60 km/h

CONCLUSION

The compressive strength of the bridge girders in different directions was determined using non-destructive testing device named Schmidt hammer. The compressive strength of the bridge superstructure from random sampling was 39.48 N/mm^2. The concrete compressive strength of the 43-year-old bridge was 31.6% higher than the as-built strength of 30 N/mm^2, which agreed with literature in that the concrete strength could increase by as much as 150% beyond the 28-day standard basis due to continued hydration. The profile of the bridge deck/girder was monitored for possibility of relative deformation, and the global positioning system (GPS) technology was employed to measure the real-time dynamic displacement response of the bridge to random vehicular traffic under normal operational or ambient conditions. The observed levels of the girder/deck connections on the northern and southern edges revealed differential displacement which was a clear indication of torsional deformation of the bridge superstructure. The bridge insignificantly slopes from the embankment towards the center pier support with the calculated slope of 0.1%.

DECLARATIONS

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Data availability

The datasets used and/or analysed during the current study available from the corresponding author on reasonable request.

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Authors' contribution

AP Adewuyi initiated the research idea, identified the research gaps in the existing literature, designed the experimental process, verified the analyzed data obtained and partly wrote and revised the manuscript. **MP Kgafela** developed the proposal, conducted the field study designed, implemented the experimental plan, analyzed the data and wrote the manuscript under the guidance and leadership of AP Adewuyi. Both authors read and approved the final manuscript.

Competing interests

The authors declare no competing interests in this research and publication.

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