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# Effect of deterioration on the performance of a steel bicycle bridge with a tuned mass damper

***David Andres Heinen***
***MSc Civil Engineering and Management***
*Integrated Civil Engineering Systems*
*Civil Engineering Structures*

**Chair graduation committee:**
Prof.dr.ir. A.G. Doree

**Supervisors:**
Dr.ing. R. Kromanis
Ir. G. Dorgelo

**External advisor:**
Dr.ing. E. Said

Faculty of Engineering Technology
Civil Engineering and Management
University of Twente

# Abstract

Vibrations in contemporary pedestrian and bicycle bridges are emerging as a significant concern due to their increasing slenderness. Vibrations can result in structural damage, discomfort for users, and, in extreme cases, bridge failure. To mitigate this issue, a tuned mass damper (TMD) can be integrated in a bridge design to reduce vibrations. Effective applications of TMDs rely on precise alignment between the bridge and the TMD system. Over time, material degradation can disrupt this alignment by altering the critical vibration frequencies of the bridge or TMD. This can diminish damping effectiveness of the TMD and increase vibrations, posing risks to the structural integrity of the bridge. This study investigates the deterioration of TMD spring stiffness and structural degradation by parametrically modelling a simply supported orthotropic bicycle bridge equipped with a TMD. Structural deterioration is a complex process. In this study it is simplified by reducing the Young's modulus of the steel elements exposed to large stresses. The aim is to assess how changes in structural dynamics caused by material deterioration affect the alignment between the bridge and the TMD. The approach consists of the following steps: parametric modelling, model calibration consisting of data collection and model updating, and an alignment analysis between the bridge and TMD. This approach has been applied to the F35 Tubantiasingel bicycle bridge in Enschede, the Netherlands. The model is calibrated three times, (1) before and (2) after installation of the TMD, and (3) when the bridge is installed in its permanent location. Subsequently the model is used to determine new natural frequencies and amplitudes reflecting changes induced by the deterioration of the bridge and TMD. Results indicate that changes in the bridge's natural frequency lead to detuning of the TMD, impairing its damping efficiency. This phenomenon may result in increased accelerations, which can exacerbate degradation of the structure and induce discomfort for its users. This highlights the importance of maintaining the TMD according to the bridge's current condition, in order to extend the bridge's lifespan.



***Keywords—*** *Tuned mass damper, bridge vibrations, natural frequencies, steel bridges, data collection, model calibration, model updating*


*Table 1: Parameters*

| Symbol | Description | Unit |
|---|---|---|
| $K_{opt}$ | Optimal deviation | - |
| $f_D$ | Natural frequency of the TMD | Hz |
| $f_H$ | Natural frequency of the main system | Hz |
| $\mu$ | Mass ratio | - |
| $m_{TMD}$ | Mass of the TMD | kg |
| $m_{struct}$ | Kinetic equivalent structural mass of main system | kg |
| $\zeta_{D,opt}$ | Optimal damping | - |
| $\mathbf{k_d}$ | Spring rate | N/m |
| $\mathbf{c}$ | Damping coefficient | Ns/m |
| $f_m$ | Measured frequency | Hz |
| $f_e$ | Estimated frequency | Hz |
| $MAD$ | Mean absolute deviation | - |

# 1. Introduction

Modern bridge designs are becoming more slender and lighter compared to older designs, driven by aesthetic considerations. This trend can lead to unwanted vibrations in bridges that may impact both structural integrity and user comfort. Maintaining structural integrity is essential for all types of infrastructure, including pedestrian bridges, which play a vital role in urban mobility by improving safety and reducing congestions (Wei et al., 2022). To enhance overall bridge performance and mitigate these vibrations, the application of a tuned mass damper (TMD) proves to be an effective solution. A TMD reduces structural vibrations by absorbing and dissipating energy from oscillations. The effectiveness of a TMD hinges on its tuning to the precise frequency of the unwanted vibration.

This effectiveness may diminish due to aging and deterioration of the TMD spring or changes in the bridge's natural frequency caused by structural decay. This issue poses a recognized challenge for the sustained operation of a TMD (Z. Wang et al., 2019). Much remains unknown about how deterioration of a bridge and TMD system affects the natural frequency and, consequently, the tuning of a TMD. This uncertainty arises from the complexity and variability of structures equipped with TMDs. Understanding how the frequency of a structure changes due to deterioration is crucial. This knowledge can help predicting when a misalignment between the structure and TMD might cause significant damage.

## 1.1 Vibrations in slender bridges

Light and slender bridges lead to low flexural and torsional stiffness, resulting in low natural frequencies (Wen et al., 2016). According to the Eurocodes, standards relevant to pedestrian bridges state that measures should be implemented if the natural frequency of the superstructure falls below 5 Hz vertically or 2.5 Hz horizontally (NEN-EN 1990+A1+A1/C2/NB, 1990). These standards are established to ensure the safety of bridge users and to prevent the recurrence of past undesirable incidents. An example of this is the opening of the Millennium bridge in London. Visitors were anxious due to the significant lateral vibrations of several centimetres caused by the lateral forces of the pedestrian on the deck surface (Dallard et al., 2001) (Eckhardt et al., 2007). Retrofitting a TMD was undertaken to mitigate and control the vibrations by damping. The challenges observed in the Millennium Bridge are not unique, as similar issues manifest in other bridges, e.g. the slender footbridge at the FEUP campus in Porto. Investigations were conducted for this bridge as the vibrations induced by pedestrian loads were discernible (Moutinho et al., 2018). The bridge exhibited multiple critical vibration modes with natural frequencies approximately around 2 Hz. Consequently, a single TMD was insufficient to effectively mitigate all modes. Another instance of excessive vibrations observed in a bridge is exemplified by the Toda Park City Bridge in Japan. This bridge exhibits a primary vibration mode ranging between 0.9 and 1.0 Hz (Ingólfsson et al., 2012). This bridge serves as a link between a bus stop and a stadium. During events, the bridge can accommodate over 2000 people. As a result of the dynamic loading caused by synchronized human motion, significant lateral vibrations occur due to the bridge’s lateral natural frequency being near 1 Hz (Fujino & Pacheco, 1993). To mitigate these vibrations, numerous small tuned liquid dampers have been installed within the box girder of the bridge, achieving the intended outcome. The low natural frequencies in these slender bridges are crucial as they are susceptible to activation by users. Pedestrians typically walk with a step frequency ranging between 1.7 and 2.3 Hz in the vertical direction. A pedestrian on a footbridge produces a time varying force with components in vertical, horizontal-lateral, and horizontal-longitudinal directions (Bachmann, 1987). The vertical component is often considered the most important as it has the largest magnitude. These forces depend on the walking velocity. Each human being exerts a force with a different magnitude and frequency (Andriacchi, 1917). If the bridge has a natural frequency that is close to these forcing frequencies, resonance can occur. Understanding a structure's natural frequencies is crucial for preventing resonance and designing a safe structure (Avitabile, 2000). This phenomenon, in conjunction with the trend towards more slender bridges with reduced mass, signifies a heightened sensitivity of bridges to vibrations induced by dynamic loading (Živanović et al., 2005).

## 1.2 Tuned mass dampers

A TMD is a device used to reduce the amplitude of unwanted vibrations in structures. The primary objective of implementing a TMD is to improve user comfort and reduce chances of structural failure caused by excessive vibrations. It offers a robust design unaffected by high temperatures, provides significant structural damping, and is relatively inexpensive (Elias & Matsagar, 2017). The classic TMD system dissipates vibration energy through dampers and springs connecting a mass to the main structure. In the frequency response function of a bridge without a TMD, a peak is observed at its natural frequency. The function of a TMD is to attenuate this response. The TMD is tuned to a frequency slightly lower than the natural frequency of the bridge. Additionally, the springs of the TMD add stiffnesses to the bridge, thereby raising its natural frequency. This results in two peaks becoming visible in the frequency response function: the lower frequency corresponds to the TMD (peak 1), and the higher frequency corresponds to the new natural frequency of the bridge (peak 2). Between these two peaks lies the original undesired natural frequency of the bridge, which is now attenuated (J. F. Wang et al., 2002). This phenomenon is illustrated in Figure 3. Each of the three elements (spring, mass, damper) of a TMD has a distinct function. The mass provides the inertia necessary to counteract the vibration of the bridge. A small TMD mass can achieve large amplitudes relative to the structure, which may cause problems if space is limited. Additionally, a small mass offers a limited effective range. Conversely, a large TMD mass achieves lower amplitudes relative to the bridge and has a broader effective range but comes at a higher cost (Maurer, 2024). The springs in a TMD provide the restoring force that allows the mass to oscillate, thereby tuning the system to the specific frequency of unwanted vibrations. The damper in a TMD dissipates the energy of oscillations by converting it into heat, thereby reducing the amplitude of vibrations. Correct application of a TMD is crucial, involving precise tuning of its three components to the structure's specific characteristics. A TMD should be strategically placed at the location of greatest amplitude of a structure, as this ensures maximum efficiency with minimal effort. The key equations for the proper tuning of a TMD are presented below (J.P. Den Hartog, 1934).

$$K_{opt} = \frac{f_D}{f_H} \tag{1}$$

$$K_{opt} = \frac{1}{1+\mu} < 1 \tag{2}$$

$$\mu = \frac{m_{TMD}}{m_{struct}} \tag{3}$$

$$k_d = (2 \cdot \pi \cdot f_d)^2 \cdot m_{TMD} \tag{4}$$

$$\zeta_{D,opt} = \sqrt{\frac{3 \cdot \mu}{8 \cdot (1+\mu)^3}} \tag{5}$$

$$c = 2 \cdot m_{TMD} \cdot 2 \cdot \pi \cdot f_d \cdot \zeta_{D,opt} \tag{6}$$

The optimal tuning is highly dependent on the natural frequency (Equation 1) of the structure and the chosen mass ratio (Equation 3), which directly affects both the optimal deviation between the TMD and the structure (Equation 2), as well as the optimal damping required for effective damping performance (Equation 5). These factors directly affect the setting of the springs (Equation 4) and damper (Equation 6). Determining the natural frequency of a bridge in advance can be challenging due to the complexity and insufficient knowledge of the stiffness values of the soil and bearings (Maurer, 2024).

## 1.3 Structure and TMD deterioration

During its lifespan, a bridge is exposed to various environmental factors and stresses that can cause fatigue or damage and change the structural dynamics. In steel structures, fatigue commonly manifests as a decrease in stiffness (Haghani et al., 2012). Structures made of metal can experience changes in their natural frequencies due to damage (Gillich et al., 2015). Human-structure interaction also affects the structural characteristics, potentially leading to a mistuned TMD (L. Wang et al., 2021). The issue of fatigue in steel structures can be addressed by implementing a TMD. For instance, in a steel truss bridge, it has been demonstrated that specific TMD configurations can extend the remaining service life of the bridge by 15% (Pipinato, 2019). Environmental factors also affect the TMD and its components, causing the spring stiffness to vary or experience fatigue under external conditions (Canale et al., 2007). Additionally, errors in design and manufacturing, as well as inaccurate estimations of the main structure's dominant frequencies, can contribute to detuning. A TMD is frequently detuned due to external factors such as temperature variations or minor fabrication tolerances. These factors can impact both the damper and spring of the TMD, thereby significantly influencing its performance (Werkle et al., 2011).

### 1.4 Study objectives

Numerous studies in the literature address the modelling of structures with TMD, as well as the modelling of deterioration in structures and TMD springs. However, a comprehensive approach that integrates these aspects to develop a model for a bridge, which elucidates the impact of deterioration on the alignment of a bridge with a TMD, is currently lacking. This study aims to develop an integrated approach addressing this knowledge gap and apply it to a case study. Offering insights into this alignment holds promise for implementing predictive maintenance, thereby enhancing the structural lifespan. The subsequent sections of this study will elaborate on the used approach in the case study: The F35 Tubantiasingel bicycle bridge. The approach comprises parametric modelling, model calibration consisting of data collection and model updating, and an analysis of the alignment between the bridge and the TMD. The findings are analysed and deliberated upon in the discussion section. Finally, key conclusions and recommendations are drawn.

## 2 Modelling and analysis approach

The proposed modelling and analysis approach is applicable from the design phase throughout the entire service life of the bridge. The first step is to create a parametric model of the bridge in which deterioration factors are included. The next step depends on the bridge's life cycle phase. If the bridge is constructed, model calibration can be employed to align it with the current state. During the model calibration process, data collection is used for model updating. If the bridge has not been built, alignment analysis between the TMD and the structure can be conducted immediately. The workflow of the approach is illustrated in Figure 1.

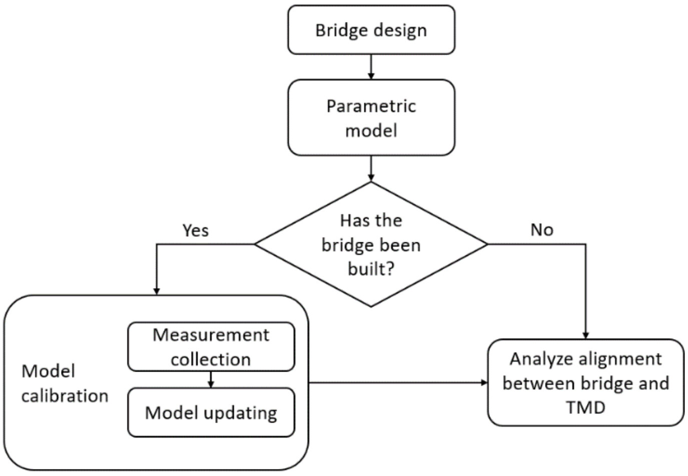


*Figure 1: Flowchart of modelling and analysis approach*

### 2.1 Parametric model

Finite element (FE) modelling can be applied to model the dynamic characteristics of a steel footbridge. A study was conducted involving eight steel footbridges, for which a FE model was constructed. This research has demonstrated that predicting the dynamic characteristics of a footbridge results in a deviation of around 10% in terms of anticipated natural frequencies (Van Nimmen et al., 2014). The model is fundamentally based on the bridge's structure and its bearings. SHELL elements are suitable for modelling a bridge existing of beams and plates (Virgina Tech, 2024). Bearings can be simplified by replacing them with fixed supports and roller supports. A structure with a TMD can be simplified to a 2DOF system, consisting of springs ($k$), masses ($m$), and dampers ($c$) (Figure 2).

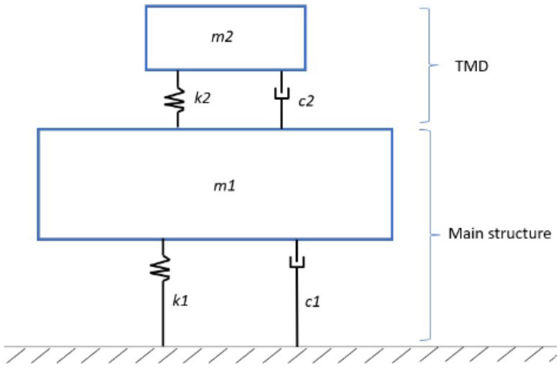

*Figure 2: 2DOF of structure with a TMD*

Although this is a major simplification, it can effectively simulate the operation of a TMD (Berardengo et al., 2015). Figure 3 presents the Welch power spectral density estimates (PSD) for both the single DOF system (main structure) and the 2DOF system (main structure + TMD). PSD is a measure of the power distribution of a signal as a function of frequency, indicating how the signal's power varies across different frequencies (Vibration research, 2024). Adding a TMD to the structure dampens the main peak and creates two new peaks. Peak 1 is caused by specific tuning of the TMD, while peak 2 indicates the resulting adjustment in the natural frequency of the structure (Figure 3) due to the additional stiffness introduced by the TMD.

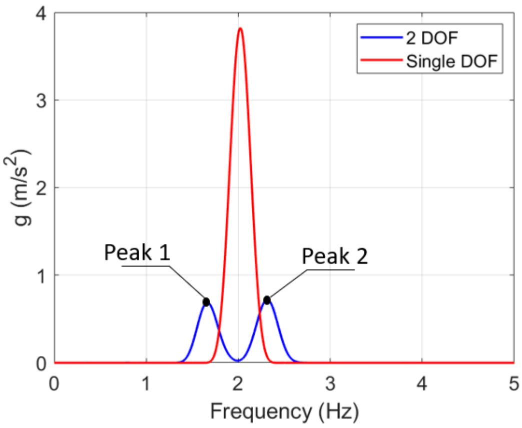

*Figure 3: PSD of a single DOF system and a 2DOF, black dots indicate peak 1 and peak 2 of 2DOF system*

In the FE model, deterioration factors for both the structure and the TMD, which affect the stiffness factors ($k$), must be included, as shown in Figure 2. Stiffness of the structure is determined by the element dimensions and the Young's modulus. Real-world deterioration of steel often involves various factors such as corrosion, micro-cracking, and other forms of damage that influence stiffness. These are complex processes that are challenging to model directly. Therefore, deterioration must be accounted for by altering the Young's modulus. By adjusting the Young's modulus, an overall effect is simplified into one parameter. To account for deterioration in the TMD, the spring stiffness can be adjusted. Analysing the alignment changes necessitates obtaining the frequencies and amplitudes from the parametric model. To achieve these findings, a harmonic analysis and a modal analysis must be conducted.

## 2.2 Model calibration

When the bridge is constructed, the model is calibrated through data collection followed by model updating. Model calibration aims to enhance the parametric model by adjusting parameters to reflect the current real-world condition of the bridge. Model calibration can be conducted at any point during the bridge's lifespan, preferably with both inactive and active TMD states. Throughout its lifespan, external factors are expected to affect the alignment between the bridge and TMD, necessitating periodic model updates.

### 2.2.1 Data collection

For an up-to-date model, data must be collected at the bridge. This can be achieved at various stages throughout its lifecycle. Updating the model requires the use of frequency data, which can be obtained using acceleration gauges to measure the vibrations from the bridge. The placement of the acceleration gauges on the bridge is important. Every natural frequency of a bridge corresponds to a specific mode shape (Avitabile, 2000). Consequently, the highest amplitude and acceleration are contingent on the location on the bridge and thus on the placement of the gauges. The lowest natural frequency is associated with the first mode shape. In this first mode shape, the highest amplitude and acceleration are typically observed at the centre of the structure. Therefore, it is essential to place a gauge at this location. The natural frequencies can be activated by ambient vibrations. A single jump on the structure may also suffice. The desired frequency data can be extracted from the raw data using a PSD analysis. The found frequencies can then be used for model updating.

### 2.2.2 Model updating

When updating the model, the discrepancy between frequency values estimated by the model and those measured on the bridge is evaluated. To ensure an accurate model, the mean absolute deviation ($MAD$) between measured values ($f_m$) and the estimated values with the model ($f_e$) must not exceed 2.5% (equation 7) (Zapico et al., 2003).

$$MAD = \left| \left( \left( \frac{f_m}{f_e} \right) - 1 \right) \cdot 100 \right| \tag{7}$$

If this criterion is not satisfied, modifications to the model are necessary. Depending on whether the TMD on the bridge is active or inactive, different parameters need adjustment. If the TMD is inactive, adjustments to the mass of the bridge are necessary when there is significant deviation. Conversely, if the TMD is active, adjustments to the spring stiffness are required in response to a significant deviation. The schematic representation of this model calibration process is depicted in Figure 4.

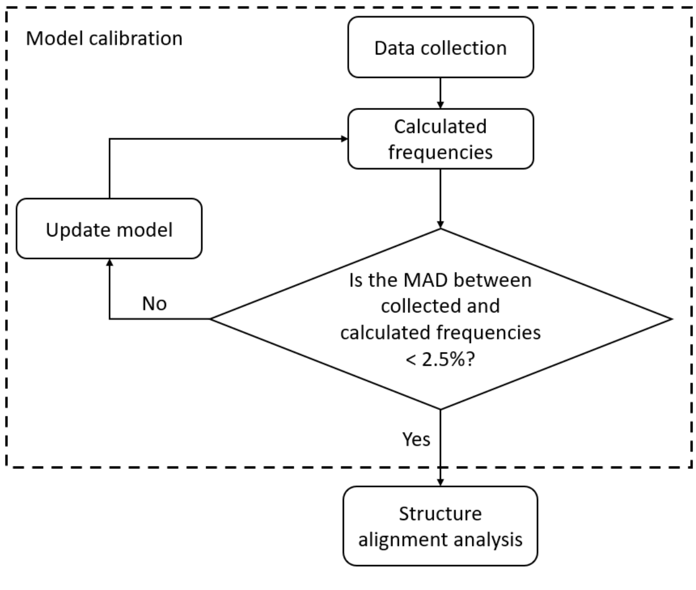


*Figure 4: Model calibration approach incorporating model updating and data collection*

## 2.3 Structure alignment analysis

It is expected that the TMD spring and structural stiffness will decrease as they age. Therefore, in the structure alignment analysis, the stiffness of the structure and the spring in the TMD is gradually reduced in five scenario's. The scenario's are listed in Table 2.

*Table 2: Structure alignment analysis scenario's*

| Scenario | Description |
|---|---|
| **Original** | Structure stiffness and spring stiffness at full capacity |
| **KST80** | Structure stiffness is gradually reduced to 80% of its full capacity |
| **KSP80** | Spring stiffness is gradually reduced to 80% of its full capacity |
| **KSP80 & KST92** | Spring stiffness is gradually reduced to 80% of its full capacity while structure's stiffness is 92.5% of its full capacity |
| **KSP80 & KST85** | Spring stiffness is gradually reduced to 80% of its full capacity while structure's stiffness is 85% of its full capacity |

The reduction in structural stiffness should be applied to areas of the structure where stresses are highest, as these locations have the greatest risk of fatigue. Only the elements of the FE model that are located in this highest stress zone will have their stiffness adjusted. Harmonic analysis is required to determine the frequency change, while modal analysis is used to assess the amplitude change. Since the amplitude in modal analysis does not accurately reflect reality, the change in amplitude relative to the model with the original stiffness must be examined. The expectation of the deterioration simulations is that the alignment between the TMD and the bridge will change due to the new natural frequencies, resulting in the TMD being sub optimally tuned. This misalignment will reduce the damping efficiency of the TMD, leading to increased amplitudes in the bridge's vibrations. A parametric model offers the advantage of simulating multiple scenarios. However, not all scenarios can be fully verified and validated. Therefore, it is crucial that the $MAD$ between the model and reality is less than 2.5% to ensure the reliability of the results.

# 3 Case study: F35 Tubantiasingel bicycle bridge

The F35 Tubantiasingel bicycle bridge in the Netherlands, Enschede (Figure 5) serves as a case study. The bridge, installed in March 2024, is an orthotropic steel box girder bridge equipped with a TMD.


*Figure 5: The F35 bicycle bridge across the Tubantiasingel: a photo of the bridge from north-west with the location of the TMD shown as a yellow box (left) and its geographical location (right)*

The bridge has a span length of 34.75 meters and is simply supported on 4 bearings. The bridge is constructed entirely from steel, with key components including main beams, diaphragms, and stiffening troughs (Cross-sectional view in Figure 6). The diaphragms are uniformly distributed across the bridge at a constant interval of 3.457 meters between each diaphragm.

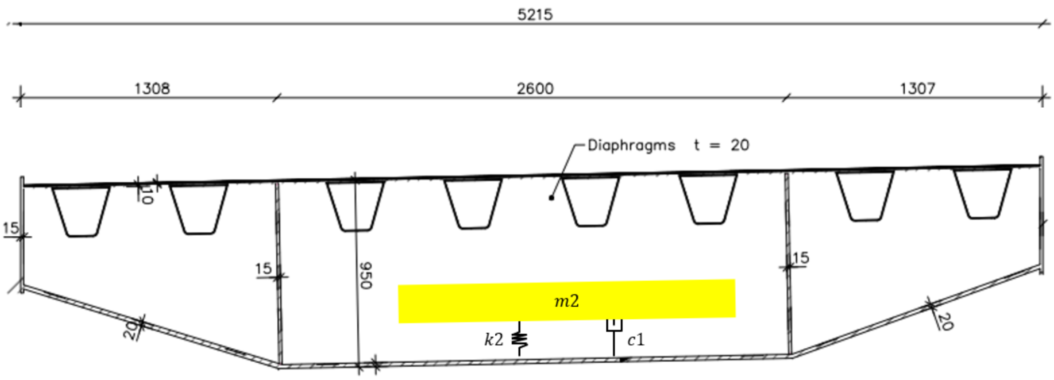


*Figure 6: Cross sectional view of the F35 Tubantiasingel bicycle bridge with an indicative schematization of the TMD as part of a 2DOF system in yellow, dimensions of the structure are in mm*

The TMD is placed inside the bridge at the midspan and is attached to the bottom of the bridge. Figure 7 shows the TMD moments before installation and a drawing with the dimensions of the TMD.

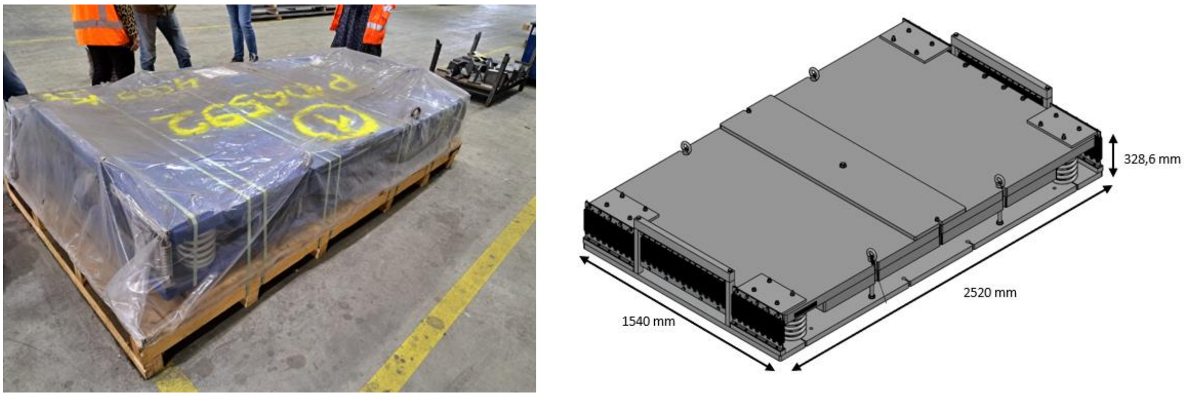


*Figure 7: Photo of the real installed TMD (left) and sketch with dimensions of the TMD (right)*

## 3.1 Parametric model

A FE model has been created based on technical drawings. The model is created in ANSYS finite element software (ANSYS, 2024). Shell elements with the following material properties are used: density $\rho$ = 7850 kg/m$^3$, Youngs modulus $E$ = 210 GPa, Poissons ratio $v$ = 0.3. Simplifications have been applied in the bridge modelling, such as the exclusion of a handrail and wearing layer. The omission of these components results in a lower overall weight, which is compensated for in the model calibration. Additionally, the bridge is supported by four bearings at its permanent location. In the FE model, these bearings are represented as boundary conditions at corresponding positions: two roller supports at one end and two fixed supports at the opposite end. The model is constructed using mesh elements, the largest of which are 720 by 300 mm. Figure 8 provides a cross-sectional perspective of the model without a diaphragm at the bridge's terminus.

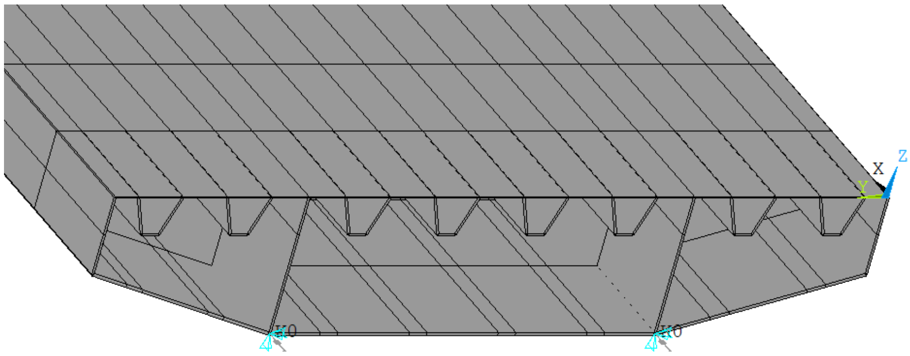

*Figure 8: Perspective of cross-section of the model without diaphragms.*

Subsequently, a COMBIN 14 element with a MASS element is connected to a node in the middle of the bridge deck. COMBIN14 is a spring with a damper system (ANSYS, 2024). The spring has a stiffness of 687588 N/m, the damping coefficient is 1984 Ns/m, and the mass is 3710 kg. Next, harmonic analysis has been performed with and without TMD. In the harmonic analysis with TMD, it is expected that two peaks will be visible: the first induced by the TMD and the second by the structure itself. Without TMD, one peak is expected. The results of the harmonic analysis can be seen in Figure 9. The results in Figure 9 have been normalized to the maximum value of the harmonic analysis results of the model without TMD.

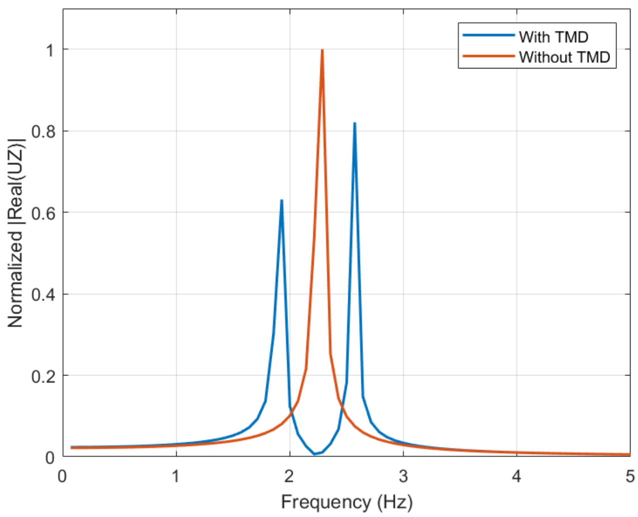


*Figure 9: Normalized harmonic analysis results of the FE model with and without TMD*

The harmonic analysis with TMD shows that these two peaks belong to the first 2 mode shapes. Without TMD, one peak is clearly visible. The y-axis in Figure 9 exhibits the absolute real values of the node in the structure where the TMD is connected during harmonic analysis. The FE model can correctly simulate the operation of a structure with TMD. Finally, a stress analysis of the FE model revealed that the highest stresses occur in the deck plate and the bottom plate at the midpoint of the bridge. Consequently, it is decided to reduce the Young's modulus for the deck plate and bottom plate over the central 11,58 meters of the bridge (Figure 10) for the structural deterioration study. In total, the stiffness of 1710 elements of the model is adjusted.

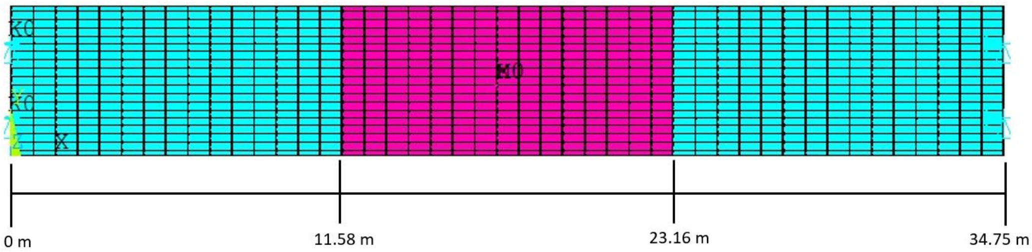


*Figure 10: Top view of the bridge model, the young's modulus is reduced for the purple elements to simulate the reduction in stiffness of the structure*

## 3.2 Model calibration

Preceding to installing the bridge, measurements of the natural frequencies were conducted with both the TMD inactive and active in the factory. The bridge was afterwards installed at its permanent location, where additional measurements were conducted. Consequently, this case study encompassed three model calibration scenarios: (1) without an active TMD in the factory, (2) with an active TMD in the factory, and (3) with an active TMD at its permanent location.

### 3.2.1 Data collection

Prior to the installation of the bridge on its permanent location in March 2024, various measurements were conducted at the factory on the bridge, both with an inactive TMD and an active TMD (Toshihisa Mano, 2023). To determine the natural frequencies, one seismic measurement system (model type ACS 1002-4 USB) was used. The measurement device was placed exactly above the TMD for the measurements. The disparity in frequencies recorded from tests conducted with inactive and active TMD was scrutinized (Figure 11). A fast Fourier transform (FFT) algorithm was employed to identify the frequencies.

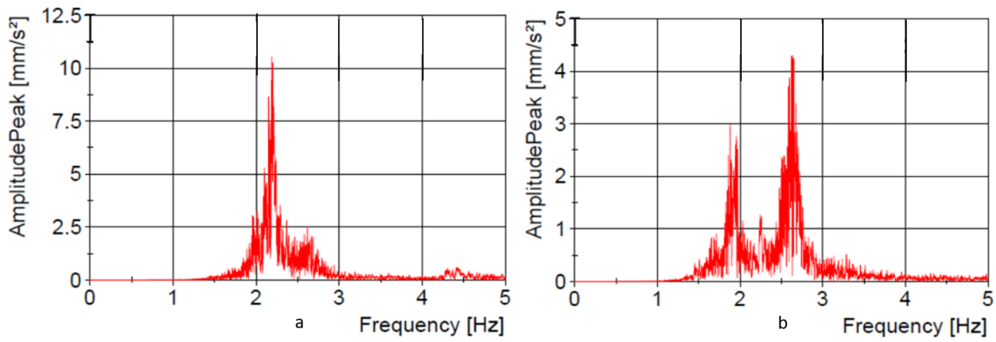


*Figure 11: FFT results in vertical direction of the measurements on F35 bridge tests in the factory with inactive TMD (a) and active TMD (b) (Toshihisa Mano, 2023)*

Figure 11a illustrates that in the absence of an active TMD, a distinct peak is observed at approximately 2.19 Hz, corresponding to the first natural frequency of the bridge. Figure 11b demonstrates the effect of a TMD with peak 1 at approximately 1.94 Hz and peak 2 at 2.64 Hz. Acceleration measurements on the permanent location of the bridge were also performed. Four accelerometers were used to capture the bridge dynamic response. Accelerometer locations on the bridge are shown in Figure 12.

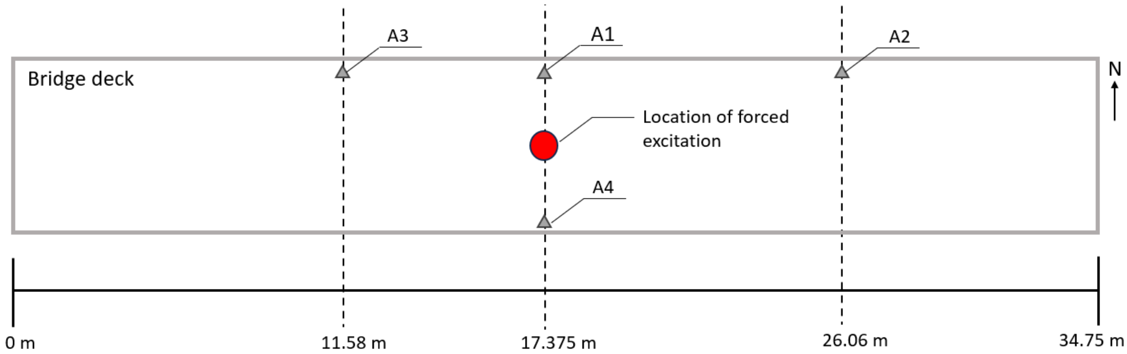


*Figure 12: Top view of the measurement setup of the bridge with, Ai (i = 1,2, .. , 4) showing the accelerometer locations*

All accelerometers are aligned in the same orientation, and only the acceleration data in the vertical direction perpendicular to the bridge deck is considered. The raw acceleration data was collected under forced excitation at the centre of the bridge (Figure 12). Subsequently, this data was processed using PSD analysis for all gauges (Figure 13).

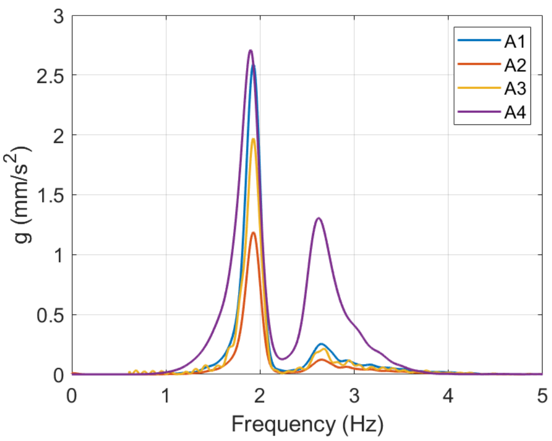


*Figure 13: PSD results of forced excitation for all accelerometers*

Figure 13 shows that there are 2 dominant frequency peaks at 1.92 Hz and 2.62 Hz for all accelerometers, which indicates the operation of the TMD. The signal strength is notably highest at the accelerometers positioned centrally on the bridge (A1 and A4). This observation aligns with the expectation that amplitudes are greatest at the midpoint. As the gauges are positioned progressively away from this central point (first A3 and then A2), the signal strength diminishes accordingly. It is noticeable that the second peak for accelerometer A4 is much stronger than that for A1, despite both being positioned in the middle of the bridge on opposite sides. This discrepancy could potentially arise from a measurement error or a forced excitation that may not have been perfectly centred on the bridge.

### 3.2.2 Model updating

The uncalibrated model without TMD estimates a natural frequency of 2.28 Hz (Figure 9). The measured natural frequency with an inactive TMD in the factory is 2.19 Hz (Figure 11a). The model is calibrated by adjusting the mass through increasing the density of the steel elements with 5.76%. The new natural frequency of the model becomes 2.19 Hz. The weight of the welds, handrail and inactive TMD is not included in the model. This makes it a rational choice to increase the mass of the bridge by 5.76 % to compensate for the lack in mass. Furthermore, the density does not impact the deterioration study, as it has no effect on the stiffness factors.

The measured values of the bridge with an active TMD indicate a frequency of approximately 1.94 Hz for peak 1 and a frequency of 2.64 Hz for peak 2 (Figure 11b). The model with an active TMD and increased weight exhibits a first peak frequency of 1.92 Hz and a second peak frequency of 2.51 Hz. This indicates that the model has not yet achieved the desired $MAD$ value. For this purpose, the spring stiffness in the model is increased by 9.5% to 76000 $N/m$ ensuring that the $MAD$ is less than 2.5%. Calibration of the model with the bridge on its permanent location is unnecessary as the values exhibit minimal change compared to the bridge with active TMD in the factory. This outcome was anticipated due to the short interval between measurements conducted at the factory and those at the permanent location.

# 4 Structure alignment analysis

The calibrated model was employed to simulate how spring and structure deterioration affect the alignment between the TMD and the bridge. This was conducted according to the five scenarios in Table 2. The results of all simulations are shown in Figure 14. Column 1 illustrates the frequency shift of the two peaks across the scenarios, while column 2 depicts the corresponding changes in amplitude of the peaks. Column 3 presents the normalized harmonic analysis results for a given scenario, compared to the original model incorporating the spring and structure at full stiffness capacity.

For scenario KST80, the frequency of peak 1 decreases by 4%, and the frequency of peak 2 decreases by 1.3% (Figure 14a). At 80% of the total steel stiffness capacity, the amplitude of peak 2 increases by 6%, while the amplitude of peak 1 decreases by 8% (Figure 14b). Figure 14c shows that the frequencies of the peaks shift slightly to the left and that peak 2 becomes slightly more dominant compared to the original situation.

For KSP80, the spring stiffness was gradually decreased by 20% from 760 $kN/m$ to 608 $kN/m$ (Figure 14d and Figure 14e). A decrease in spring stiffness results in both peaks getting a lower natural frequency (Figure 14d), with a greater impact observed on peak 2. At 80% of the original spring stiffness, the natural frequency of peak 1 decreased by 4.5%, while peak 2 decreased by 7%. At 80% of the spring stiffness capacity, changes are observed in the amplitudes of both peaks (Figure 14e). The amplitude of peak 1 increases by 15% at 80% capacity, while the amplitude of peak 2 decreases by 22% at 80% capacity. The harmonic analysis results indicate a slight leftward shift in the frequencies of the peaks, with an increase in the prominence of peak 1 compared to the original situation (Figure 14f). Following, the effect of spring stiffness deterioration and steel deterioration in the middle of the bridge at the same time is investigated (scenario KSP80 & KST92 in Figure 14g-i, scenario KSP80 & KST85 in Figure 14j-l).

The alignment deteriorates when compared to the optimally tuned condition. The results indicate that changes in alignment lead to increased amplitudes of the bridge with the TMD. This corresponds with the expectations. From the results it appears that the alignment between structure and TMD is more dependent on the spring stiffness of the TMD than on the stiffness of the structure. The results indicate that a decrease in spring stiffness (KSP80) causes peak 1 to increase in amplitude, while the amplitude of the bridge (peak 2) decreases (Figure 14d-f). A decrease in the structure's stiffness (KST80) produces the opposite effect (Figure 14a-c). In both cases the frequency peaks have a lower frequency. In scenario KSP80 & KST92 and scenario KSP80 & KST85 the effect of deterioration is applied to both the spring and the structure. Although the alignment deteriorates in these scenarios, the effects of spring and structure deterioration do not add up cumulatively. It can be observed that KSP80 causes the peak of the bridge to decrease by more than 20% (Figure 14e-f). However, for scenario KSP80 & KST85, the decrease in the bridges peak (peak 2) is less than 20% (Figure 14k-l). This may be attributed to the different phases of the new natural frequencies, indicating that the TMD and bridge do not amplify each other and may even dampen each other. However, further research is required to draw definitive conclusions.

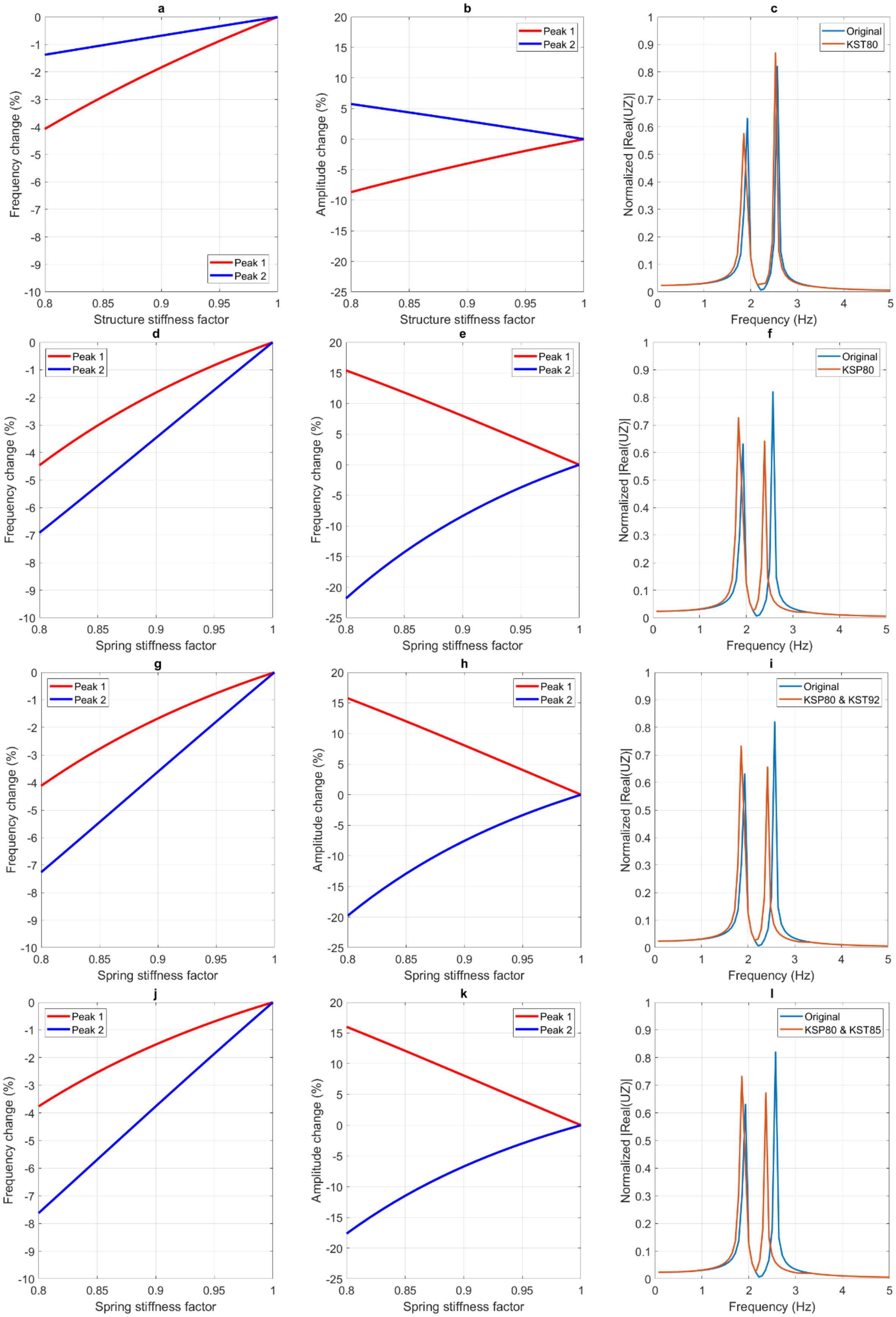


*Figure 14: KST80 (a), KSP80 (d), KSP80 & KST92 (g), KSP80 & KST85 (j) change of frequency and KST80 (b), KSP80 (e), KSP80 & KST92 (h), KSP80 & KST85 (k) change of amplitude and KST80 (c), KSP80 (f), KSP80 & KST92 (i), KSP80 & KST85 (l) normalized harmonic analysis results of scenarios compared with original scenario*

# 5 Discussion

The modelling and analysis approach applied to the case study has led to the development of an FE model that accurately simulates the working principle of a bridge equipped with a TMD. It provides insights into changes in amplitudes and shifts in frequencies caused by a decrease in stiffness in the bridge and TMD springs. The validity of the model is supported by measurements taken from the actual bridge on-site. However, the model remains a representation of reality. This indicates that simplifications have been applied, with varying levels of impact on the model's outcomes. This results in points of discussion regarding the modelling and analysis approach.

- The modelling and analysis approach (Figure 1) is intended for use from the design phase through the service life of the bridge. However, it has become apparent that after creating the parametric model, model calibration is necessary due to deviations between the measured values and the model estimations. The parametric model could be enhanced by limiting the amount of simplifications, e.g., by adding operational and environmental loads present on the bridge and by modelling the bearings with greater complexity than fixed supports. By incorporating details and reducing simplifications in the model, accuracy is expected to improve. However, it is advisable to evaluate which details have the greatest impact on the estimations and are therefore most important.
- The model with TMD has been updated by increasing the spring stiffness by 9.5%, which is a significant adjustment. This substantial increase may be attributed to modelling choices. The TMD has been modelled with one spring, while in reality, the TMD is constructed with multiple springs. This might affect the estimations since the interaction and combined effect of multiple springs can influence the TMD's overall stiffness and damping characteristics. Adjusting the spring stiffness helps to account for this simplification and better align the model with the actual performance. The model also incorporates simplified boundary conditions (i.e., roller and fixed supports), whereas, in reality, the bearings of the bridge perform complex functions. Additionally, the structure-soil interaction has not been included, which can also influence the dynamics of the bridge.
- The application of deterioration factors is arbitrary. A straightforward moment analysis for a beam indicates that stresses are greatest at the centre of the span, making it logical that this is where the likelihood of stiffness reduction is the highest. The deterioration is currently represented by a single parameter (Young's modulus), while this is more complex in a real bridge than modelled in this study. In such scenario, the stiffness of the structure would not be uniformly distributed in its centre as modelled in the case study. Further research into stiffness loss in steel pedestrian bridges is needed to enhance the accuracy of the model.
- The model updating is based on changing parameters of a single or double DOF system, mass ($m$), spring ($k$), and damper ($c$) system. In the single DOF system, the mass was increased to align with reality and change the natural frequency. Stiffness adjustment was omitted due to the bridge's recent construction and the absence of anticipated deterioration. However, for updating models of older bridges, stiffness adjustment could be viable. In the 2DOF system, model updating focused on adjusting spring stiffness, which is logical because TMD mass does not change over time and dampers have minimal influence on natural frequency. Updating based on these three parameters is effective but limited, as it does not fully capture real-world conditions. Reality is complex and more factors exert their influence on the natural dynamics of a bridge. Additional research into other factors for model updating is recommended.

# 6 Conclusions and recommendations

Bridges constructed with a TMD are complex systems, whose effectiveness relies on the alignment between the bridge and the TMD. The effectiveness of the TMD can be negatively influenced by structural changes caused by material deterioration. The TMD is tuned to the bridge's original natural frequency. As both the TMD and the bridge age, these natural frequencies change due to deterioration. Consequently, the TMD becomes detuned due to shifting natural frequencies. To restore optimal performance, the TMD must be retuned. Applying the proposed approach to the case study yields the following conclusions.

- The natural frequency of the bridge in the case study can be estimated with an $MAD$ less than 2.5% after applying model calibration: (1) before and (2) after installation of the TMD, and (3) on the permanent location of the bridge. This confirms the suitability of the model for the study.
- Structural deterioration has a negative effect on the alignment between the bridge and the TMD. With a decrease in structural stiffness, the first two natural frequencies (peak 1 and peak 2) decrease, and the vibration amplitude of the structure (peak 2) increases and the vibration amplitude resulting from the TMD (peak 1) decreases.
- Spring stiffness deterioration has a negative effect on the alignment between the bridge and the TMD. With a decrease in spring stiffness, the first two natural frequencies (peak 1 and peak 2) decrease, the amplitude of peak 2 decreases while the amplitude of peak 1 increases.
- In the case of both structural and spring stiffness deterioration occurring simultaneously, their combined effect on the alignment between the bridge and TMD do not add up cumulatively. The spring stiffness deterioration exerts a greater influence on the alignment between the bridge and the TMD compared to structural deterioration.

Knowledge of dynamic changes in a bridge equipped with a TMD, combined with an appropriate model, could support maintenance efforts, and increase its life cycle. However, it necessitates additional elements to be incorporated into the model. Precise dynamic loads are required to accurately simulate accelerations and displacements to find when limits are exceeded. Monitoring the spring or the stiffness of the bridge allows for predicting when the spring of the TMD needs to be retuned. Timely intervention can prevent the structure from failing to meet requirements or causing damage. It is recommended to conduct research on improving the parametric model without model calibration. This will further enhance the applicability of the approach during the design phase. Further research is also recommended on integrating deterioration factors in structures and TMDs, exploring their potential for enhanced accuracy within a parametric model. Finally, it is recommended to explore the potential of this approach to contribute to maintenance activities that extend the service life of bridges.

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