1. Introduction

Damage in a structure affects its performance adversely and can cause excessive stresses, displacements and vibration in the structure which in turn negatively affect the safety and serviceability of the structure. Hence, it is important to detect the damage in a structure at its onset and carry out appropriate retrofitting to prevent sudden structural failure. In such situations, Structural Health Monitoring (SHM) is a recommended solution which includes both local and global methods of damage detection and involves the use of in-situ, continuous or regular measurement and analysis of key structural and environmental parameters under operating conditions to inform abnormal stages or accidents at an early stage to avoid fatalities as well as to provide maintenance advice (Shih, Thambiratnam, & Chan, 2009). SHM can be categorized into four levels, level 1: detection of the presence of damage, level 2: location of the damage, level 3: quantification of the severity of the damage, level 4: diagnosis of the existence and location of damage (Zhou, Sparling, & Wegner, 2005). The dynamic characteristics of a structure such as natural frequencies and mode shapes are functions of its stiffness and mass distribution and changes in modal frequencies and mode shapes are an effective indication of structural deterioration (Shih et al., 2009). Frequencies of a structure can be accurately measured, but they are not very sensitive to local damage (Zhou, Xia, & Weng, 2015). Some well-known VBDD methods include modal flexibility, modal strain energy, mode shape curvature, uniform load surface curvature, stiffness change, etc. (Shih et al., 2009).

Vibration Based Damage Detection methods (VBDDs) have been successfully applied to rotating machinery (Shives & Mertaugh, 1986), well-defined aerospace or mechanical systems (Hunt, Weiss, West, Dunlap, & Freemeyer, 1990), and simple structures such as beams or trusses (Pandey & Biswas, 1994). More recently, Shih et al. (2009) used the modal flexibility and modal strain energy methods of VBDD techniques to determine damage in beam and plate and Zhou et al. (2015) assessed damage assessment in beams through frequency data analysis. In addition, several researchers (Roy, Chakraborty, & Sarkar, 2006; Shih, Thambiratnam, & Chan, 2013; Siddique, Wegner, & Sparling, 2005; Siddique et al., 2006; Tan, Thambiratnam, Chan, Gordan, & Razak, 2019; Zhou, Wegner, & Sparling, 2007; Zhou et al., 2005) utilized VBDD...
methods to detect damage in slab-on-girder bridges of single and multiple spans. The bridges selected by the researchers were composed of concrete deck and concrete/steel girders.

Similarly, Panesso, Marulanda, and Thomson (2019) and Kostic and Gul (2017) adopted VBDD method for locating damage in steel truss pedestrian bridges under constant temperature and varying temperature, respectively. Ali, Sandhu, and Usman (2019) used the same method to detect damage in a precast double-tube composite pedestrian bridge. Shih et al. (2013) and Tan et al. (2019) found flexure dominant vibration modes for steel slab-on-girder bridge and steel-concrete slab-on-girder bridge, respectively. Ghosh and Karbhari (2012) used VBDD method for progressive damage assessment in slab-on-girder bridge strengthened with Fiber Reinforced Polymer (FRP).

Researchers have also used other methods to detect damage in pedestrian bridges. For instance, Zhang, Wu, Li, and Chen (2020) considered long-gauge strain measurements by using fiber bragg grating sensors to detect damage in a simply supported concrete pedestrian bridge. Azam, Rageh, and Linzell (2018) used proper orthogonal modes to detect and locate damage in a steel truss pedestrian bridge. Furthermore, a model-free damage detection method was used by Ruffels, Gonzalez, and Karoumi (2020) for a steel pedestrian arch bridge. Damage detection of a movable steel pedestrian bridge using hybrid sensor camera monitoring was carried out by Zaurin, Khuc, and Catbas (2016). Weinstein, Sanayei, and Brenner (2018) considered artificial neural networks to identify damages in a composite pedestrian bridge which consisted of concrete deck and three steel girders.

The literature informs that well known VBDD techniques and other methods were considered by the past researchers for identifying damages in the conventional pedestrian bridges. However, there is scope to explore the suitability of the popular VBDD techniques to detect and locate damage in the proposed innovative composite pedestrian bridges. This paper describes a combined numerical and laboratory-based experimental study undertaken to ascertain the potential of a VBDD technique for detecting and locating low levels of damage in the girder and deck of an innovative light and slender pedestrian bridge. Pedestrian bridges using light deck materials have received much attention over the past few years. Researchers have proposed new light deck materials such as aluminum (Dey, Walbridge, & Narasimhan, 2015), Fiber Reinforced Polymer (FRP) (Votsis, Stratford, Chryssanthopoulos, & Tantele, 2017), Glass Fiber Reinforced Polymer (Sa, Guerreiro, Gomes, Correia, & Silvestre, 2017; Wei, Russell, Zivanovic, & Mottram, 2017) and evaluated the performances of the bridge deck. In this study, a new bi-layer composite deck is considered by introducing GRFP at the bottom layer and laminated glass at the top layer of the bridge deck. Laminated GRFP is selected due to the advantages of light weight, ease of installation, aesthetics and being economical. Laminated glass is prepared using two pieces of glass with an interlayer of adhesive such as polyvinyl butyral. This material is easy to install, durable, tough and transparent.

The use of such type of composite deck in a pedestrian bridge was found to be remarkably effective in controlling vibration compared to that in a pedestrian bridge with deck made up of conventional materials (Ali, Thambiratnam, Fawzia, Nguyen, & Leung, 2020). The bi-layer composite deck is prepared of flexible material (high tensile strength) in the bottom layer and tough material (high compressive strength) in the top layer. Due to using different materials in the two different layers of the bridge deck, the structural behavior of the pedestrian bridge in both the healthy and damaged states might be different compared to that of conventional pedestrian bridges. This implies that the vibration characteristics of such bridges can also be different. Hence, research is required to test the effectiveness of VBDD methods for pedestrian bridges with this innovative composite deck to assist the structural health monitoring of such new type of pedestrian bridges, when introduced in the future. The present study uses the simple but effective Modal Flexibility (MF) method to detect and locate the damage in the proposed slab-girder pedestrian bridge composed of GFRP-laminated glass bi-layer deck and supported by a steel girder. The application of the approach is demonstrated through experimental and numerical studies under single and multiple damage scenarios.

2. Modal flexibility matrix

Modal flexibility matrix is used to determine the change in modal flexibility which is then used to identify the location of damage. It includes the influences of both mode shapes and natural frequencies and is the accumulation of the contributions from all available mode shapes and corresponding natural frequencies. The modal flexibility matrix associated with the referenced degrees of freedom is expressed by the following equation:

$$[F] = [\varphi][1/\omega^2][\varphi]^T \quad (1)$$

In the above equation, $[F]$ is the modal flexibility matrix; $[\varphi]$ is the mass normalized modal vector; and $[1/\omega^2]$ is a diagonal matrix containing the reciprocals of the squares of natural frequencies in ascending order.

The modal contribution to the flexibility matrix decreases as the frequency increases, i.e., the flexibility matrix converges rapidly with increasing values of frequency. From only a few of the lower frequency modes, therefore a good estimate of the flexibility can be made. The change in the flexibility matrix $\Delta[F]$ due to structural deterioration is given by

$$\Delta[F] = [F^d] - [F^h] \quad (2)$$

where indices ‘h’ and ‘d’ refer to the healthy and damaged states, respectively. Theoretically, structural deterioration reduces stiffness and increases flexibility. Increase in structural flexibility can therefore serve as a good indicator of the degree of structural deterioration (Shih et al., 2009).
3. Experimental study

The experimental procedure consisted of measuring the dynamic properties (mode shapes and natural frequencies) of the innovative slab-girder pedestrian bridge specimen in its healthy state and then incrementally inducing new states of damage and measuring the similar dynamic properties associated with each state. The structural performance of the similar slab-on-girder pedestrian bridge was previously evaluated by Ali et al. (2020). Several impact vibration tests were carried out to measure the dynamic properties of the undamaged and damaged specimens of the bridge.

3.1. Description of innovative slab-on-girder pedestrian bridge

The bi-layer composite slab of the innovative slab-on-girder pedestrian bridge specimen was prepared with laminated GFRP in the bottom layer and laminated glass in the top layer of slab. The length and width of the slab were 1 m and 400 mm respectively while the overall thickness of the composite slab was 16.5 mm.

The thicknesses of laminated GFRP layer and laminated glass layer were 10 mm and 6.5 mm respectively in both specimens. A single 150 UB steel beam with total height of 155 mm and flange width of 75 mm was used to support the composite slab. The web thickness and the flange thickness of the steel beam were 8 mm and 9.5 mm, respectively. High strength adhesive was used to fix the steel beam to the bottom of the steel beam as shown in the Figure. Clamps were used to connect the bottom flange of the steel beam to the aluminum blocks. This set-up enabled to resist movement of the bridge specimen in the longitudinal and transverse directions, but it was not capable of resisting external moments and hence the it was considered as a hinged boundary condition (as also validated by simple support conditions in the FE model).

3.1.1. Impact vibration testing of undamaged slab-girder pedestrian bridge specimen

Two identical slab-girder pedestrian bridge specimens were selected to carry out the impact vibration tests initially under undamaged condition. Figure 1 also shows the experimental set up for impact vibration test on the slab-girder pedestrian bridge specimens. In total, fifteen accelerometers were mounted on the top surface of the composite deck in three rows as shown in Figure 1. Figure 2 depicts the spacing of the accelerometers on the composite deck of the pedestrian bridge specimen.

Each accelerometer was mounted using a small square steel plate which was attached with a magnet (Figure 3(a)). Cable RH50 was used to connect each accelerometer to the Data Acquisition System (NI 9234) (Figure 3(b)). Rubber tapping was used in the impact vibration tests and a rubber hammer was used to apply the taps on the top surface of slab-girder specimen in two different ways. At first, taps were applied only in one location on the top surface of the deck as shown in the left picture of Figure 3(c) for around 60 secs at an interval of 2 sec between two taps. Later, taps were applied at multiple locations on the top surface of the composite deck for around 60 sec following the direction of arrow as shown in the right picture of Figure 3(c). The signals from the accelerometers were recorded by the Data Acquisition System (NI 9234) which was used to extract the natural frequencies and the corresponding mode shapes of the slab-girder bridge specimen after converting through ARTeMIS software.

Table 1 presents the mode shapes and the corresponding natural frequencies for the two slab-girder bridge specimens obtained from the impact vibration tests. Figure 4 shows the first five modes of vibration of the bridge. It is interesting to note that all the first five modes were torsional in nature and the absence of the usual flexural modes. This feature is different from that in other slab-girder bridges which exhibited flexure dominant early modes of vibration (Shih et al., 2013; Tan et al., 2019). This is possibly because of the proposed flexible-stiff composite deck of the bridge where flexible GFRP laminated plate and stiff laminated glass were used in the bottom and top layers of the composite deck, respectively.

3.1.2. Impact vibration testing of damaged slab-girder pedestrian bridge specimen

Damage in the form of irregularities on the composite deck of the bridge specimens were induced by placing lumped masses in one or two locations for treating single and double damages, respectively. In total, six different damage scenarios, as shown in Figures 5(a–f), were considered and the dynamic response (mode shapes and natural frequencies) of the bridge specimens were determined through impact vibration tests for each damage case. Damage was treated using circular lumped masses with diameter of 75 mm, height of 25 mm and weight of around 1 kg. Numbers and locations of lumped masses were varied for the selected six damage scenarios.

Figures 5(a), (d) and (e) display single damage cases with a single lumped mass placed on the sideline in 5(a) and 5(d) and a single lumped mass on the centerline in 5(e), placed on the top surface of the composite deck. Figures 5(b), (c) and (f) present double damage cases with two lumped masses on the sidelines in 5(b) and 5(c) and a lumped mass each on the sideline and the centerline in 5(f), placed on top surface of composite deck. The instrument set up and accelerometer arrangement were similar as mentioned for the undamaged slab-girder bridge specimen. Tables 2–7 shows the mode shapes and the corresponding natural frequencies of two identical damaged composite bridge specimens under different single and double damage scenarios. The natural frequencies of the innovative slab-on-girder pedestrian bridge model are higher than those of highway (truck) bridges. This is due to the relatively shorter
span (1 m) of the innovative slab-on-girder pedestrian bridge model in the impact vibration tests.

The results in the Tables illustrate that the values of the natural frequencies differ a little between the 2 specimens. Average values will be taken for the damage detection. This damage bridge and all subsequent damaged bridges exhibited torsion dominant early modes of vibration, similar to those in the undamaged bridge, and different from other (traditional) slab-girder bridges which exhibited flexure dominant early modes (Shih et al., 2013; Tan et al., 2019).

4. Finite element analysis

4.1. FE analysis & validation of undamaged slab-girder bridge specimen

A three-dimensional FE model of the undamaged slab-girder bridge specimen as shown in Figure 6 was developed in ABAQUS to compare the FE and experimental results of the impact vibration tests described in section 2 and hence to validate the innovative slab-girder bridge model. The FE model was built up with three parts to represent the bottom GFRP layer and the top laminated glass layer of composite deck and the steel beam. The two layers of the composite deck and the steel beam were considered as solid bodies and three-dimensional linear brick stress elements (C3D8R) were used to these parts of the bridge model. Material properties as presented in Table 8 were used in material modelling of GFRP, laminated glass and steel beam. Full connection was defined between the two layers of the composite deck and also between the bottom layer of composite deck and the steel beam.

Boundary conditions were defined in the FE model of the bridge following test conditions as shown in Figure 5 by applying hinge supports at the regions of the bottom flange of the steel beam that were directly adjacent to the aluminium blocks (Figure 1). Mesh sizes of 10 mm and 7.5 mm were selected for the laminated GFRP and laminated glass layers, respectively and 9 mm mesh size was found suitable for the steel beam after mesh convergence study. Table 9 compares the natural frequencies of the slab-girder composite bridge model obtained from the FE analysis and the impact vibration tests on two identical slab-girder bridge specimens.

The deviations of FE results from test results of samples 1 were found to be 0%, 8.4%, 3.8%, 1.5% and 3.3% respectively for the first five modes, while those from the test results of sample 2 were found to be 8.3%, 5%, 7.9%, 4.2% and 1.3% respectively for the first five modes. In addition, the mode shapes from the present FE model (Figure 7) agreed well with those from the self-performed impact vibration tests (Figure 4) on the composite bridge specimens. Torsion dominant mode shapes were found for the first five modes from both impact vibration tests and FE analysis results. As all the deviations of the FE results for natural frequencies from the corresponding test results are less than 10%, and the mode shapes agree reasonably well, the FE model of bridge can be considered to be adequate for predicting its response in damage detection studies to follow.

4.2. Validation of damaged steel beam specimen

4.2.1. Experimental investigation

Two identical damaged 150 UB steel beam specimens were considered to carry out impact vibration tests. A 6 mm cut was made across the width of the bottom flange of the steel beam.
beam at its mid-span. The 150 UB steel beams had total depth of 155 mm and flange width of 75 mm. The web thickness and the flange thickness of the steel beams were 8 mm and 9.5 mm respectively. The steel beam specimens were simply supported by hollow aluminum blocks (400 mm × 75 mm × 75 mm) as shown in Figure 8 and clamps were used to connect the bottom flange of the steel beams to the aluminum blocks to resist their uplift.

Figure 8 also depicts the experimental set up for impact vibration test on the steel beam specimens. In total, ten accelerometers were mounted on the top surface of each steel beam in two rows as shown in Figure 8. The accelerometers were simply supported by hollow aluminum blocks (400 mm × 75 mm × 75 mm) as shown in Figure 8 and clamps were used to connect the bottom flange of the steel beams to the aluminum blocks to resist their uplift.

Table 1. Mode shapes and natural frequencies of undamaged slab-girder bridge specimens.

| Specimen 1          | Specimen 2          |
|---------------------|---------------------|
| Mode no. | Mode shape | Natural frequency (Hz) | Mode no. | Mode shape | Natural frequency (Hz) |
| 1       | Torsional  | 18                  | 1       | Torsional  | 16.5                |
| 2       | Torsional  | 57                  | 2       | Torsional  | 55                  |
| 3       | Torsional  | 82                  | 3       | Torsional  | 78.5                |
| 4       | Torsional  | 106                 | 4       | Torsional  | 109                 |
| 5       | Torsional  | 196                 | 5       | Torsional  | 192                 |

Figure 3. Impact vibration test on composite deck (a) mounting of accelerometers (b) DAQ (c) Circles showing tap locations and arrow showing tapping sequence (one location-left, multiple locations-right).

4.2.2. FE analysis and validation

A three-dimensional FE model of the damaged steel beam specimen was developed in ABAQUS to compare the FE and experimental results of the impact vibration tests described in the above sub-section and hence to validate the damaged steel beam model. The FE model of the damaged steel beam was considered as a solid body and
10-node quadratic tetrahedron (C3D10) elements were used to model the beam. A cut at mid-span of beam following the test condition was provided in the FE model as shown in Figure 9. Material properties of steel as presented in Table 8 were used in the material modeling of the steel beam.

Boundary conditions were defined in the damaged steel beam FE model following test conditions as shown in Figure 8 by applying hinge in the regions of the bottom flange of the beam directly adjacent to the aluminium blocks (Figure 1). Mesh size of 15 mm was selected for the damaged steel beam after mesh convergence study. Figure 10 compares the mode shape of fundamental mode of the damaged steel beam model obtained from the FE analysis and the impact vibration tests on two identical damaged steel beam specimens.
Figure 5. Damage scenarios selected for slab-girder bridge specimens (a) first (b) second (c) third (d) fourth (e) fifth (f) sixth damage scenarios.

Table 2. Mode shapes and natural frequencies of damaged slab-girder bridge specimens in damage scenario 1.

| Specimen 1 | Specimen 2 |
|------------|------------|
| Mode no.   | Mode shape | Natural frequency (Hz) | Mode no.   | Mode shape | Natural frequency (Hz) |
| 1          | Torsional  | 16                      | 1          | Torsional  | 13                      |
| 2          | Torsional  | 51.5                    | 2          | Torsional  | 51.5                    |
| 3          | Torsional  | 82                      | 3          | Torsional  | 77.5                    |
| 4          | Torsional  | 105.5                   | 4          | Torsional  | 98                      |
| 5          | Torsional  | 185                     | 5          | Torsional  | 180                     |
It is evident from Figure 10 that similar types of mode shapes were obtained from the FE analysis and impact vibration test. The upper flange shown in green lines in Figure 10 and the web of the steel beam were found to exhibit a coupled lateral-torsional mode during its fundamental natural frequency, as also shown in Figure 10a. The fundamental natural frequency was found to be 88 Hz and 86 Hz for specimens 1 and 2 respectively from the impact vibration tests, whereas from the FE analysis it was found 82 Hz. The deviation of FE results from the test results of specimens 1 and 2 are hence 6.8% and 4.8% respectively. Overall, the FE results of the damaged steel beam model were found to be in good agreement with the test results. Hence, the steps adopted to establish the damaged model can be considered as accurate and the same procedure can be applied for determining the response of composite bridge with the damaged beam.

### 4.3. Dynamic response of damaged slab-girder pedestrian bridge model

The strategies carried out in section 3.1 and sub-section 3.2.2 for FE model validation were followed to prepare the FE model of slab-girder pedestrian bridge under different damage scenarios. The next sub-sections discuss the dynamic response

| Table 3. Mode shapes and natural frequencies of damaged slab-girder bridge specimens in damage scenario 2. |
| --- | --- |
| Specimen 1 | Specimen 2 |
| Mode no. | Mode shape | Natural frequency (Hz) | Mode no. | Mode shape | Natural frequency (Hz) |
| 1 | Torsional | 15 | 1 | Torsional | 12.5 |
| 2 | Torsional | 52.5 | 2 | Torsional | 49.5 |
| 3 | Torsional | 82 | 3 | Torsional | 77.5 |
| 4 | Torsional | 100 | 4 | Torsional | 95 |
| 5 | Torsional | 159.5 | 5 | Torsional | 148 |

| Table 4. Mode shapes and natural frequencies of damaged slab-girder bridge specimens in damage scenario 3. |
| --- | --- |
| Specimen 1 | Specimen 2 |
| Mode no. | Mode shape | Natural frequency (Hz) | Mode no. | Mode shape | Natural frequency (Hz) |
| 1 | Torsional | 16 | 1 | Torsional | 13 |
| 2 | Torsional | 52.5 | 2 | Torsional | 52 |
| 3 | Torsional | 81.5 | 3 | Torsional | 76.5 |
| 4 | Torsional | 104.5 | 4 | Torsional | 102.5 |
| 5 | Torsional | 176.5 | 5 | Torsional | 165.5 |

| Table 5. Mode shapes and natural frequencies of damaged slab-girder bridge specimens in damage scenario 4. |
| --- | --- |
| Specimen 1 | Specimen 2 |
| Mode no. | Mode shape | Natural frequency (Hz) | Mode no. | Mode shape | Natural frequency (Hz) |
| 1 | Torsional | 16.5 | 1 | Torsional | 13 |
| 2 | Torsional | 54 | 2 | Torsional | 53 |
| 3 | Torsional | 82 | 3 | Torsional | 76.5 |
| 4 | Torsional | 101 | 4 | Torsional | 103 |
| 5 | Torsional | 194.5 | 5 | Torsional | 190.5 |

| Table 6. Mode shapes and natural frequencies of damaged slab-girder bridge specimens in damage scenario 5. |
| --- | --- |
| Specimen 1 | Specimen 2 |
| Mode no. | Mode shape | Natural frequency (Hz) | Mode no. | Mode shape | Natural frequency (Hz) |
| 1 | Torsional | 16.5 | 1 | Torsional | 12 |
| 2 | Torsional | 56 | 2 | Torsional | 55 |
| 3 | Torsional | 82 | 3 | Torsional | 76 |
| 4 | Torsional | 105 | 4 | Torsional | 109 |
| 5 | Torsional | 193 | 5 | Torsional | 191.5 |

| Table 7. Mode shapes and natural frequencies of damaged slab-girder bridge specimens in damage scenario 6. |
| --- | --- |
| Specimen 1 | Specimen 2 |
| Mode no. | Mode shape | Natural frequency (Hz) | Mode no. | Mode shape | Natural frequency (Hz) |
| 1 | Torsional | 16.5 | 1 | Torsional | 10.5 |
| 2 | Torsional | 54 | 2 | Torsional | 52.5 |
| 3 | Torsional | 82 | 3 | Torsional | 73.5 |
| 4 | Torsional | 102 | 4 | Torsional | 104 |
| 5 | Torsional | 187.5 | 5 | Torsional | 190.5 |
4.3.1. Single damage in girder of slab-girder pedestrian bridge model

Damage scenario one was designed by considering a 6 mm cut across the bottom flange of the steel girder at the mid span of the innovative slab-girder pedestrian bridge (Figure 6). (mode shapes and natural frequencies) of the slab-girder pedestrian bridge model under the considered damage scenarios.

Figure 6. Slab-girder bridge specimen (a) FE model (b) FE meshing.

Table 8. Material properties of slab-girder bridge specimen.

| Material     | Thickness (mm) | Density (kg/m³) | Elastic modulus (GPa) | Poisson’s ratio |
|--------------|----------------|-----------------|-----------------------|----------------|
| GFRP         | 10             | 1800            | 2                     | 0.30           |
| Laminated glass | 6.5           | 2500            | 23                    | 0.22           |
| Steel        |                | 7680            | 200                   | 0.20           |

Table 9. Experimental and FE results of impact vibration analysis.

| Type of Mode | Sample 1 | Sample 2 | FE analysis | Deviation (%) | Deviation (%) |
|--------------|----------|----------|-------------|---------------|---------------|
| First mode   | 18       | 16.5     | 18          | 0             | 8.3           |
| Second mode  | 57       | 55       | 52.2        | 8.4           | 5             |
| Third mode   | 82       | 78.5     | 85.3        | 3.8           | 7.9           |
| Fourth mode  | 106      | 109      | 104.4       | 1.5           | 4.2           |
| Fifth mode   | 196      | 192      | 189.43      | 3.3           | 1.3           |
Material properties mentioned in Table 8 were used for the material modeling of the composite bridge. The two layers of the composite deck and the steel beam were considered as solid bodies and three-dimensional linear brick stress elements (C3D8R) and 10-node quadratic tetrahedron (C3D10) elements were used to model these layers and the damaged steel girder, respectively. Mesh size was selected according to the validated FE models mentioned in section 3.1 and sub-section 3.2.2 and hence mesh size of 10 mm and 7.5 mm were selected for the GFRP and laminated glass layers, respectively and 9 mm mesh size was used for the damaged steel girder. Boundary conditions were also considered to be the same as in the validated FE models where the sides of the deck were free. However, the bridge model was...
re-run by providing fixed supports at the shorter sides of deck. There were no significant differences in the magnitudes of the natural frequencies through which damages in the bridge model were identified. The mode shapes were coupled torsion dominant modes. The proposed method was yet found to be suitable for detecting and locating damages in the innovative slab-on-girder bridge model.

Figure 12 displays the first five mode shapes of the innovative slab-girder pedestrian bridge with single damage in the girder and Table 10 shows the natural frequencies of the first five modes of this damaged model. This damaged bridge model and all subsequent damaged bridge models exhibited torsion dominant early modes of vibration, similar to those in the undamaged bridge model, and different from other (traditional) slab-girder bridges which exhibited flexure dominant early modes (Shih et al., 2013; Tan et al., 2019).
4.3.2. Double damage in girder of slab-girder pedestrian bridge model

Damage scenario-two was designed by providing two 6 mm cuts across the bottom flange of the steel girder at the mid and quarter spans of the bridge model (Figure 11(b)). All other steps adopted for the present FE model of the bridge with double damage were similar to those of the damaged bridge in sub-section 4.3.1. Table 10 shows the natural frequencies of first five modes of the damaged bridge model. Mode shapes of the bridge under double damages in girder...
were found to similar globally to those of the bridge model with a single damage in girder as shown in Figure 12.

4.3.3. Single damage in deck of slab-girder pedestrian bridge model

Damage scenario-three was designed by making a 6 mm cut through the thickness of the composite deck at the mid span of the bridge (Figure 13(a)). Damage was simulated by 6 mm cut through the thickness of the composite deck at the midspan of the bridge as shown in the Figure 13(a). The damage was considered at about 150 mm on the composite deck at one side of the top flange of the steel beam. Table 10 presents the corresponding natural frequencies.
4.3.4. Double damage in deck of slab-girder pedestrian bridge model

Damage scenario-four was designed by considering two 6 mm cuts through the thickness of the composite deck at the quarter and spans in the bridge model (Figure 13(b)). The damage was considered at about 150 mm on the composite deck at one side of the top flange of the steel beam. Table 10 shows the corresponding natural frequencies.

Table 10. Natural frequencies of damaged composite bridge.

| Natural Frequency (Hz) at mode | Damage scenario |
|-------------------------------|----------------|
| 1  | 2   | 3   | 4   | 5   |
| 1  | 18  | 17.96 | 18  | 18  | 17.95 |
| 2  | 51  | 50.98 | 50.21 | 50.94 | 50.71 |
| 3  | 83.43 | 81.92 | 84.67 | 83.92 | 84.02 |
| 4  | 104.05 | 103.90 | 103.65 | 103.27 | 103.55 |
| 5  | 189.43 | 189.43 | 189.42 | 189.23 | 189.39 |

4.3.5. Damage at deck and girder of slab-girder pedestrian bridge model

Damage scenario-five was designed by making a 6 mm cut through the thickness of the composite deck at the mid span of the bridge model. The damage was considered at about 150 mm on the composite deck at one side of the top flange of the steel beam. In addition, a 6 mm cut across the bottom flange of the steel girder at quarter span of the bridge model. Figure 14 shows the damaged slab-girder pedestrian bridge model. Table 10 presents the corresponding natural frequencies. The values of natural frequencies for all the damage scenarios were found directly from the Finite Element software (ABAQUS). A small cut (only 6 mm width) was found to be adequate to obtain the natural frequencies for damage detection by modal flexibility method. Hence, the values of natural frequencies mentioned in Table 10 did not differ much from each other.
5. Damage assessment by modal flexibility change

The location and intensity of damage in the innovative slab-girder composite pedestrian bridge specimens with damage simulated by using lumped masses in the experimental study (section 2) and by making cuts in the numerical analysis (section 3) were determined using Modal Flexibility Change (MFC) as mentioned in section 1. In the experimental study, six damage scenarios were considered including single and double damages. Five damage scenarios including single and double damages were considered in the numerical study.

Figure 15(a) and (b) show the changes in modal flexibility along the bridge length obtained from the experimental and numerical studies respectively. The damage scenarios pertaining to Figure 15(a) and (b) were discussed in sections 2 and 3, respectively. The values of MFC for experimental study and numerical study as shown in Figure 15(a) and (b), respectively were calculated under different conditions. Lump masses were considered in the experimental study as it was not feasible to inflict damage in the bridge model, whereas 6 mm cuts were considered in the numerical study. Hence the results from the two different studies represent different damage scenarios and cannot be compared but show that both methods can be used simultaneously to determine damage locations with added confidence.

The first five natural frequencies and the associated mode shapes obtained from impact vibration tests and FE analyses were used along with Equations (1) and (2) to calculate the MFCs at different locations in the bridge and develop the MFC curves shown in Figure 15(a) and (b), respectively. In all damage scenarios, the peak values of the MFC curves indicate the location of damage in the bridge. It can also be seen that higher the peak, greater the damage intensity in the bridge.

Figure 15(a) was plotted for three single damage scenarios (1, 4 & 5) and three double damage scenarios (2, 3 & 6). Hence, single and double peaks in the MFC curves occur for damage scenarios 1, 4, 5 and damage scenario 2, 3, 6 respectively. In addition, the peaks were found at the damage locations (shown in Figure 5) considered for the composite pedestrian bridge which implies that the Modal Flexibility method can accurately detect the location of damage for this innovative bridge. Moreover, Modal Flexibility method also provides an indication of the severity of the damage qualitatively indicated by the values of the peaks in the different damage scenarios.

For instance, for single damage cases, Figure 2 presents that the damage severities from high to low occur in the order damage scenarios 1, 4 and 5. This is because the 2 kg weight was placed on the sideline of the composite deck in damage scenario 1 while a 1 kg weight was placed on the sideline of the deck for damage scenario 4 and a 1 kg weight...
was placed on the centerline of the deck for damage scenario 5. The steel beam supported the deck below its centerline and hence the overall stiffness of the composite bridge is higher at the centerline compared to the side which indicated the lowest relative damage severity for scenario 5. Similarly, for double damage cases, the peak value of MFC was found to be maximum for damage scenario 2, followed by damage scenario 3 and then damage scenario 6. In damage scenarios 2 and 3 two numbers of 2 kg weights and two numbers of 1 kg weights were placed on the side lines respectively, while in damage scenario 6, a 1 kg weight was placed on the centerline and another on the sideline. As discussed above, the stiffness is higher at the centerline due to the steel beam and hence the minimum value of MFC was found for this damage scenario. The Modal Flexibility method can therefore be recommended for determining the location and severity of damage in the innovative slab-girder pedestrian bridge through experimental testing.

Figure 15(b) was developed using the results of FE analysis for two single damage scenarios (1 & 3) and three double damage scenarios (2, 4 & 5). The MFC curves for damage scenarios 1 and 3 show single peaks while the MFC curves for damage scenario 2, 4, 5 show two unequal peaks. These peak values, in all damage cases occur at the damage locations considered in the bridge (as discussed in section 3). In evaluating the damage severities for the single damage cases, it was found that the MFC value was higher for damage scenario 1 compared to damage scenario 3. Damage scenario 1 pertained to a single damage in the steel beam while damage scenario 3 pertained to a single damage at one side of composite deck. The steel beam supported the composite deck and hence damage in steel beam indicates a relatively higher damage intensity in the bridge compared to damage in the composite deck. Again, the evaluation of damage severities for double damage cases revealed that the highest damage severity was indicated for damage scenario 2, followed by damage scenario 4 and then damage scenario 5.

Damage scenarios 2 and 4 are double damages in the steel beam and the composite deck respectively, whereas damage scenario 5 is single damage in the steel beam and a single damage in the composite deck. Like single damage cases, the double damage cases also indicate that damage in the steel beam is more significant and shows a higher damage severity in the bridge compared to damage in the composite deck. In addition, the higher peaks of MFC curves at mid-span compared to those at quarter span indicates that the damage at mid-span in the steel beam or the composite deck is more effective in increasing the damage severity of this composite pedestrian bridge. In damage scenario 5, one damage was considered at mid-span of the deck and the other damage was considered at quarter span of the steel beam. The second peak at mid-span is slightly higher than the first peak at quarter span location. These results once again show that the Modal Flexibility method can be effective for predicting the location and the relative severity of damage in this innovative slab-girder pedestrian bridge through numerical simulations.

6. Conclusions

This present paper presented a combined experimental-numerical approach for damage detection in an innovative composite slab-girder pedestrian bridge. Impact vibration tests were carried out on the undamaged and the damaged composite pedestrian bridge under single and multiple damage scenarios using lumped masses to simulate damage. Natural frequencies and the corresponding mode shapes obtained from the impact vibration tests were used along with the modal flexibility method to determine the location and relative severity of damage.

Numerical (FE) models of the undamaged bridge and the damaged beam specimen were developed in ABAQUS and validated using impact vibration test results. The validated modelling techniques were used to develop FE models of the bridge under single and multiple damage scenarios. To simulate damage, 6 mm cuts were made in the numerical models. Vibration responses from FE analyses and the modal flexibility method were used to determine the location and severity of the damage in the composite pedestrian bridge, under a range of damage scenarios.

The major findings from this study are as follows:

- This innovative pedestrian bridge with the new bi-layer composite deck, has early vibration modes that are dominant in torsion, both in its healthy and damaged states as shown by both experimental and numerical results. This phenomenon is different to that in slab-girder bridges made of traditional materials, in which the early vibration modes are dominant in flexure.
- Peaks of the Modal Flexibility Changes (MFC) along the length of the bridge determined using the experimental results and then the numerical results correctly predicted the damage locations in both single and multiple damage scenarios.
- The experimental study which used lumped masses to simulate damage showed that damage in the sideline of the composite deck had higher damage severity than damage in the centerline along which the steel beam supported the deck.
- The numerical study in which cuts were used to simulate damage showed that the damage in the steel beam is more significant in increasing the damage severity of the bridge compared to the damage in the composite deck which was supported by the beam.
- Numerical results inferred that damage at mid-span has higher severity compared to damage elsewhere.

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Data availability

The processed data required to reproduce these findings cannot be shared at this time due to legal reasons.
Disclosure statement
No potential conflict of interest was reported by the authors.

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