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Structural evaluation of all-GFRP cable-stayed footbridge after 20 years of service life

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Abstract. The paper presents the study on a change in modal parameters and structural stiffness of cable-stayed Fiberline Bridge made entirely of Glass Fiber Reinforced Polymer (GFRP) composite used for 20 years in the fjord area of Kolding, Denmark. Due to this specific location the bridge structure was subjected to natural aging in harsh environmental conditions. The flexural properties of the pultruded GFRP profiles acquired from the analyzed footbridge in 1997 and 2012 were determined through three-point bending tests. It was found that the Young's modulus increased by approximately 9%. Moreover, the influence of the temperature on the storage and loss modulus of GFRP material acquired from the Fiberline Bridge was studied by the dynamic mechanical analysis. The good thermal stability in potential real temperatures was found. The natural vibration frequencies and mode shapes of the bridge for its original state were evaluated through the application of the Finite Element (FE) method. The initial FE model was created using the real geometrical and material data obtained from both the design data and flexural test results performed in 1997 for the intact composite GFRP material. Full scale experimental investigations of the free-decay response under human jumping for the experimental state were carried out applying accelerometers. Seven natural frequencies, corresponding mode shapes and damping ratios were identified. The numerical and experimental results were compared. Based on the difference in the fundamental natural frequency it was again confirmed that the structural stiffness of the bridge increased by about 9% after 20 years of service life. Data collected from this study were used to validate the assumed FE model. It can be concluded that the updated FE model accurately reproduces the dynamic behavior of the bridge and can be used as a proper baseline model for the long-term monitoring to evaluate the overall structural response under service loads. The obtained results provided a relevant data for the structural health monitoring of all-GFRP bridge.

Keywords: GFRP bridge structure; finite element analysis; free-decay vibration; dynamic characteristics; dynamic mechanical analysis; structural stiffness

1. Introduction

All bridge structures in their service life are subjected to various unfavorable factors, e.g., aggressive environmental conditions, moisture, temperature, solar radiation, service loads, fatigue or chemical corrosion, which may cause deterioration of their structural materials. From the point of view of bridge engineers, this problem is one of the most important issues considering the expected lifetime of bridge structures. The proper design of such structures regarding the long-term maintenance costs requires the proper selection of construction materials. For this reason, the material specifications, in particular, for structural materials used in bridge structures must be known, among others, in terms of both: (1) long-term durability, and (2) long-term dynamic behavior. The first issue is related to a possible deterioration, or even improvement of mechanical properties of structural materials, e.g., modulus of elasticity or tensile and bending strength, affecting the overall stiffness or bearing capacity of structures after many years of service life. Such a change in mechanical properties is caused by broadly understood environmental impacts, material aging processes as well as repetitive service loads. The second problem is connected with a possible change in dynamic characteristics and vibration amplitudes of structures excited by dynamic loads during many years of service life. The above issues are well understood for bridge structures made of conventional materials like steel or concrete. However, in the case of structures made of innovative materials, these problems require a great effort and could be recognized based on a long-term monitoring of existing structures under real environmental conditions.

One of the most innovative material being slowly introduced in bridge structures over the last decades is Fiber Reinforced Polymer (FRP). The primary bridge in the world made entirely of FRP material was Miyun Bridge, constructed in 1982 in China with a span length of 20.7 m (Keller 2003). Then, one of the first pedestrian bridges constructed in 1992 in Europe was Aberfeldy Bridge in Scotland, UK (Skinner 2009). The bridge is 113 m long, with a main span of 63 m. Thenceforward, the number of bridges in the world made entirely of FRP composite is constantly increasing. Before the year 2013, 82 all-FRP

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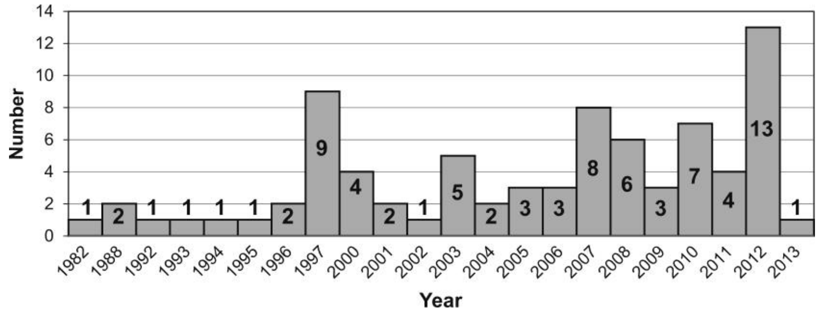

Fig. 1 Number of all-FRP bridges built in the world in particular years (based on data published by Potyrala 2011, Murphy 2013)

bridges were built (Potyrala 2011, Murphy 2013). Fig. 1 depicts the number of all-FRP bridges built in the world in particular years.

Nowadays, three main types of fibers are used in FRP as reinforcement, i.e., glass (GFRP), carbon (CFRP) and aramid (AFRP) fibers. Unfortunately, the costs of FRP products are higher than conventional materials like steel or concrete. Thus, because of cost considerations, the most widely used in bridge engineering is composite reinforced with glass fibers. There are only few examples of all-GFRP composite bridge structures, reported in the literature. Among them the following footbridges can be mentioned: the cable-stayed Scripps Bridge in La Jolla, USA (Cortright 2003), Aberfeldy Bridge in Scotland, UK (Skinner 2009), Pontresina Bridge with a truss structural system in Switzerland, depicted in Fig. 2(a) (Kutz 2002), Lleida Bridge with arch girders in Spain, depicted in Fig. 2(b) (Fiberline Composites A/S website I 2018), the girders footbridge constructed in Tainan, Taiwan (Li *et al*. 2014) or Fiberline Bridge in Kolding, Denmark, considered in this paper. Moreover, in modern hybrid footbridges the GFRP profiles are willingly combined with some conventional materials, usually used as a bridge deck system. Some examples of this type of structures are given in the paper (Stankiewicz and Tatara 2015).

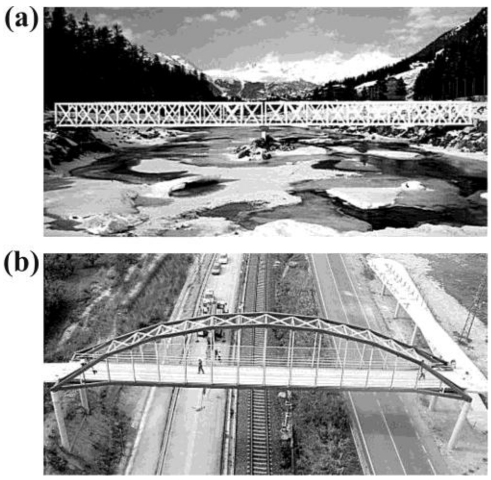

Fig. 2 Some examples of all-GFRP composite foot-bridges: (a) Pontresina Bridge, Switzerland (Kutz 2002) and (b) Lleida, Spain (Fiberline Composites A/S website I)

In comparison to traditional construction materials, the GFRP material has some important advantages such as relatively high tensile strength, low self-weight, resistance to corrosion, electromagnetic transparency, easy and quick assembly and creation of innovative shapes of pultruded profiles as well as whole structures. Moreover, El-Salakawy and Benmokrane (2004) showed that slabs with a CFRP or GFRP reinforcement ratio equivalent to the balanced reinforcement ratio satisfy serviceability and strength requirements of considered design codes. Besides these advantages GFRP has also some limitations like high initial costs, relatively low modulus of elasticity (Young's modulus), low alkaline and fire resistance and low long-term strength due to stress rupture. Inter alia, the relatively low Young's modulus requires the use of relatively large cross-sections of structural elements in order to minimize serviceability problems of all-GFRP bridges. Also the reduced self-weight of GFRP footbridges can leads to problems associated with excessive vibrations (Caetano *et al*. 2009). For this reason, the examinations of mechanical properties and long-term durability of GFRP material are still being performed in order to improve the quality of manufacturing technology and, consequently, to increase the use of composite material in civil engineering structures.

In the future, taking into consideration the relatively high costs of GFRP material in comparison with traditional materials, the widespread application of GFRP composite in bridge structures would be justified by a greater long-term durability leading to reduction of maintenance costs over the lifetime of the bridge. In relation to the term of the long-term durability, GFRP composites are subjected to deterioration over time due to effects of natural environment conditions such as temperature, irradiation, moisture, corrosive chemical environments as well as repetitive service loads (Skinner 2009). The durability can also be significantly affected by the damage due to delamination mechanism. The mentioned deterioration of GFRP material usually causes a negative change in the global structural stiffness due to reduction in Young's modulus, that, consequently produces a change in dynamic characteristics of structures. Unfortunately, the deterioration of GFRP is not visible like in case of steel corrosion or cracking of concrete.

Although in structural applications understanding changes in the material's strength and stiffness is an essential issue, the effects of environmental agents on mechanical properties of GFRP profiles adopted in bridge structures were still not extensively detailed in the literature, especially in relation to the long-term durability. However, a number of studies, mainly in laboratory conditions, were conducted on the effects of exposure environments on the short-term durability of GFRP composites. Karbhari and Pope (1993) or GangaRao *et al*. (1995) investigated the impact of freeze–thaw cycles on composite materials. They found that such exposure can negatively change the thermo-mechanical properties of the resin. On the other hand, Wu *et al*. (2006), based on findings from their own study, concluded that freeze-thaw cycling between -17.8℃ and 4.4℃, up to 1,250 hours and 625 cycles caused very insignificant or no change in the flexural strength, storage modulus and loss factor of FRP conditioned in dry air, distilled water and saltwater. Verghese *et al*. (1998) found that the degradation is primarily associated with the micro-cracking that occurs when the volume of absorbed water changes. Several commercial composites under environmental exposure were studied by Jamond *et al*. (2000) and Malvar *et al*. (2002). The major outcome of these studies proved that seawater immersion and salt-fog exposure caused the greatest degradation in the mechanical properties. Lopez-Anido *et al*. (2004) investigated the performance of the adhesive bonds of the FRP composite under freeze–thaw cycles. They stated that the bond was reduced significantly by exposure to freezing and thawing cycles and the failure mode was changed. Tuwair *et al*. (2016) described how various environmental exposures had changed tensile strength of GFRP bridge panel. Thermal cycling conditioning reduced the ultimate tensile strength and increased the tensile modulus of elasticity for the face sheet coupon specimens by approximately 6% and 0.5%, respectively. The flexural behavior of the GFRP sandwich panels exposed to thermal cycling in an environmental chamber resulted in 24% degradation in the ultimate strength, but with an increase in the stiffness of about 11%. Failure of the conditioned panels under the subsequent static loading occurred in the same manner as the control panels. Lu *et al*. (2016) found that the combined action of both UV and water on polymers as a function of time and temperature could be even more severe than the individual effects. It should be stated that the objective of that work was to investigate the fundamental aspects of the combined UV radiation and water aging processes applicable to any GFRP material. In that paper, it was shown that commercially available unidirectional GFRP composites when exposed to individual and cyclic UV radiation, water condensation and temperature develop significantly different degradation mechanisms with strong synergistic effects. A tabulated summary of experimental results reported by previous studies concerning the effects of thermal cycles on the thermo-mechanical properties of composite materials can be found in the paper (Grammatikos *et al*. 2016).

Sousa *et al*. (2016) compared, among others, variation of the flexural strength as a function of the exposure time of two types of GFRP profiles made of polyester and vinylester resins after exposure to artificial ultraviolet accelerated aging up to 3000 hours and natural aging in the Mediterranean environment during 3.5 years. Flexural properties were measured by means of three-point bending flexural tests. As a result of the tests the vinylester profile suffered a decrease in the flexural strength of about 13% and 22% after both types of exposure, respectively, while the polyester profile presented flexural strength decreasing of about 5% after natural aging and increasing of about 12% after artificial aging process. Finally, it should be concluded that because of the very low rate of degradation processes in real conditions most of the laboratory studies were performed under accelerated exposure simulating accelerated aging process, requiring extrapolation to normal service conditions. In this case many factors, such as temperature, irradiation, moisture, chemicals and presence of an active physical stress, can significantly affect the durability of the polymeric material. Synergism in the global aging often occurs when the simultaneous action of several stresses results in an aging effect that differs from that which would be observed if the individual stresses were applied sequentially. For this reason aging models formulated based on laboratory tests have a limited use for practical applications because real life aging conditions are usually far more complex than those which can be simulated in a laboratory. When assessing whether the conditions of a polymeric material are critical, it is usually better to conduct periodic tests in situ on existing structures, like visual inspections, dynamic tests, chemical and physical measurements, than to rely on existing aging models.

Until now rather limited research was performed evaluating in situ the long-term dynamic behavior and durability of existing bridge structures made entirely of GFRP composite material. Stratford (2012) examined the condition of the cable-stayed Aberfeldy Footbridge after 20 years of service including, among others, its overall structural performance and dynamic characteristics. The footbridge was constructed in 1992. It was found that primary structural components performed well, however, the primary several natural frequencies of the footbridge were reduced slightly over the last twenty years. Finally, it was concluded that changes in the natural frequencies were probably caused by the increase in mass of the bridge deck due to its strengthening in 1997, due to moisture absorption during the test period (winter), or potentially due to reduction in structural stiffness. Consequently, changes in natural frequencies cannot be taken to identify changes in the structural condition.

The flexural behavior of GFRP beams or trusses as well as the dynamic parameters of all-GFRP bridges in full-scale were already investigated regarding some aspects. Some contemporary achievements in this field are presented in the papers (Bačinskas *et al*. 2017, Ascione *et al*. 2015, Votsis *et al*. 2017, Ji *et al*. 2010, Bai and Keller 2008), however, the long-term dynamic behavior or durability were not investigated. Therefore, the question concerning these issues is still arising. The appropriate way to determine long-term changes in the dynamic behavior of bridge structures is to perform in situ the vibration-based structural health monitoring of these structures existing for several dozen years, let say at least twenty years under real environmental conditions. In this case, the vibration-based method either examines differences between measured dynamic structural response for both the original and experimental states of the structure or changes in dynamic characteristics of the structure for both the original and experimental states. This idea of the long-term dynamic behavior investigating the possible changes in modal parameters was applied in this paper to the Fiberline Bridge in Kolding, Denmark. The experimental state should be defined as the condition of the structure after many years of use in the natural environment. The vibration-based method is based on the assumption that the material deterioration usually causes a change in the global structural stiffness due to a change in modulus of elasticity, which consequently produces a change in modal characteristics of the structure. The estimated change in the structural stiffness could be used to predict lifetime of all-GFRP bridge structures as well as to estimate the lifetime safety factor for the assumed design lifetime of these structures. Therefore, the main purpose of the vibration-based monitoring is to identify a change in modal parameters of all-GFRP bridge structure after many years of service life and to establish the aging model of the existing structure in real usage conditions. Such problems are very important, especially in case of all-GFRP bridges for which the specific design codes are not available and insufficient number of studies in existing literature is observed. Thus, the investigation of long-term dynamic behavior of all-GFRP bridge structures after many years of service life still requires further studies and currently is under way.

The primary aim of this paper is to study a change in dynamic characteristics and the structural stiffness of the cable-stayed Fiberline Bridge made entirely of Glass Fiber Reinforced Polymer (GFRP) composite used for 20 years in the fjord area of Kolding, Denmark. Due to this specific location the bridge structure was subjected to natural aging in harsh environmental conditions with temperature amplitudes, solar radiation as well as aggressive impact of sea salt contained in the moisture in the air around the Kolding coastal areas. The flexural properties of the pultruded GFRP profiles acquired from the analyzed footbridge in 1997 and 2012 were determined through three-point bending tests. It was found that the Young's modulus increased by approximately 9%. Moreover, the influence of the temperature on the storage and loss modulus of GFRP material acquired from the Fiberline Bridge was tested by the dynamic mechanical analysis (DMA) for both the intact and actual composites. It was found that both composite materials have similar thermomechanical properties and the GFRP material after 20 years of service life has excellent thermal stability in potential real temperatures occurring in natural environmental conditions. In the first part of this paper the numerical estimation of natural frequencies and mode shapes of the Fiberline Bridge for the original state is presented. In order to perform the numerical analysis of undamped free vibrations of the analyzed bridge a three dimensional (3D) Finite Element (FE) model was developed using the ROBOT commercial program (ROBOT 2018). The initial FE model, reproducing the dynamic properties of the bridge in its original state, was created carefully using the geometrical and material data based on the available detailed design data and results of three-point bending test of a virgin composite GFRP material carried out in 1997. Then, the free-decay vibrations of the Fiberline Bridge, under human jumping, after exposure to natural environmental conditions during 20 years of service life were experimentally investigated in order to obtain the dynamic characteristics for the experimental state. Two jumper positions across the bridge deck were chosen, i.e., at the center and the edge of the deck at mid-span in order to excite both bending and torsion types of mode shapes. Based on in situ free-decay measurements using accelerometers seven natural frequencies, corresponding mode shapes and damping ratios of the bridge were identified. The Peak Picking (PP) and Frequency Domain Decomposition (FDD) approaches were applied to identify the natural frequencies and mode shapes. The corresponding damping ratios (as a fraction of critical damping) were extracted based on filtered modal decays using the least square curve fitting method. The numerical and experimental results were compared. The next objective of this research was to assess a long-term change in the global structural stiffness of the Fiberline Bridge, expressed by a change in the Young's modulus of GFRP material. The analysis was made based on a direct method in which the measured fundamental frequency at the experimental state was compared with the predicted one for the original state. It was found that the global structural stiffness of the considered bridge increased by about 9%. Thus, it was confirmed twice, that the stiffness of the Fiberline Bridge increased after 20 years of service life. Data collected from this study were used to validate and to update the FE model. A good correlation between the natural frequencies and mode shapes predicted by the updated FE model and identified by experimental results was achieved. Thus, it can be concluded that the updated FE model accurately reproduces the dynamic behavior of the considered bridge in the experimental state and can be used as a proper baseline model for the further long-term monitoring to evaluate the overall structural response under service loads. The obtained results provided a relevant data for the procedure of the calculation model verification, calculation model updating, dynamic response prediction and structural health monitoring of all-GFRP composite bridge after many years of service life subjected to natural aging process. The authors hope that this work will become an inspiration for other studies on the durability of existing all-GFRP composite bridges used in natural environmental conditions during decades.

2. Fiberline Bridge, mechanical properties of its material and natural environment conditions

2.1 Mechanical properties

The bending strength and Young's modulus of the pultruded profiles made of GFRP material used in the

Fiberline Bridge were tested in 1997. The experimental set-up and measurement results are described in the Technical Report (2013). Bending tests were performed for two types of rectangular beams varying in dimensions, i.e., the dimensions for type I were 255×10×8 mm, while for type II were 255×55×8 mm. The flexural properties of the beams were determined under three-point bending tests according to the ISO Standard (1998). A similar test was repeated in 2012 for specimens with the same dimensions acquired from the Fiberline Bridge after 15 years of the bridge's life. All test specimens were cut from the cross-bracing element with angle cross-section and dimensions of 75×75×8 mm. In order to perform the tests, the cross-bracing element was removed from the structural system in the mid-span of the main span of the footbridge deck and replaced with a new element. The measured variability of the bending strength and Young's modulus of all specimens type II in 1997 were within the range of 241 to 552 MPa and 20.2 to 23.1 GPa, respectively, with the average strength and Young's modulus values of 449 MPa and 21.4 GPa, respectively. The same flexural properties measured for specimens from 2012 varies within the range of 337 to 485 MPa and 22.7 to 24.4 GPa, with the average values of 407 MPa and 23.4 GPa, respectively. Based on the mentioned results, it should be noticed that the average bending strength value of the GFRP material was reduced by approximately 10%, while the average Young's modulus value increased by approximately 9% after 15 years of the bridge's life. These average values of the Young's modulus were used in studies presented in this paper.

The results mentioned above are in agreement with the experiment results obtained by Correia *et al*. (2005). In this study, based also on the flexural three-point tests it was found that the Young's modulus of GFRP specimens with dimensions of 300×15×10 mm exposed to various aging environment conditions during more than 2,000 hours increased up to 10%. Apicella *et al*. (1982) suggested that there is a competitive effect between matrix plasticization as a result of water sorption and stiffness increase due to the loss of low molecular weight substances. Moreover, the Young's modulus of GFRP material is only affected by temperatures approaching polymer glass transition temperature. Nishizaki *et al*. (2015) reported also an increase in the Young's modulus within the range of 1-29% for three types of pultruded GFRP specimens after a 10-year (from 2003 to 2013) outdoor exposure. The specimens differed with the laminate system and were stored in a moderate climate with the air temperature changed from about -3°C (in winter time) to 32°C (in summer time). According to the authors of the cited paper the increase in the Young's modulus was probably due to the effect of post-curing of the resin. On the other hand, Sousa *et al*. (2014) showed that both the flexural strength and modulus of the GFRP composite may exhibit the reduction of about 11% and 24%, respectively, for the unsaturated polyester profiles, and about 13% and 25%, respectively, for the vinylester profiles. All specimens were exposed to 190 thermal cycles, simulating outdoor applications in Mediterranean mild climates (-5°C to 40°C) in a dry condition.

(a)
(b)
(c)

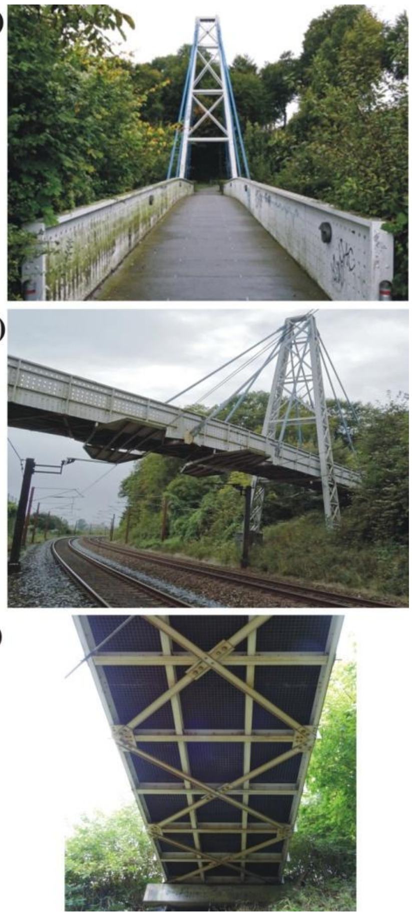

Fig. 3 Fiberline Bridge in Kolding: (a) front; (b) side; and (c) bottom views

2.2 Description of the footbridge

The considered cable-stayed Fiberline Bridge is located near a salt water fjord in Kolding city in Denmark. This Scandinavia's first composite bridge was constructed about twenty years ago and was officially opened on June 18, 1997. The bridge provides pedestrian and cycle traffic along Strandvejen street, crossing one of the main railway line leading to the Kolding city center. In such location the very important advantage is that the GFRP composite does not conduct electricity. It means that no magnetic interference with the electrified railway is observed. The whole bridge construction, including bridge deck, pylon and cables, was made entirely of GFRP composite using 12 different pultruded profiles. Only bolts for assembly and abutments at the foundations were made of steel. The bridge was fully installed in just 18 hours over 3 nights. Such a very short time of installation is a clear advantage of all-GFRP bridges. The general views of the footbridge are shown in Fig. 3.

The 40.3 m long and 3.21 m width footbridge comprises two continuous spans with lengths of 27.9 and 12.4 m, supported by a single A-shaped 17.61 m tall pylon. Four pairs of cables with square cross-sections with dimensions of 100×100×8 mm and a length of 17.85, 17.66, 13.58 and 13.36 m are secured to the top of the pylon. The structural system of the deck comprises two I-shaped pultruded

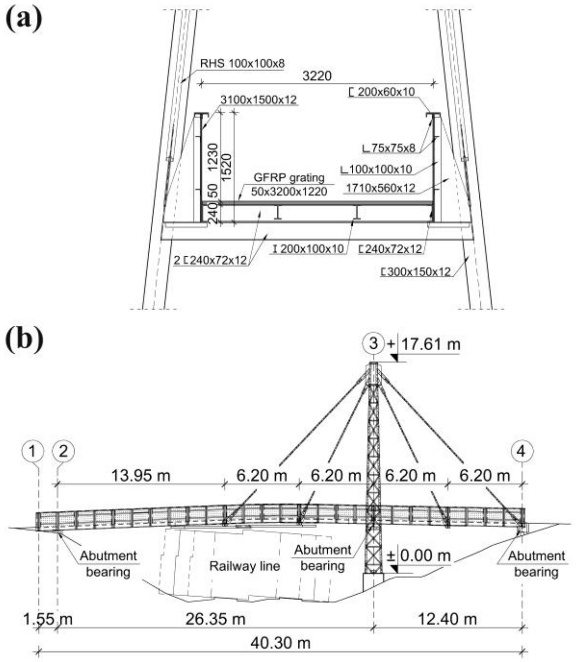

Fig. 4 Fiberline Bridge in Kolding: (a) cross-section; and (b) longitudinal section

profiles with a height of 1.5 m, a thickness of 12 mm and a length of 3.1 m used as main girders as well as parapets. The upper belt of the girders were designed from C-profiles with dimensions of 200×60×10 mm and 75×75×8 mm, being the handrail of the parapets. Additional protection against buckling was made of transverse and longitudinal ribs constructed from angle beams with dimensions of 100×100×10 mm and 75×75×8 mm, respectively with a distance of 1.55 m and 0.73 m. The platform deck was made of composite platform gratings with a thickness of 50 mm. The supporting structure of the deck comprises cross-bars with C-shaped profiles and dimensions of 240×72×12 mm, and side-bars with I-shaped profiles and dimensions of 200×100×10 mm. The outer dimensions of the pylon are varying from 5.22×1.29 m at the bottom to 1.39×0.49 m in the upper part of the pylon. Total weight of the bridge is just 12 tones, while the designed load capacity is 500 kg/m^2. The longitudinal and cross-sections of the Fiberline Bridge with the most important dimensions are shown in Fig. 4. More details about the footbridge can be found in the website (Fiberline Composites A/S website II).

2.3 Natural environment conditions

Because of the location of the Fiberline Bridge, i.e., near salt water Kolding Fjord, the bridge construction was exposed to severe natural environment conditions such as the influence of varying air temperatures, solar radiation, sea salt contained in the moisture of the air, gust-winds, precipitation, snow and icing. During the last 20 years of the bridge's life the air temperature changed from about -20°C (in winter time) to 30°C (in summer time), the relative humidity of the air varied from 30% to 100%, the maximum mean wind velocity was about 30 m/s with the maximum gust-wind of 50 m/s, and maximum amount of precipitation was about 42 mm (https://www.wunderground.com/history). In spite of these disadvantageous environmental conditions, the expected lifetime of the Fiberline Bridge was minimum 100 years. Moreover, it was assumed that the maintenance costs of the bridge over the next 50 years will cover only the cosmetic maintenance costs related mainly to the removal of graffiti and cleaning of algae.

In this case the following important question arises: what is the long-term durability regarding the mechanical properties of the bridge GFRP material. Considering the long expected lifetime of the Fiberline Bridge it seems necessary to make the assessment of the long-term durability of all-GFRP composite bridge after 20 years of service life in natural environmental conditions. Therefore the vibration-based test by measuring long-term changes in modal parameters was performed to identify significant changes in the structural condition. The results are crucial to have a reliable data concerning the long-term dynamic behavior of the bridge.

3. DMA of GFRP material after 20 years of service life

At a stage of the investigation of dynamic characteristics of existing bridge, an important issue may be the influence of the air temperature on a change in the global structural stiffness of the bridge, expressed by a change in the modulus of elasticity. Based on the vibration monitoring test of two-span continuous steel frame Dowling Hall Footbridge, Moser and Moaveni (2011) found the nonlinear relationship between natural frequencies and air temperature. They observed that the identified natural frequencies of the footbridge varied from 4 to 8%, while the measured temperature varied in the range from -14°C to 39°C. On the other hand, such temperature dependence was not observed for the identified mode shapes or damping ratios.

In order to determine the influence of air temperature on the storage and loss modulus of GFRP material acquired from the Fiberline Bridge the dynamic mechanical analysis (DMA) was performed for both (1) the intact GFRP pultruded material, namely virgin material, denoted as Sample No. 1 (the test was done in 1997); and (2) the actual GFRP material acquired after 20 years of natural aging, denoted as Sample No. 2 (the test was done in 2017).

The DMA analysis is one of the most useful tool for studying the viscoelastic thermal behavior of polymers and composites. Within the frame of DMA, samples are subjected to a sinusoidal mechanical oscillation at a fixed frequency, while the temperature increases at a constant rate. Under such conditions the amplitudes of the load, deformation cycles as well as the phase angles between these cycles are measured. Finally, DMA provides quantitative determination of mechanical properties of a sample under oscillating load as a function of temperature, time and frequency. In this study, DMA was carried out on an DMA/SDTA861^e apparatus from Mettler Toledo (Fig. 5). An oscillatory dual cantilever bending deformation with displacement amplitude of 10 μm, force amplitude of 2N at the constant frequency of 5 Hz and at the heating rate of

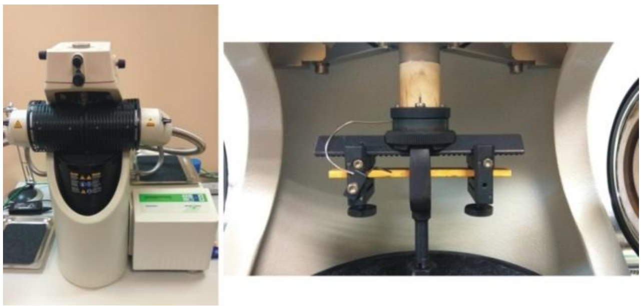

Fig. 5 Mettler Toledo DMA/SDTA861^e apparatus during the test in 2017

2 K/min (the possible temperature range was from -40°C to +200°C) were applied. The values of storage modulus *E*', loss modulus *E*'' and loss factor *tan δ* = *E*''/*E*' were recorded as functions of temperature. Samples No. 1 (1997) and No. 2 (2017) as rectangular beams measuring 90×10× 3.5 mm were inserted into clamps for sample fixation. Sample material was made of alternating layers of unidirectional E-glass fiber roving and strand mats embedded in polymer resin matrix.

Fig. 6 presents changes in *E*' and *tan δ* as functions of temperature determined for Samples No. 1 and No. 2 within the range of the natural environmental temperature from -20°C to +30°C. The thermograms for both samples are similar, i.e., almost linear relationship between *E*' and temperature was observed (see Fig. 6(a)). The storage modulus slightly decreased with increasing temperature. However, Sample No. 2 has higher resistance to deformation than Sample No. 1 within the entire temperature range, which leads to the higher *E*' values. For example at 5°C the *E*' value is 8654 MPa for Sample No. 1 and 9071 MPa for Sample No. 2, while at 25°C the *E*' value is 8622 MPa and 8969 MPa, respectively. Simultaneously, an inverse relation is observed for *tan δ* (see Fig. 6(b)).

Finally, it can be concluded that GFRP composite material used as the structural material in the Fiberline Bridge, has similar thermomechanical properties and excellent thermal stability after 20 years of service life.

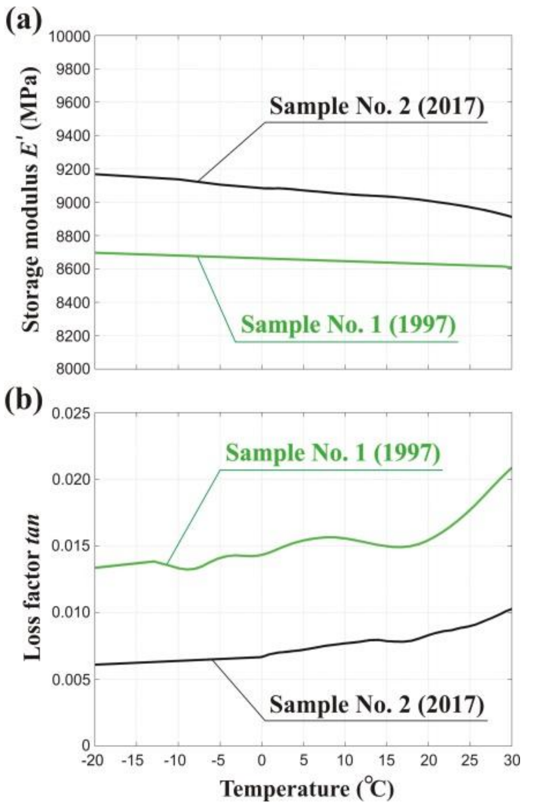

Fig. 6 DMA thermograms of Samples No. 1 (1997) and No. 2 (2017): (a) storage modulus *E*'; and (b) loss actor *tan δ* as a function of temperature within real environmental temperature range

4. FE model and structural identification of the bridge

According to the authors' knowledge, the investigation in situ on the dynamic behavior of the bridge for its original state was not performed before. Therefore, the initial dynamic characteristics were estimated on the basis of theoretical assumptions. In order to perform the undamped free vibrations numerical analysis of the Fiberline Bridge, assuming its linear-elastic behavior, a three dimensional (3D) Finite Element (FE) model was developed using ROBOT software. The initial FE model was created carefully using the real geometrical and material data obtained from the available detailed design data and based on the results of flexural properties test carried out in 1997 for the intact composite GFRP material used for structural elements of the bridge, given in Section 2.1.

Based on the study of the dynamic behavior of the 51.3 m span FRP Wilcott suspension footbridge, Votsis *et al*. (2017) demonstrated that the modulus of elasticity of structural materials, that affect the overall stiffness, have a great affect on the dynamic characteristics of the bridge. Moreover, they found that non-structural members such as parapets can also affect the stiffness as well as the dynamic behavior of the bridge structure.

Hence, in order to predict the real behavior of the Fiberline Bridge the Young's modulus of the intact GFRP composite material was adopted as E = 21.4 GPa based on the test carried out in 1997. The density of the GFRP profiles was 18.0 kN/m^3. The moment of inertia of all profiles was estimated from the geometry of its cross-sections. In order to predict a more realistic structural behavior of the bridge in the 3D FE model all structural elements and accessory parts were taken into account. The concrete pillars (foundations) of the bridge, considerably much stiffer than other structural elements, were not modeled. Additionally, in order to obtain results comparable with the experimental results of free-decay vibrations of the bridge under human jumping excitations, described in Section 5, the mass of one person standing in the middle of the bridge deck near the reference point was assumed.

Two kinds of finite elements were used to model different structural members. The final model of the entire Fiberline Bridge, shown in Fig. 7, was composed using 4429 one-dimensional frame elements and 11010 two-dimensional shell elements, in total with 59884 active

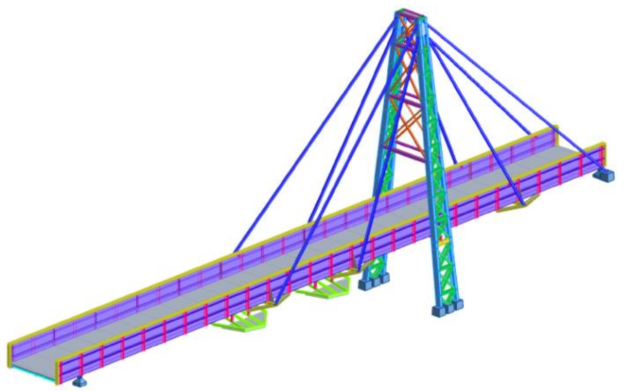

Fig. 7 3D FE model of the Fiberline Bridge in Kolding

degrees of freedom.

The set of natural frequencies and corresponding natural mode shapes of the considered bridge was calculated using ROBOT software. Since bending and torsion types of modes were deemed as the most relevant for assessment of the dynamic behavior of the bridge only numerical results in the vertical direction are given in Fig. 8. In this figure, primary eight major mode shapes with corresponding natural frequencies are presented. The predicted natural frequencies f_k for the original state of the bridge were denoted as f_1 = 4.17 Hz, f_2 = 6.33 Hz, f_3 = 10.52 Hz, f_4 = 11.02 Hz, f_5 = 15.01 Hz, f_6 = 16.65 Hz, f_7 = 18.06 Hz, and f_8 = 22.12 Hz.

Data collected from vibration tests of the bridge, described in Section 5, and test results of flexural properties (especially the Young's modulus) of the composite GFRP material for samples from 2012, given in Section 2.1, were used to validate the reliability and accuracy of the initial 3D FE model of the bridge. In Section 6, based on results of the FE model updating procedure it was proved the exact match of the natural frequencies (see Table 2) and mode shapes (see Fig. 17) predicted numerically and experimentally for the experimental state of the bridge. It confirms the high quality of the FE model. Based on these results, it was assumed that the 3D FE structural model accurately reproduces the dynamic properties of the bridge, i.e., the natural frequencies and corresponding mode shapes predicted from the FE analysis accurately describe the original state of the bridge. However, one should be aware that without experimental data for the original state and FE model calibration it is impossible to obtain very accurate FE model especially for a structure existing for 20 years.

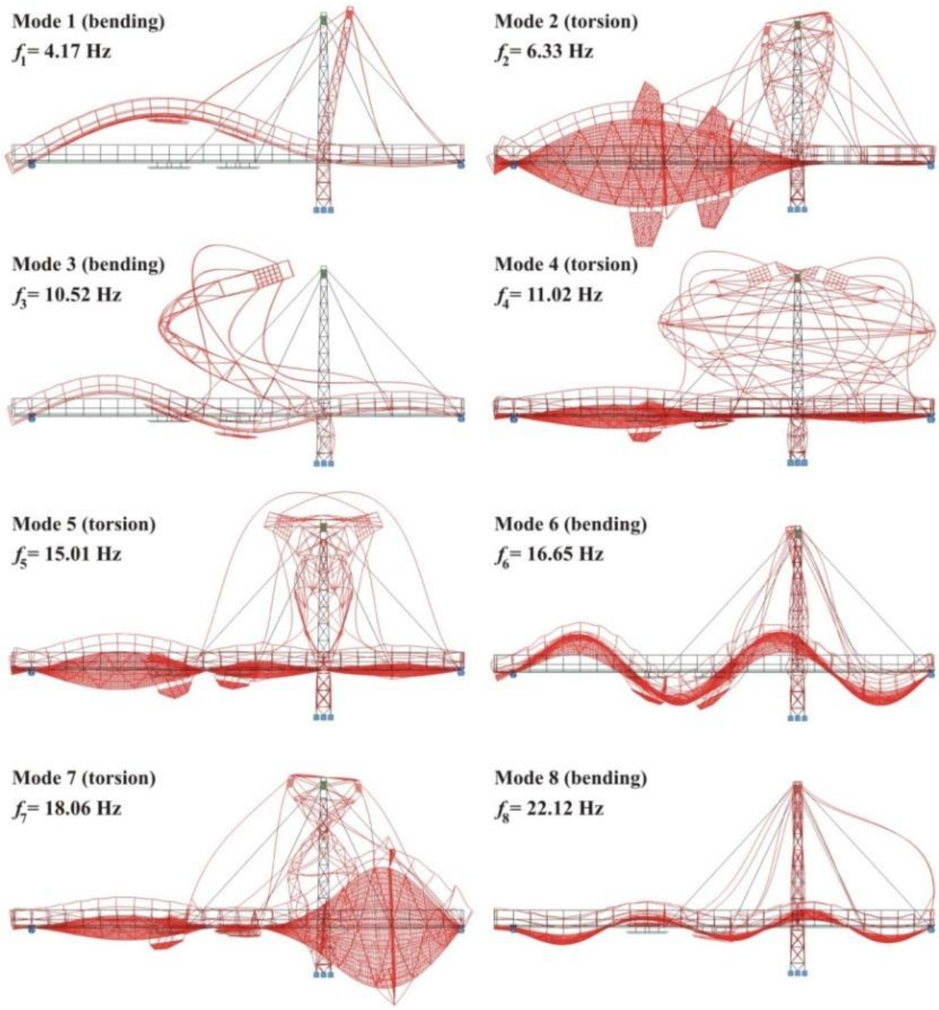

Fig. 8 Structural identification of the Fiberline Bridge for the original state (primary eight major mode shapes and corresponding natural frequencies in the vertical direction) by FE analysis

Among other, this is caused by additional uncertainties such as boundary conditions, cables state and their tension, parapets functioning or ambient conditions during tests.

5. Experimental set-up and modal parameter investigation of the bridge after 20 years of service life

5.1 Experimental set-up and measurement results

The field test was carried out on October 1, 2016. During this test the air temperature changed in the range from about 11 to 15°C and relative humidity of the air from 74% to 94%. Moreover, full cloudiness and negligible wind were observed at the same time. The goal of the test was to investigate modal parameters of the Fiberline Bridge after 20 years of service life, i.e., the corresponding natural frequencies, mode shapes and damping ratios, based on free-decay vibrations under human jumping excitations classically measured using accelerometers. The investigation of the natural modes of the bridge deck required the use of two accelerometers simultaneously. In this field test two uniaxial high sensitivity accelerometers type PCB 3711E112G, based on the Micro-Electro-Mechanical System (MEMS) technology, were used. The measurement range of the sensors was ±2 g, the frequency range 0-400 Hz, the broadband resolution 0.1 mg rms in 0.5 to 100 Hz bandwidth, and the spectral noise 15 μg/√Hz in 1 to 100 Hz bandwidth. Such sensitivity is adequate for studying dynamic behavior of the bridge.

In order to examine the torsion modes the locations of the accelerometers were adjusted mainly within 3.1 m distance along both edges of the bridge deck, i.e., along two girders, namely girders denoted as “A” and “B”. In total 28 measurement points, depicted in Fig. 9, were selected. During the field measurement one reference accelerometer was fixed at the reference point, while the second roving accelerometer was moved step by step from the measurement point No. 1 to point No. 27. The reference point was selected in the place where a major modal component of significant modes was predicted, i.e., near the center of the longer span. Moreover, the measurements were carried out at one point on the pylon in the horizontal direction, along the longitudinal axis of the bridge, about 2.3 m above the floor deck, and at one point on each cable, in the vertical plane across the longitudinal axis of the cable, about 2.5 m above the floor deck. With every rearrangement of the roving accelerometer, i.e., at each measurement point, the series of two human jumps were performed separately in two positions, across the bridge deck, near the reference point. First position, namely the impact point No. I, was chosen at the centerline of the deck, while the second one, namely the impact point No. II, was chosen at the edge of the deck near the girder “A” (see Fig. 9). Such approach was adopted to investigate the influence of the jumper position on the excited mode shapes. Free vibration responses of the bridge were recorded during at least 25 s with the sampling rate of 200 Hz. Since no significant lateral vibrations of the bridge were exhibited only the vertical acceleration component was measured.

Fig. 10(a) depicts vertical accelerations of the bridge girder “A” at the reference point and measurement point No. 25 (abutment point) surveyed due to two human jumps at impact point No. II. The acceleration produced by jumps reached value close to 5 m/s^2. According to the Eurocode Standard (2004) as well as Sétra guidelines (2006) the obtained maximum acceleration indicates unacceptable vibration serviceability in relation to structures with the fundamental natural frequency below 5 Hz. Therefore, it can be assumed that the analyzed footbridge is characterized by lively dynamic behavior.

The measured free-decays were analyzed by the Fast Fourier Transform. The Power Spectral Density (PSD) functions were estimated for the total duration of the time interval of all free vibration responses. The PSDs of measured vertical accelerations of the bridge girder “A” at the reference point and measurement point No. 25 are presented in Fig. 10(b). These PSDs are dominated by seven well-separated peaks representing seven identified natural frequencies of the considered bridge. Based on the comparison of seven specific natural mode shapes obtained using the FDD method from experimental data, presented in Figs. 14-15, with those obtained from the FE numerical analysis, presented in Fig. 8, it was found that the indicated frequencies correspond to seven natural mode shapes No. 1-6 and 8. Hence, the identified natural frequencies f_k^{PP} by the

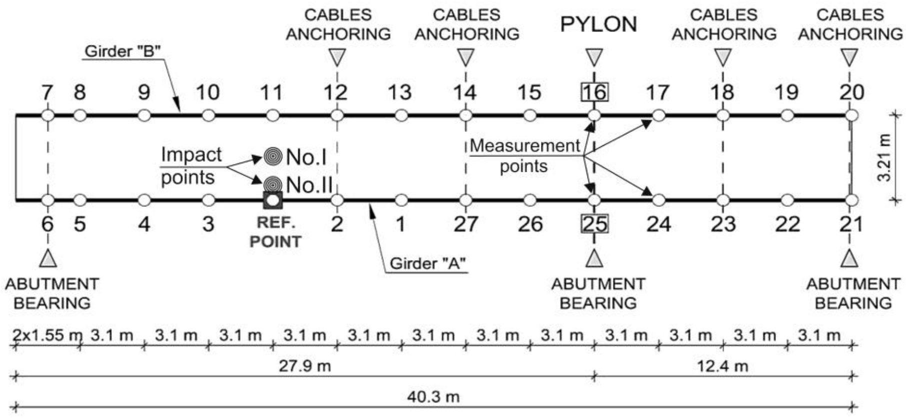

Fig. 9 Locations of the measurement points on the bridge girders

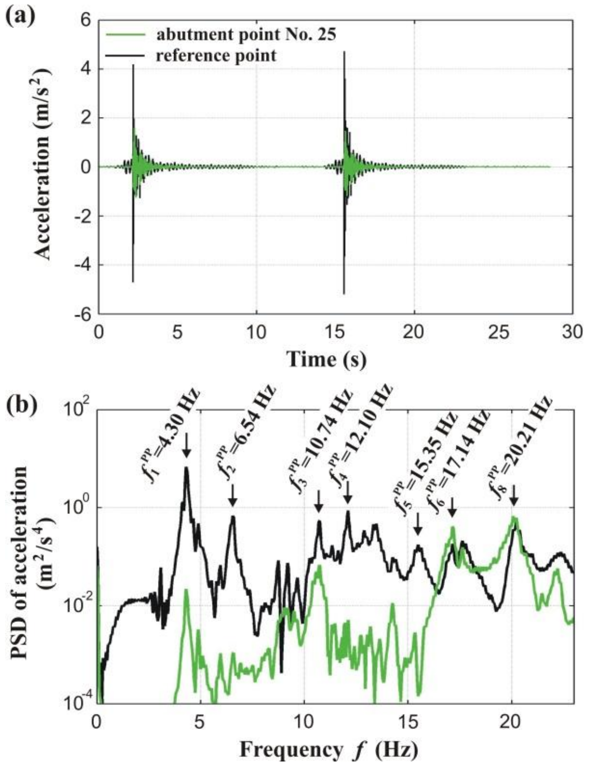

Fig. 10 (a) Recorded free-decay vertical vibrations of the bridge girder "A" at the reference point and measurement point No. 25 (abutment point) excited by two human jumps at impact point No. II; and (b) the PSDs of the recorded accelerations

PP method were denoted as f_1^{PP} = 4.30 Hz, f_2^{PP} = 6.54 Hz, f_3^{PP} = 10.74 Hz, f_4^{PP} = 12.10 Hz, f_5^{PP} = 15.35 Hz, f_6^{PP} = 17.14 Hz, and f_8^{PP} = 20.21 Hz. Fig. 11 shows PSD functions obtained for measured free-decays accelerations of four stay cables anchored at points No. 2, 27, 23 and 21, while Fig. 12 shows the PSD of measured free-decays horizontal accelerations of the pylon. By comparing the natural frequencies of the bridge deck and stay cables it can be stated that the cable-deck interaction is possible.

5.2 Mode shapes and natural frequencies identification using FDD technique

The dynamic characteristics of the considered bridge were evaluated using the FDD technique based on the free-decays measured for all arrangement of two accelerometers at the reference and measurement points denoted as No. 1 to No. 27 (see Fig. 9).

The FDD method was introduced by Brincker *et al*. (2000) as the frequency domain signal processing method used in an output-only modal analysis. The method is appropriate to obtain the natural frequencies and corresponding mode shapes using the Singular Value Decomposition (SVD) of the PSD matrices of structural responses at each discrete frequency calculated with the assumption that the structural damping is light. Such decomposition is performed in order to identify a structure as a single degree of freedom (SDOF) system for each singular value corresponding to the resonant frequency of the structure and corresponding to the singular vector

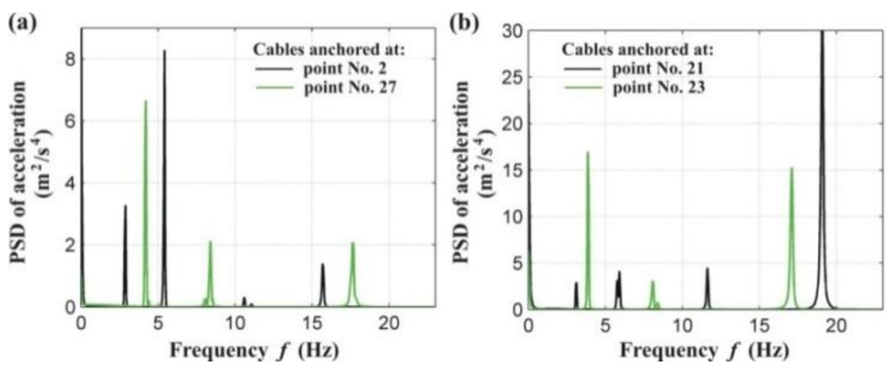

Fig. 11 PSDs of recorded free-decay accelerations of four cables anchored at points: (a) No. 2 and No. 27; (b) No. 21 and No. 23

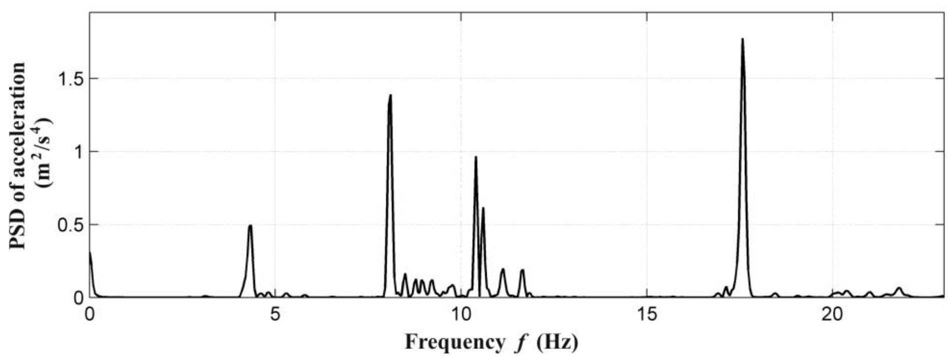

Fig. 12 PSD of recorded free-decay horizontal accelerations of the pylon excited by two human jumps

containing the information about the mode shape of the structure.

A set of PSD matrices of structural response is estimated at each discrete frequency ω_i. These matrices are Hermitian. In case of the considered field measurement of the analyzed footbridge performed simultaneously in two measurement points the PSD matrix $\mathbf{G}_{xx}$ of the response at an i-th frequency ω_i is expressed as follows

$$\mathbf{G}_{xx}(\mathrm{i}\omega_i) = \begin{bmatrix} PSD_{11}(\mathrm{i}\omega_i) & CSD_{12}(\mathrm{i}\omega_i) \\ CSD_{21}(\mathrm{i}\omega_i) & PSD_{22}(\mathrm{i}\omega_i) \end{bmatrix}, \quad (1)$$

where $PSD(\mathrm{i}\omega)$, as elements on the diagonal, are magnitudes of the PSD of the structural response at a selected point of the structure, which for a real stochastic process are real elements, $CSD(\mathrm{i}\omega)$, as elements on the off-diagonal, are the cross PSD of the structural response at two various selected points of the structure and are the complex conjugate, Hermitian's PSDs, which fulfill equation $CSD_{qr}(\mathrm{i}\omega) = CSD^{*}_{rq}(\mathrm{i}\omega)$, whereas $q \neq r$, the symbol „*" denotes a complex conjugate value, $\omega = 2\pi f$, and $\mathrm{i} = \sqrt{-1}$ is the imaginary unit.

Using the SVD technique the $\mathbf{G}_{xx}$ matrix can be decomposed at each discrete frequency ω_i into the diagonal matrix containing a set of discrete singular values $s_j(\omega_i)$, and the unitary matrix containing column singular vectors $\{u_{ij}\}$. Since the FDD technique is well-known in the existing literature the further explanations of the technique principles are omitted. The PSDs and CSDs of the discrete digital signal, representing the free response of the considered bridge, were estimated using the modified averaged periodogram method, called the Welch's technique. According to this technique the time series of the signal are split and then windowing procedure is overlapped before averaging them together. This technique minimizes the bias noise caused by the spectral leakage. In order to further reduce the harmful influence of the leakage error as well as for the spectral averaging, the windowing procedure using the Hanning time-window function with 75% overlap was applied. Based on the FDD approach a multi-step procedure was adopted to write a computer program in Matlab® which was used for numerical calculations.

The plots of two primary singular values $s_1(f)$ and $s_2(f)$ of the decomposed PSD matrices $\mathbf{G}_{xx}$ were estimated for the total length of interval time series of recorded free vibration responses of the bridge equal to 25 s with the frame of 8192 data points. Fig. 13 shows the singular values of the decomposed PSD matrices of recorded free vibration responses of the bridge girder "A", measured at the reference point and measurement point No. 25. Two jumper positions on the bridge deck were considered, i.e., at the impact point No. I (see Fig. 13(a)) and No. II (see Fig. 13(b)). In total, seven natural frequencies of the bridge were identified by the FDD method at which natural mode shapes of the bridge were appeared, i.e., f_{01}-f_{03}, f_{06} and f_{08}, shown in Fig. 13(a), and f_{01}, f_{02}, f_{04} and f_{05}, shown in Fig. 13(b). The seventh frequency f_{07} was not identified. The normalized mode shapes and their theoretical approximation excited by human jumps at the impact point No. I and No. II are shown in Figs. 14-15, respectively. It can be concluded that the application of an impulse due to human jumping in a free vibration test at the impact point No. I was the best to capture five specific vibration modes No. 1, 2, 3, 6 and 8, while the human jumps executed at the impact point No. II were able to excite four vertical vibration modes No. 1, 2, 4 and 5. Summarizing, the identified modes No. 1, 3, 6, 8 can be classified as vertical bending type of modes, while modes No. 2, 4, 5 are vertical torsion modes.

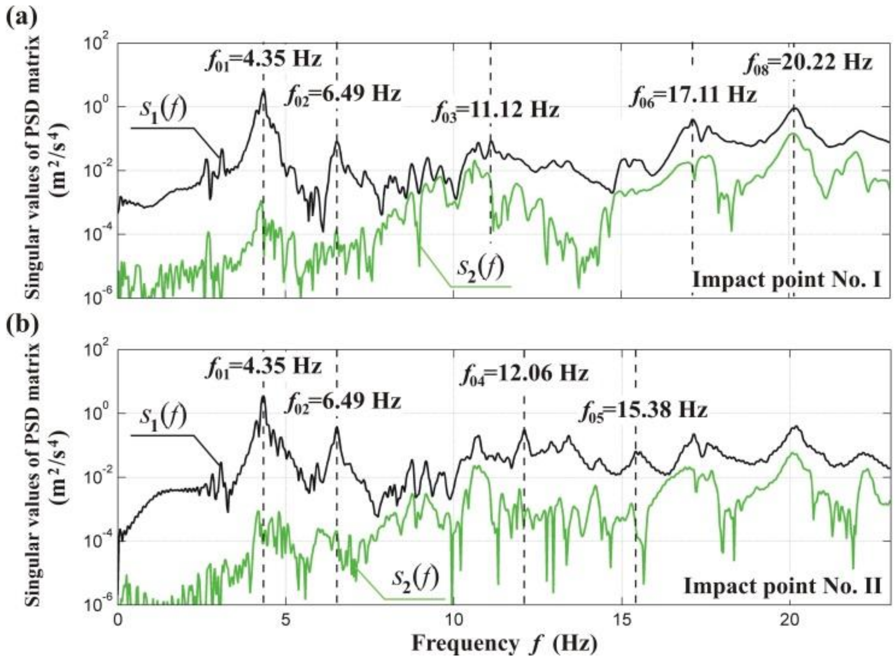

Fig. 13 Singular values of the decomposed PSD matrices of the free vibration responses of the bridge at the reference point and measurement point No. 25 (abutment point) excited by human jumps at impact point (a) No. I; and (b) No. II

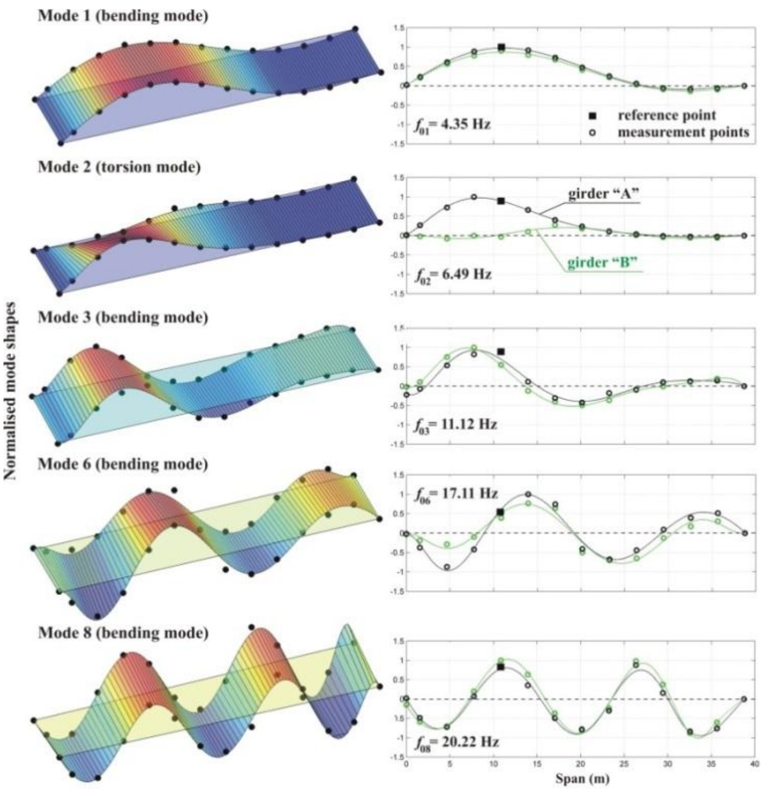

Fig. 14 Normalized mode shapes, their theoretical approximation and corresponding natural frequencies identified from recorded free-decay vertical vibrations of the Fiberline Bridge excited by human jumps at the impact point No. I

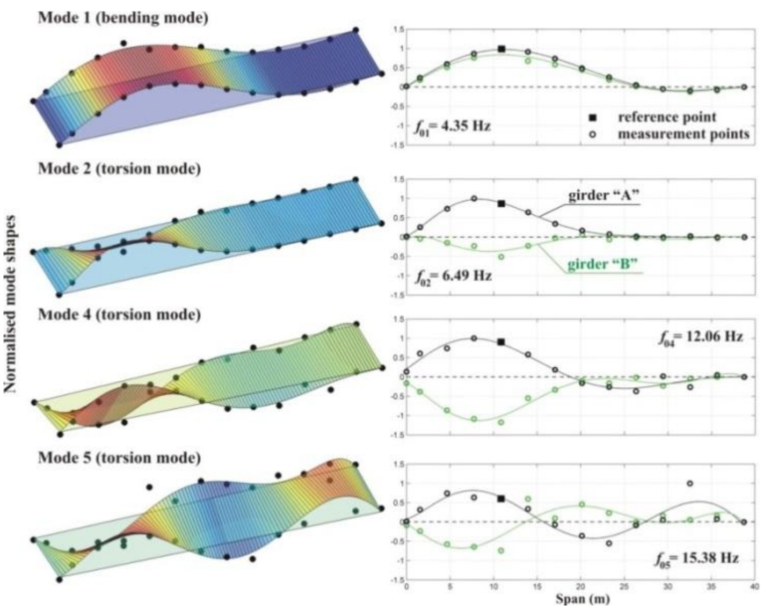

Fig. 15 Normalized mode shapes, their theoretical approximation and corresponding natural frequencies identified from recorded free-decay vertical vibrations of the Fiberline Bridge excited by human jumps at the impact point No. II

5.3 Damping estimation

The modal damping ratios ξ_k (as a fraction of critical damping), corresponding to natural frequencies f_{0k} of the Fiberline Bridge, were analyzed based on the free decays response measured in the vertical direction at the reference point. The analyzed response was decomposed into seven specific modal decays, characterized by the analyzed vibration frequencies f_{01}-f_{06} and f_{08}, by isolating the contributions of seven natural modes of the bridge. For this purpose, the filtering procedure using the eighth-order Type 1 Chebyshev band-pass digital filters with passband ripple 1 dB was applied. The following pass-bands in the filtering procedure were chosen: 3.5-5.0 Hz, 6.0-6.8 Hz, 11.0-11.5 Hz, 11.9-12.2 Hz, 15.3-15.6 Hz, 16.8-17.3 Hz, and 19.5-20.7 Hz.

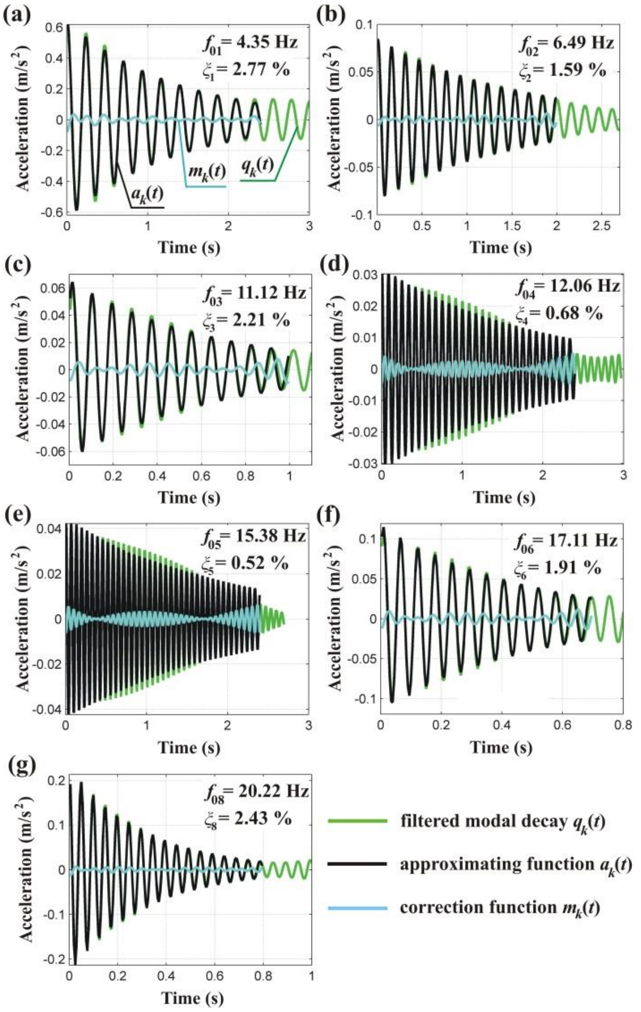

Fig. 16 Theoretical approximation of modal decays provided by filtering procedure using the eighth-order Type 1 Chebyshev band-pass digital filter with passband ripple 1 dB and pass-bands: (a) 3.5-5.0 Hz; (b) 6.0-6.8 Hz; (c) 11.0-11.5 Hz; (d) 11.9-12.2 Hz; (e) 15.3-15.6 Hz; (f) 16.8-17.3 Hz, and (g) 19.5-20.7 Hz

Fig. 16 depicts seven modal decays provided by the filtering procedure. A similar decomposition procedure of measured free decays of several structures was used widely by other researchers, e.g., Magalhães *et al*. (2010). The modal damping ratios ξ_k were estimated directly by approximation of filtered time series $q_k(t)$, containing only the contribution of a single k-th mode, using the least square curve fitting method regarding four unknown parameters, i.e., the frequency f_{0k}, the damping ratio ξ_k, the initial acceleration of the modal decay x_{0k}, and the phase shift φ_k. The approximation of $q_k(t)$ can be performed using the damped free oscillation function $a_k(t)$ according to the following equation

$$q_k(t) = a_k(t) + m_k(t) = \frac{x_{0k}}{\sqrt{1-\xi_k^2}} e^{-2\pi\xi_k f_{0k} t} \cos\left(2\pi\sqrt{1-\xi_k^2} f_{0k} t - \varphi_k\right) + m_k(t), \quad (2)$$

where $m_k(t) = q_k(t) - a_k(t)$ is the correction function of the approximation of filtered time series $q_k(t)$.

The calculation results for the filtered modal decays $q_k(t)$ are presented in Fig. 16. In this figure the approximating functions $a_k(t)$, the correction functions $m_k(t)$ of the approximation and the obtained damping ratio values, corresponding to seven specific natural frequencies and mode shapes of the Fiberline Bridge are given. The obtained damping ratio values are: ξ_1 = 2.77%, ξ_2 = 1.59%, ξ_3 = 2.21%, ξ_4 = 0.68%, ξ_5 = 0.52%, ξ_6 = 1.91%, and ξ_8 = 2.43%.

5.4 Comparison of theoretical and experimental results

Table 1 summarizes the modal parameters of the Fiberline Bridge predicted from the FE analysis for the original state and identified experimentally for the experimental state. The predicted numerically and identified experimentally mode shapes (their theoretical approximation) were verified at all measurement points through the use of the Modal Assurance Criteria (MAC). These values are shown in Fig. 17. Usually, the MAC value higher than

Table 1 Summary of the modal parameters identified from FE analysis for the original state and free-decay vibration test for the experimental state of the bridge

Mode No.	Mode type	Natural frequency (Hz)		$\frac{\lvert f_{0k}-f_k\rvert}{f_{0k}}$	Damping ratio
		f_k (original state)	f_{0k} (experimental state)	(%)	(%)
1	Bending	4.17	4.35	4.14	2.77
2	Torsion	6.33	6.49	2.46	1.59
3	Bending	10.52	11.12	5.40	2.21
4	Torsion	11.02	12.06	8.62	0.68
5	Torsion	15.01	15.38	2.41	0.52
6	Bending	16.65	17.11	2.67	1.91
7	Torsion	18.06	-	-	-
8	Bending	22.12	20.22	9.40	2.43

0.8 is considered as a good match, whereas the MAC value less than 0.4 is considered as a poor match (Gentile and Bernardini 2008). It means that only torsion mode No. 2 and bending mode No. 6 could not be considered as very well correlated, however, in the case of these modes the MAC values are also relatively high (about 0.7). The other five mode shapes are well correlated which proved the good quality of the measurement results as well as the FE model.

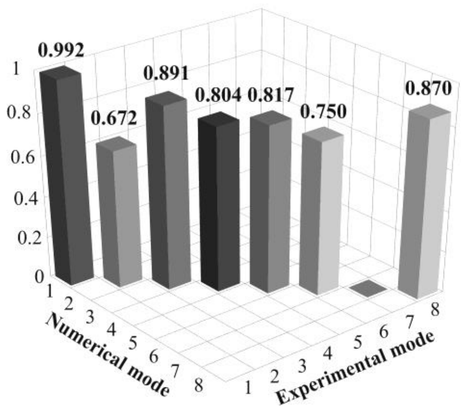

Fig. 17 Comparison of predicted numerically and experimentally mode shapes using the MAC criterion

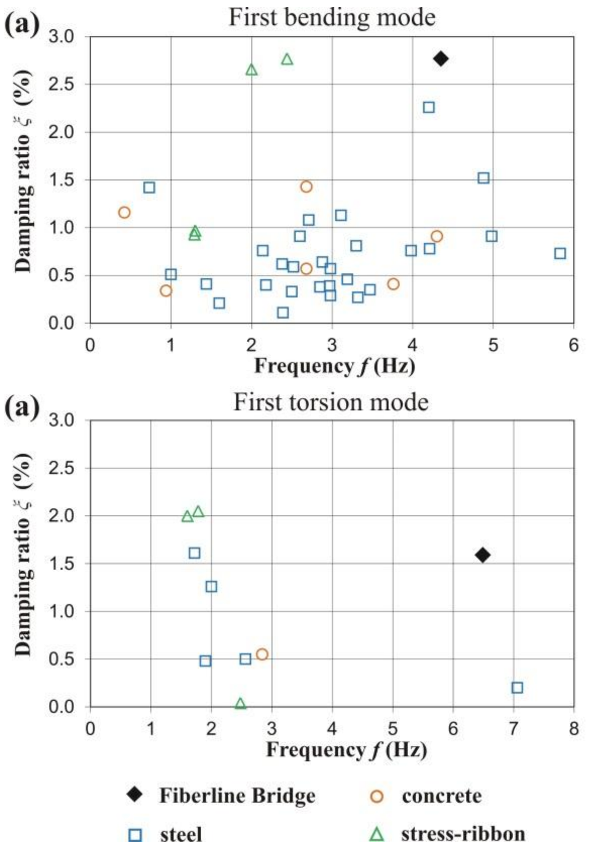

Fig. 18 Comparison of measured damping ratios of the Fiberline Bridge with data published in the literature for selected footbridges made of various materials regarding their natural frequencies in relation to the first (a) vertical bending; and (b) torsion modes

The comparison of the measured damping ratio values of the Fiberline Bridge with data published in the literature (Butz *et al*. 2007, Brownjohn *et al*. 2016, Maes *et al*. 2016, Pańtak *et al*. 2018) for several selected footbridges made of various conventional materials in relation to the first vertical bending and torsion modes is presented in Fig. 18.

6. Assessment of change in structural stiffness

The assessment of a change in the structural stiffness of the Fiberline Bridge after 20 years of service life was performed based on the idea of the vibration-based method, i.e., from possible changes of natural frequency for both the original and experimental states of the bridge. It should be noticed that the visual inspection of the bridge, performed on October 1, 2016, showed no visible damages both in structural elements and their connections. Since no significant changes in mass and geometrical features of the bridge elements were found, it was assumed that changes in modal parameters are the result of a change in the global structural stiffness of the bridge due to a change in the modulus of elasticity of GFRP material.

In order to solve this problem some inverse method, developed for the application of finite element model updating, should be applied. The review of direct and iterative inverse techniques is presented in the book (Friswell and Mottershead 1995).

The iterative methods allow for the iterative modification of selected physical parameters of the structural material, assigned to particular finite elements of the numerical model. The procedure of such methods usually requires minimizing non-linear functions of several variables using the sensitivity analysis, considering the difference between the analytical and measured data with respect to the model parameters. The problem arises when a number of available changes between measured and predicted natural frequencies is much smaller than the unknown changes in selected physical parameters of each structural finite elements. In this case, the problem in mathematics becomes underdetermined and has infinite solutions. It can be solved uniquely after the introduction of an optimality criterion (Hassiotis and Jeong 1995). However, some sensitive parameters selected as a result of calculations may assume unrealistic values.

The direct methods are computationally more simple and often effective. Such methods allow for very accurate updating of the global stiffness or mass matrices of the FE model in one calculation step, i.e., without iterations. A procedure of the direct method do not take into account the physical meaning of updating matrix elements. However, it is generally suitable for application to a structure containing a large number of degrees of freedom.

For this reason to assess a change in the structural stiffness of the Fiberline Bridge a procedure of a simple direct method was applied. Taking into account the limited accuracy of computations and measurements of the higher bridge modes as well as the decreasing importance of higher modes, the approach proposed below considers the difference between modal data collected for the first mode of the bridge. Thereby, it was assumed that changes in the first natural frequency, as a global property of the bridge, is least susceptible to measurement and calculation errors, and may be used as a measure of a change in the structural stiffness. Thus, the basis of the direct method is the difference between the fundamental natural frequency and mode shape predicted numerically, for the original state, and experimentally, for the experimental state of the bridge. The change in the global stiffness matrix was derived utilizing the orthogonality conditions for the first eigenvector.

Let's consider two eigenvalue problems of the free vibration of an undamped structure for the first mode in the original (Eq. (3)) and experimental (Eq. (4)) states described by the equations

$$\omega_1^2 \mathbf{M}\phi_1 = \mathbf{K}\phi_1, \tag{3}$$

$$\omega_{01}^2 \mathbf{M}\phi_{01} = (\mathbf{K} + \Delta\mathbf{K})\phi_{01}, \tag{4}$$

where ω_1, ω_{01} and ϕ_1, ϕ_{01} are the first natural circular frequencies and mode shapes in the original and experimental states, respectively, $\mathbf{M}$ and $\mathbf{K}$ are the mass and stiffness matrices in the original state, respectively, and $\Delta\mathbf{K}$ is a change of the global stiffness matrix of the structure.

Thus, the change in the structural stiffness of the structure is defined as a change of its stiffness matrix by an amount of $\Delta\mathbf{K}$. It was assumed that the stiffness change is not accompanied by a change in mass of the bridge. Further, the change in the stiffness produces changes in the natural frequency from the initial value of ω_1 to the final (experimental) value denoted here as ω_{01}. After subtracting Eq. (4) from Eq. (3) it can be written as

$$\mathbf{M}(\omega_{01}^2 \phi_{01} - \omega_1^2 \phi_1) = \mathbf{K}(\phi_{01} - \phi_1) + \Delta\mathbf{K}\phi_{01}. \tag{5}$$

Based on the MAC value computed for the first mode shape, a very good agreement for the original ϕ_1 and experimental ϕ_{01} mode shapes of the Fiberline Bridge was confirmed, i.e., the obtained MAC value was 0.992. Therefore, assuming that $\phi_1 \cong \phi_{01}$, Eq. (5) can be simplified to the form

$$\mathbf{M}\phi_1(\omega_{01}^2 - \omega_1^2) = \Delta\mathbf{K}\phi_1. \tag{6}$$

After multiplying Eq. (6) by ϕ_1^{T} gives

$$\phi_1^{\mathrm{T}} \mathbf{M}\phi_1(\omega_{01}^2 - \omega_1^2) = \phi_1^{\mathrm{T}} \Delta\mathbf{K}\phi_1. \tag{7}$$

The change of the global stiffness matrix of the structure $\Delta\mathbf{K}$ can be expressed by the non-dimensional coefficient α_K in the form

$$\Delta\mathbf{K} = \alpha_K \mathbf{K}. \tag{8}$$

Noting that the first eigenvector ϕ_1 can be normalized with respect to the mass matrix, $\phi_1^T \mathbf{M}\phi_1 = 1$, as well as the stiffness matrix, $\phi_1^T \mathbf{K}\boldsymbol{\phi_1} = \omega_1^2$, and substituting Eq. (8) into Eq. (7), the non-dimensional coefficient α_K can be given by

$$\alpha_K = \frac{\omega_{01}^2 - \omega_1^2}{\omega_1^2}. \tag{9}$$

Finally, the global stiffness matrix for the experimental state of the structure can be determined according to the equation

$$\mathbf{K}_0 = \mathbf{K} + \Delta\mathbf{K} = \mathbf{K}(1 + \alpha_K). \tag{10}$$

The Eq. (10) relates the change in the stiffness of a structure to the change in its natural frequency. Solving the eigenproblem for the original state of the Fiberline Bridge based on the FE analysis the fundamental natural frequency was obtained as ω_1 = 26.20 rad/s, while the measured value, representing the experimental state of the bridge was equal to ω_{01} = 27.33 rad/s. The calculated non-dimensional coefficient is α_K = 0.088. It means that the global stiffness of the considered bridge, expressed by the Young's modulus of GFRP material after 20 years of service life increased by about 8.8 %, i.e., from E = 21.4 GPa to E = 23.3 GPa. Therefore, the change of the global bridge stiffness matrix can be expressed by $\mathbf{K}_0$ = 1.088·$\mathbf{K}$. This increase in the Young's modulus of the composite GFRP material obtained experimentally using the vibration-based method after 20 years of service agrees excellently with the material test results, given in Section 2.1, performed in 1997 and 2012 on the GFRP pultruded profiles.

Data collected from vibration test were used to validate the FE model reliability and its accuracy in prediction of the modal parameters of the bridge. For this purpose, the eigenvalue problem of free vibration of the undamped structure, considering the 3D FE model with the updated stiffness matrix $\mathbf{K}_0$, was calculated again. In order to validate the updated calculation model, the comparison of natural frequencies obtained from measurements f_{0k} and those computed based on the updated FE model $f_{k,\mathrm{mod}}$ is presented in Table 2. It was demonstrated that the natural frequencies predicted by the updated model agree well with the experimental results for the experimental state of the bridge. Thus, it could be stated that the updated FE model accurately reproduces the dynamic behavior for the considered bridge after 20 years of service life and can be used in the future as a proper baseline model for a long-term monitoring to evaluate the overall structural response under the service loads.

Table 2 Comparison of the natural frequencies computed for the updated FE model $f_{k,\mathrm{mod}}$ and obtained from the free-decay vibration test for the experimental state of the bridge f_{0k}

Mode No.	Mode type	Natural frequency (Hz)		$\frac{\lvert f_{0k} - f_{k,\mathrm{mod}} \rvert}{f_{0k}}$ (%)
		$f_{k,\mathrm{mod}}$	f_{0k}	
1	Bending	4.35	4.35	0.0
2	Torsion	6.58	6.49	1.39
3	Bending	10.98	11.12	1.26
4	Torsion	11.50	12.06	4.64
5	Torsion	15.66	15.38	1.82
6	Bending	17.30	17.11	1.11
7	Torsion	18.85	-	-
8	Bending	23.07	20.22	14.09

7. Conclusions

The presented study provided relevant information concerning the modal parameter identification of the cable-stayed Fiberline Bridge made entirely of GFRP composite material, such as natural frequencies, mode shapes and structural damping after 20 years of service life. Moreover, a comparative study of dynamic characteristics of the bridge for its original and experimental states was included in the presented investigations. Unfortunately, the natural frequencies of the considered bridge were not determined experimentally directly after its construction. So, it was adopted that the natural frequencies describing the original state were reproduced numerically using the 3D FE model. The initial FE model was created using the real geometrical and material data obtained from both the design data and three-point bending test results. The flexural test was performed in 1997 for the intact composite GFRP material used in the structural system of the Fiberline Bridge. The flexural properties of the pultruded GFRP profiles acquired from the analyzed footbridge were tested again in 2012. It was found that the bending strength of the GFRP material was reduced by approximately 10%, while the average Young's modulus value increased by approximately 9% after 15 years of the bridge's life. Then, the actual dynamic parameters were identified experimentally in 2016 using the vibration-based method, i.e., based on in situ free-decay measurements using accelerometers. To identify modal parameters the PP and FDD techniques were applied. A good agreement was found between the modal frequencies obtained from two applied techniques. By the application of an impulse due to human jumping in a single free vibration test the specific seven vibration modes in the vertical direction were successfully identified within the frequency range of 0-21 Hz. It was found that the first natural frequency obtained experimentally using FDD for the experimental state of the bridge and predicted through the application of the FE method for its original state differ about 4.1% from each other.

In total, seven natural frequencies, corresponding mode shapes and modal damping ratios were determined experimentally. The modes No. 1, 3, 6, and 8 are vertical bending type of modes. The modes No. 2, 4, and 5 are torsion type of modes of the bridge deck. The best jumper position to capture the bending modes was the position at the centerline of the deck, while to capture the torsion modes the best position was at the edge of the deck. The computed and experimentally identified mode shapes exhibit a good match at the measurement points. The MAC criteria has at least the value of about 0.8 for the modes No. 1, 3, 4, 5 and 8, while for the modes No. 2 and 6 the MAC value was about 0.7. The accuracy of identification of some mode shapes, especially the torsion modes, was limited due to a limited sensors accuracy, computational errors in the FDD technique as well as a way of vibration excitation. Since the FE model is an idealization of the state of the existing structure, such problem was not observed in numerical results of free vibration analysis.

The comparison of the estimated damping ratios of the Fiberline Bridge with data published in the literature for selected footbridges with comparable length made of various conventional materials showed that the modal damping of the cable-stayed footbridge made entirely of GFRP composite is relatively high for the first bending and torsion modes. The obtained value of the damping ratio for the first bending mode was 2.77%. The corresponding damping ratios for concrete footbridges were within the range of 0.34 to 1.43%, for steel ones within the range of 0.11 to 2.26%, and for stress-ribbon ones within the range of 0.93 to 2.77%.

The influence of the temperature on the storage modulus of GFRP material, representing the elastic material behavior, was tested by DMA. The test was performed for both the intact material used in the Fiberline Bridge construction in 1997 and actual material acquired from the Fiberline Bridge after 20 years of natural aging. DMA has shown that both materials have similar thermomechanical properties what suggests an excellent thermal stability of composite material after 20 years of service life. It was found that the storage modulus of this material varies with almost linear relation from 9168 MPa to 8908 MPa within the range of the natural environmental temperature occurring in the bridge location conditions, i.e., from -20℃ to 30℃. For example, the change of the temperature for 10℃ causes the change of the storage modulus for about 0.5%.

A possible change in the global structural stiffness of the considered bridge after 20 years of service life was determined based on a change in the fundamental natural frequency computed for the original state and experimentally tested for the experimental state of the bridge. However, without experimental data for the original state it is impossible to obtain very accurate FE model reproducing the original state of the bridge. It was found that the primary natural frequency increased by about 4.1%, thus, the global stiffness, expressed by the Young's modulus of GFRP material after 20 years of service life increased by about 8.8%. Similar conclusion was formulated based on the mentioned above three-point bending tests performed for the pultruded GFRP profiles in 1997 and 2012. Thus, it was confirmed twice, that the structural stiffness of the Fiberline Bridge increased by about 9% after 20 years of service life. Thereby, it can be concluded that the structural stiffness of the bridge has apparently increased and the GFRP composite used as a structural material of the considered bridge avoided deterioration due to various deterioration processes.

In the case of the considered bridge, taking into account the expected design lifetime of 100 years, it is very advisable to conduct structural health monitoring in the future, i.e., in the next decades, in order to monitor possible structural deterioration or damage due to many factors. The long-term changes in Young's modulus can be used to identify significant changes in the bridge condition. In case of severe problems the respective owner managements can make decisions for additional inspections of the bridge structure in order to secure its safety. Data collected in the presented study could be used as a reference data for the further long-term monitoring of the Fiberline Bridge, let's say every ten or twenty years, which is the best way to determine its durability. Thus, the structural health monitoring procedure for this bridge will be created for the following years. The authors hope that this work will become an inspiration for other studies on the durability of existing all-GFRP composite bridges used in natural environmental conditions during decades.

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