A practical approach for modeling tendon and wire failures for model-based damage detection of prestressed concrete bridges

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ABSTRACT: Predictive maintenance management which eliminates structural deficits before consequential damage occurs offers great potential to meet the challenges of an aging infrastructure. The OSIMAB project pursues this strategy with a holistic approach to monitoring and assessment of road bridges.

This paper focuses on the results of model-based damage detection and calibration of a finite element model to sensor data. Road bridges must withstand a multitude of exposures from traffic and environment. Tendon failure is critical for the load-bearing capacity and is particularly difficult to detect. A method for modeling tendon and wire fractures is presented, which efficiently considers important effects such as the re-anchoring of the tendons in the grout. A numerical simulation of a damaged bridge girder is used to demonstrate possibilities of detecting and localizing damage with a finite element update.

1 INTRODUCTION

As a result of rising traffic volumes combined with an advanced age of the infrastructure, many road bridges in Germany are in an unsatisfactory condition. The resulting need for action has accelerated research in predictive maintenance management. In addition to periodical inspections of the current bridge condition, durable monitoring systems can be used to detect structural shortcomings at an early stage. For the operator of the bridge an advantage in a higher planning reliability and lower maintenance costs by reducing subsequent damages is conceivable. Today, a variety of sensors for Structural Health Monitoring (SHM) of road bridges are available at decreasing costs. The storage of large amounts of data, on local systems or in cloud solutions, has become unproblematic. While the progress in data acquisition is huge, research needs to address analysis methods to convert data to insights about a structure's performance and reliability.

The joint project OSIMAB (Online Safety Management System for Bridges) is taking up this challenge with a holistic concept to monitor and assess the condition of road bridges. For this purpose, the know-how of the partners Bundesanstalt für Straßenwesen, Hasso-Plattner-Institut, Hottinger Baldwin Messtechnik, ITC Engineering and Technische Universität Berlin is applied. The starting point is an analysis of the building stock at the German trunk road network using existing data sets on climate, traffic and structures. The data is supplemented by additional information from long-term monitoring of relevant individual structures. Data mining algorithms and calibrated finite element models provide tools for early detection of structural changes and deficits. The concept is completed by a risk management which takes a safety concept adapted to the current structural condition into account.

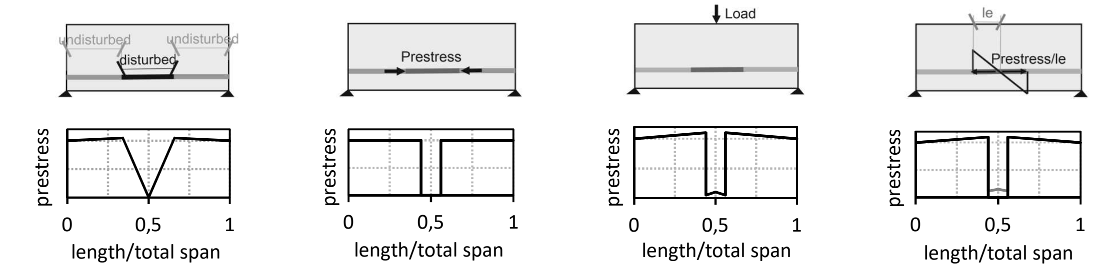
Due to the complexity of bridge structures, several damage mechanisms are possible. A particularly critical damage type for prestressed concrete bridges is the failure of bonded post-tensioned tendons because of high relevance for the bearing capacity, their possible brittle failure form and limited recognizability, König et al. (1990). Reasons for steel fracture lie in chloride-induced corrosion due to environmental influences and de-icing salts, in fatigue due to recurrent traffic loads or in stress corrosion cracking due to incorrect material choice in combination with humidity. Research on damage detection concentrates on the use of non-destructive test methods such as Acoustic Emission, Magnetic Flux Leakage, Radiography, Ultrasonic Guided Waves, see e.g. Azizinamini (2012) for an overview. Despite the increased reliability of the test methods in recent years, their disadvantage of high cost and effort remains.

To date, only limited research has been conducted to simulate the failure of post-tensioned elements with the Finite Element Method (FEM). In Siegert et al. (2015) a conservative calculation of the damage is performed by assuming a loss of prestressing force over the entire tendon length. However, experiments on prestressed members under cable failure, e.g. Coronelli et al. (2009), have shown that a broken tendon has a residual loading capacity due to its re-anchoring in the surrounding grout. The required distance from the point of fracture to the point at which the tendon regains its original prestressing force is known as the transfer length. According to Eichinger (2003), the distance is 0.7 m for multi-wire strands in good bond to normal-strength grout. Due to the high calculation effort, precise modeling of the damage, as carried out in Abdelatif et al. (2012) and Coronelli et al. (2009), is limited to the analysis of structural members with confined dimensions and confined number of tendons. A numerical study of a larger girder is carried out in Cavell et al. (2001) and one on a bridge in Vill (2005).

A disadvantage of the presented investigations is that only the failure of whole tendons and not their components, the individual wires, is considered. Further research is needed to simulate wire and tendon fractures with a precise but practical approach. For this purpose, a modeling method is being developed that takes the re-anchoring of the prestressing elements into account, but can still be integrated into a complete numerical model of a bridge. It can be used as a mathematical model for damage detection with a finite element update.

2 SIMULATION OF WIRE AND TENDON FAILURES

First, the procedure for damage modeling is presented at a single span, prestressed beam with a single flat tendon (Figure 1-a). For better visualization, a tendon failure in mid-span with an enhanced transfer length is assumed. Due to its wide use in bridge design and its ability to perform physically nonlinear calculations, the program SOFiSTiK, version 2018, is used. The complex behavior of the concrete, e.g. his nonlinear behavior and the tension stiffening effect, is considered to ensure a realistic calculation of subsequent concrete damages. The prestressing force is automatically applied to the concrete member as an external force, taking into account prestressing losses and load transfers. Activating the construction phase “dismantling of tendons” that enables the simulation of a tendon failure leads to the release of prestressing force on defined concrete elements. The resulting sudden drop in prestress leads to a concentrated transfer of high forces, which causes the formation of stress singularities in the concrete. This method shall be referred to as the simplified method due to its low modeling effort and the inaccurate prediction of stresses. An advanced method is developed that avoids stress singularities by taking into account the bond anchorage of the broken cable, following the stress curve shown in Figure 1-a.



(a) Division of tendon and desired prestress curve

(b) Load case: Post-tensioning

(c) Load case: Loading

(d) Load case: Tendon failure

Figure 1. Static system with exterior forces and prestress curve; l_e= transfer length.

The starting point is the division of the tendon over its length into an area undisturbed and disturbed by the damage (Figure 1-a). The prestressing force is applied automatically for the undisturbed area and manually for the disturbed area (Figure 1-b), in order to allow a change of the prestressing curve in each load case. Neglecting the friction losses in draped tendons would be acceptable for this area due to its small extent compared to the bridge length. To consider the load distribution, a stress-free tendon with a stress-strain curve starting at prestressing level is inserted at the disturbed area (Figure 1-c). In order to simulate the tendon fracture, the prestressing force is applied at the end of the disturbed area with opposite direction and distributed over the transfer length (Figure 1-d). Due to the redistribution of stresses residual forces remain in the tendon at its breaking point, which are subsequently eliminated. The modeling of a wire break can be performed in the same way by creating a separate tendon for the remaining and omitted part of the cross section.

Four-point bending tests of a beam under successive wire failure, carried out by König et al. (1990), are used to validate the presented simulation method. The study examines a single span T-girder (Figure 2 Left) whose 14 prestressed wires are severed step-wise in mid-span. The wire break is first simulated using the extended method assuming a transfer length of 0.7 m and 1.4 m. Subsequently, the results are compared with the simplified procedure with a transfer length of 0.7 m. Figure 2 (Middle) shows the measured and calculated deflection of the girder in mid-span. In contrast to the experiment, the calculation shows a significant reduction in stiffness only at the appearance of concrete cracks in load case 9 and not at the first wire break in load case 3. A later investigation of the girder by König et al. (1990) suggests that the reason for the deviation lies in the presence of a poor bond between grout and tendon. Figure 2 (Right) shows the concrete stresses over the beam height resulting from a fivefold wire failure in mid-span. Except for the tensile maximum at tendon height, the calculation can sufficiently reproduce the results.

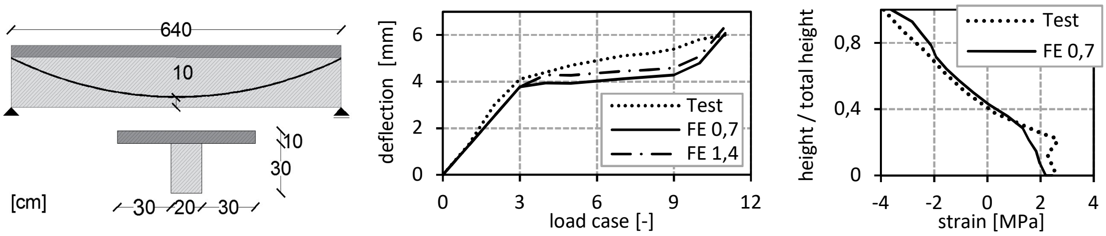


Figure 2. Left: Static system; Middle: Deflection line; Right: Concrete stresses; FE-analysis with transfer length of 0.7 and 1.4 m.

According to König et al. (1990), the sudden stress increase results from the transmission of internal stresses due to the re-anchoring of the tendons, which is only indirectly considered in the numerical calculation. However, the deviation is close to the tendon. Hence, the stress on the bottom side is barely affected.

Figure 3 shows the concrete stresses resulting from a break of five wires, calculated with the simplified and extended method. The differences in stresses compared to the intact state are depicted on the bottom side of the beam (Left) and at tendon height (Right). The application of the simplified procedure leads to an inaccurate prediction of stresses near the tendon, which occurs, as a convergence analysis shows, due to numerically induced stress singularities. The influence of the selected modeling method on the calculated deflection and inclination is negligible. Therefore, the simplified method is not suitable for damage detection, but for rough estimation of the residual load-bearing capacity.

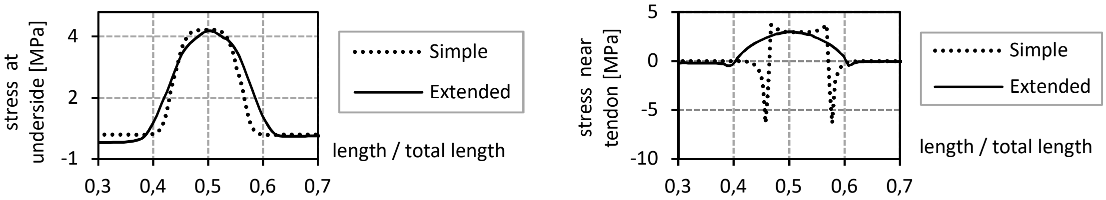


Figure 3. Concrete stresses at failure of 5 wires with transfer length of 0.7 m; Left: At bottom side; Right: At tendon height.

3 FINITE ELEMENT UPDATE FOR DAMAGE DETECTION

Even the structural response calculated with detailed finite element models of bridges often deviates from the response measured with a sensor system on a real structure. Measured data can be used to calibrate a finite element model in order to reproduce the structural response. This process is commonly referred to as finite element update. Calibrating a finite element model yields the obvious benefit of a close to realistic model for structural analysis. Additionally, this approach can also be used for damage detection.

In civil engineering, finite element update has been a viable field of research for over thirty years with early contribution dating back to the 1980s. A full literature review is beyond the scope of this article. The interested reader is referred to notable publication and reviews such as Mottershead et al. (1993), Teughels et al. (2005) or Alkayem et al. (2017). At first, it is important to understand possible reason for errors between measured and calculated responses. Loosely based on Mottershead et al. (1993) these errors shall be classified into three categories: (i) model errors, e.g. insufficient simplifications or calculation theory (linear/nonlinear), (ii) parameter errors, e.g. deviating values for material, stiffness or mass properties and (iii) measurement errors, e.g. insufficient sampling frequencies or erroneous analysis methods.

The approach presented here aims to determine the most likely model parameters by minimizing the parameter errors. Model and measurement errors are not rectified in the process. Therefore, experience and engineering judgment is required to choose a suitable model and similarly, a suitable sensor system. The presented procedure can be divided into the following five steps:

1. Sensor layout: First, a sensor system must be designed to measure the structural response. Sensor types and location should be chosen to monitor the global structural behavior rather than local effects. In most of the literature the dynamic properties (eigenfrequencies, mode shapes, modal damping) are considered for this purpose. In recent years, efforts are made to combine static and dynamic response for finite element update, e.g. Xiao et al. (2014).
2. Definition of structural parameters: To fit a model to the measured response, parameters that can be adjusted in the optimization step must be defined. These parameters can account for material properties, the stiffness of springs representing bridge bearings or connections as well as prestress forces or other properties. If the updating process is used for damage detection, parameters for a damage model must be provided. The calibration of a model can only be successful if suitable parameters are selected.
3. Definition of an objective function: The discrepancy between the simulated and the measured response must be quantified by an error or objective function. For example, the squared relative error between eigenfrequencies is an often applied measure. A minimum of the objective function is computed in the optimization step in order to determine the values of the structural parameters.
4. Sensitivity analysis: This step is optional and can be used to exclude parameters with no significant influence.
5. Optimization: Finite element update is a nonlinear optimization problem that in most cases does not possess a unique solution. Multiple model parameters can have a similar influence on the objective function and the problem may be overdetermined or underdetermined. Solving this inverse ill-posed problem might require several iterations to choose a suitable algorithm with appropriate initial values and hyperparameters. There are countless optimization algorithms that in general can be applied to minimize the objective function, each with strengths and weaknesses. A full review about computational methods for finite element update can be found in Marwala (2010). This study uses the BFGS algorithm and the Simple Genetic Algorithm (SGA).

According to the second axiom of SHM by Farrar et al. (2012) damage assessment requires a comparison of two system states. Applied to finite element update, this implies that the model parameters are firstly determined with measurement data from a reference state. Secondly, the procedure is repeated with data from a current state. Damage in the structure can be deduced from changes in the model parameters. In addition to an explicit comparison of two system states, damage detection can to some extent also rely on implicit assumptions. For example, large local deviations of stiffness parameters, especially a local reduction, is a strong indication of damage.

In order to not only identify but also to quantify damage in a structure, a mathematical model for the damage must be specified. A model for tendon and wire failures is introduced in Section 2. However, due to its complexity, calibrating this damage model directly will result in high computational costs. Instead, an approach with a simpler damage model is proposed for damage identification and localization.

The complex model is then adapted and optimized at the given location in a next step. A parabolic damage function first introduced by Teughels et al. (2005) is used as the simple model. The damage model shall be defined by a function $N(c, w, p)$, where c is the center of the damaged area, w is the width and p is a stiffness reduction factor.

4 NUMERICAL EXAMPLE

To demonstrate the damage detection procedure, a single span prestressed bridge girder with realistic dimensions is used as a numerical example. The length of the T-girder is 34 m, the height is 2.2 m and the width of the carriageway slab is 5 m (Figure 4). The cross section consists of a precast prestressed element with an in-situ concrete slab (both C40/45). The beam has eight parabolic prestressed tendons with subsequent bond. Two finite element models of the structure are created: a nonlinear reference model for damage simulation and a linear beam model for damage detection. The reference model uses rectangular shell elements with a width of 0.2 m. The prestressed tendons are modeled as described in Section 2.

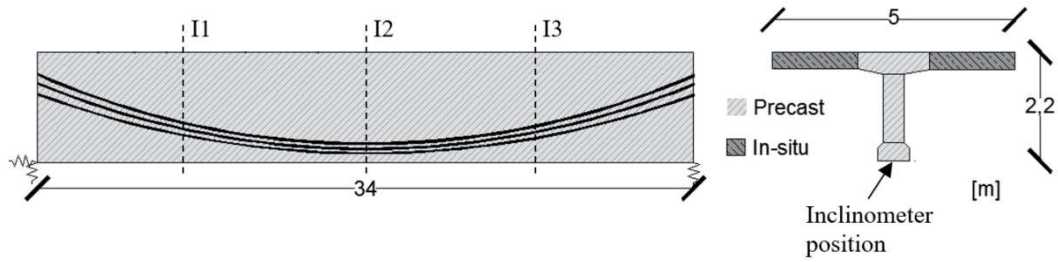


Figure 4. Bridge girder for numerical example.

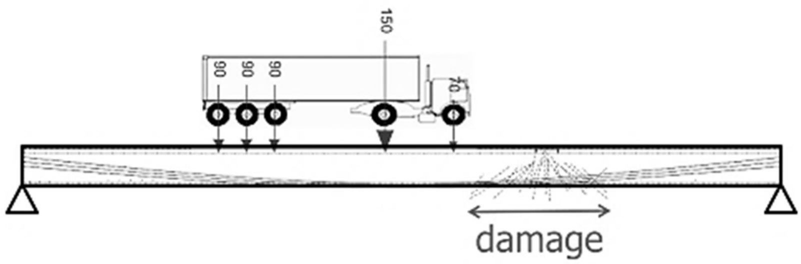


Figure 5. Cracked bridge girder under simulated traffic load (5 axle truck).

The beam model uses 68 elements with a length of 0.5 m and does not include the tendons. The sensor system consists of 3 inclinometers at ¼ (I1), ½ (I2) and ¾ (I3) of the span. Recent research shows promising result for the use of inclinometers for damage detection on concrete bridges, Bolle et al. (2017).

Cracks in a prestressed concrete bridge can be the result of a loss of prestress force due to partial tendon failure. If the combination of temperature and traffic loads causes decompression and an exceedance of the concrete's tensile strength, cracks will begin to appear. Due to the prestress force, the cracks will almost completely close again once the load is gone. This effect is simulated with an external load of 1140 kN at 23.5 m from the left abutment which induces cracks with a maximum width of 0.27 mm. After the release of the load, the remaining maximum crack width is 0.03 mm, a width that is hardly visible to the human eye. The damaged but unloaded model is used to simulate the crossing of a truck with 5 axles (Figure 5).

The calibration procedure follows the second axiom of Farrar et al. (2012) and directly compares a healthy state with a damaged state. In a real-world application, the first step would be to determine global structural parameters such as the Young's modulus of the precast concrete or the stiffness of the bridge bearings in the healthy state. The resulting model could then be further improved by considering the increased bending stiffness due to the prestressed cables which will be the first step in this numerical example. For this purpose, the beam is divided into 14 stiffness sections resulting in same number of model parameters for optimization. Signals of a truck crossing the bridge are simulated with 40 load cases with the reference model as well as with the beam model (Figure 6). The sum of the absolute squared differences between the respective maxima and minima of the signals are considered as the objective function. The BFGS algorithm is used to solve the optimization problem in the healthy state. The resulting bending stiffness distribution is illustrated in Figure 7 (Left). The shape corresponds to the parabolic tendon profile. An increase of almost 8 % is in good accordance with rough estimates.

The stiffness distribution of the healthy state serves as the baseline for the update process in the damaged state. The damage function $N(c, w, p)$ from Section 3 is applied as the damage model. As a result, there are 3 parameters in the optimization step for which the SGA is used as a solver. Figure 7 (Right) shows the resulting stiffness distribution with a stiffness reduction up to 24 %. As can be seen, the center of the damage function (c = 24.5 m) is off by 1 m but still allows for a good localization of the damage. The resulting width w = 7.5 m of the damaged area is an accurate estimate. The simulated signals with the updated beam model are in good accordance with the damaged reference model, especially for the sensors I1 and I2 (Figure 6 Left and Middle). The reference signal form I3 (Figure 6 Right) shows a sharp bend after reaching the maximum in the damaged state. This bend is induced by a nonlinear stiffness change in the damaged area under the heavy rear axles of the truck. The linear beam model cannot account for the nonlinear behavior.

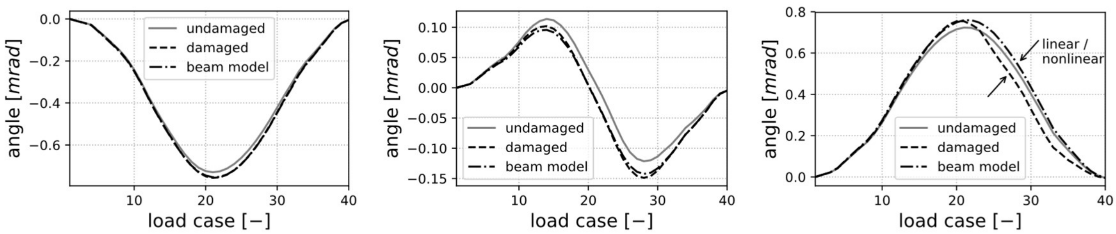


Figure 6. Simulated inclinometer signals during truck crossing from undamaged/damage reference model and beam model; Left: I1; Middle: I2; Right: I3.

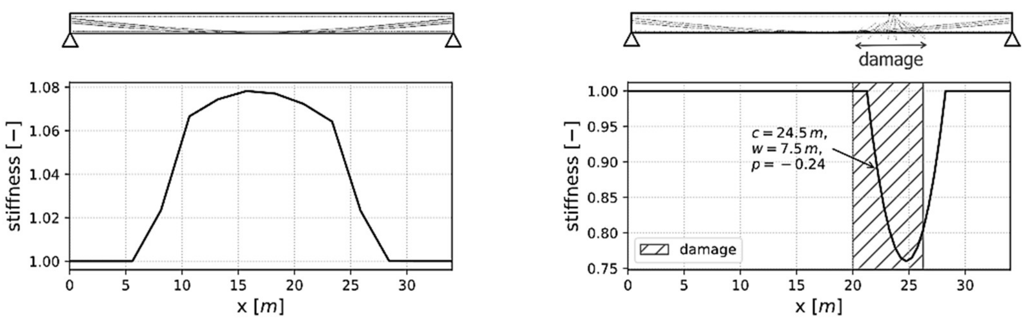


Figure 7. Stiffness change after update of beam model; Left: Stiffness increase due to prestressed tendons in undamaged state; Right: Stiffness decrease due to damage.

5 CONCLUSION

In order to achieve a predictive maintenance management of road bridges, two essential questions must be answered: what are possible damage scenarios and how can these scenarios be detected before the damage reaches a critical threshold? This paper investigates the damage scenario of wire and tendon failure in prestressed concrete bridges. The prerequisite is a mathematical model of the damage, for which a practical approach is introduced. The model takes the residual load-bearing capacity of the broken tendon and the ability of the prestressed concrete for internal load distribution into account. Validation with test results shows a sufficient accuracy of the approach in case of a good bond between tendon and grout. Extensive experimental studies are required to quantify the effects of a reduced bond. A strategy for damage detection using finite element update is presented and applied in a numerical example. In the example, a simple damage model and the signals from only three sensors is sufficient to detect and localize damage that causes concrete cracks. The simple model gives an estimate of the extent of the damage but fails to reproduce nonlinear effects. In a next step, the identified structural damage should be further examined with the more sophisticated damage model for tendon failure which can account for nonlinear behavior and therefore allows for an accurate damage assessment. In our future work we intend to investigate how the complex damage model can be incorporated directly into the updating process. Moreover, it is important to examine how different sensor types and setups can improve damage detection.

6 REFERENCES

Abdelatif, A., Owen, J. and Hussein, M., 2012, Modeling the re-anchoring of a ruptured tendon in bonded post-tensioned concrete. *Bond in Concrete 2012: Bond, Anchorage, Detailing, Volume: 1.*

Alkayem, NF, Cao, M., Zhang, Y., Bayat, M. and Su, Z., 2017, Structural damage detection using finite element model updating with evolutionary algorithms: a survey. *Neural Computing and Applications*, *30(2):389–411.*

Azizinamini, A., 2012, Improved inspection techniques for steel prestressing/post tensioning strand. *Final Report to Florida Department of Transportation, BDK80-977-13.*

Bolle, G., Mertzsch, O. and Marx, S., 2017, Messtechnische Dauerüberwachung zur Absicherung der Restnutzungsdauer eines spannungsrisskorrosionsgefährdeten Brückenbauwerks. *Beton- und Stahlbetonbau*, *112(2):75–84.*

Cavell, DG and Waldron, P., 2001, A residual strength model for deteriorating post-tensioned concrete bridges. *Computers & Structures, 79(4):36–373.*

Coronelli, D., Castel, A., Vu, NA, and François, R., 2009, Corroded post-tensioned beams with bonded tendons and wire failure. *Engineering Structures, 31(8):1687–1697*.

Eichinger, EM, 2003, Beurteilung der Zuverlässigkeit bestehender Massivbrücken mit Hilfe probabilistischer Methoden. *Technische Universität Wien, Fakultät für Bauingenieurwesen.*

Farrar, CR and Worden, K., 2012, Structural health monitoring: a machine learning perspective. *John Wiley & Sons.*

König, G. and Maurer, R., 1990, Sicherheit von Spannbetonbrücken. *Bundesmin. für Verkehr, Abt. Straßenbau, Forschung Straßenbau und Straßenverkehrstechnik, 590*.

Marwala, T., 2010, Finite element model updating using computational intelligence techniques: applications to structural dynamics. *Springer Science & Business Media*.

Mottershead, J. and Friswell, M., 1993, Model updating in structural dynamics: a survey. *Journal of Sound and Vibration, 167(2): 347–375.*

Siegert, C., Holst, A., Empelmann, M. and Budelmann, H., 2015, Überwachungskonzepte für Bestandsbauwerke aus Beton als Kompensationsmaßnahme zur Sicherstellung von Standsicherheit und Gebrauchstauglichkeit. *Berichte der Bundesanstalt für Straßenwesen, B 118*.

Teughels, A. and De Roeck, G., 2005, Damage detection and parameter identification by finite element model updating. *Archives of Computational Methods in Engineering*, *12(2):123–164.*

Vill, M., 2005, Zum Tragverhalten von Massivbrücken mit geschädigten Spanngliedern. *Beton- und Stahlbetonbau, 100(2):211–214.*

Xiao, X., Xu, Y. and Zhu, Q., 2014, Multiscale modeling and model updating of a cable-stayed bridge II: Model updating using modal frequencies and influence lines. *Journal of Bridge Engineering*, *20(10):04014113.*