Damage Detection Based on Structural Response to Temperature Changes and Model Updating

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DAMAGE DETECTION BASED ON STRUCTURAL RESPONSE TO TEMPERATURE CHANGES AND MODEL UPDATING

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ABSTRACT

The paper proposes use of measured structural response to temperature loads for purposes of damage identification. As opposed to the most common approaches, which rely on suppressing temperature effects in damage detection, the method proposed herein utilises measured changes of structural sensitivity to temperature loads. Applicability of this approach varies by structural type and therefore results of system identification are presented for different structural types. The method aims at identifying structural damages that produce stiffness change as well as deformation states and boundary condition stiffness. In particular, the method is applicable for evaluations without available measurements known as healthy baseline. Presented examples comprise 2 bridge monitoring applications.

KEYWORDS : *damage, detection, temperature, model updating, sensitivity.*

INTRODUCTION

Successful damage detection on bridges highly depends on extracting suitable indicators from monitoring data. These indicators should fulfil the requirement of being highly correlated with damage phenomena. One of the first indicators used for damage detection were dynamic properties: eigenfrequencies, mode shapes and later also damping. Model updating based on data from vibration monitoring was a topic of many scientific contributions. A summary of vibration-based damage detection techniques was given by Doebling [1]. The research headed later towards combination of different indicators, example of which is the contribution of Shih [2], where a multi-criteria approach using vibration-based indicators was proposed. The combination of different indicators was extended to the use of static measurements. Application of such approach on a long-span suspension bridge was shown by Wang [3]. The necessity of engineering judgement in the process of model updating was described by Schlune [4], who emphasized the need of manual model adjustments prior to model updating.

The effect of temperature on damage detection results was detected early. Eigenfrequency changes caused by temperature effects on different structures were described in the works of Peeters&DeRoeck [5], Farrar [6] or Ralbovsky [7]. The effect of temperature was considered as an obstacle for damage detection. Various methods for removing this effect were proposed, for example in the works of DeRoeck [8], Hu [9] and many other scientists.

The present paper aims not at supressing the temperature effect, but rather the opposite: its use for damage detection purposes. The methodology is explained in chapter 1, following by presentation of simple demonstrative numerical examples in chapter 2. The method is applied on two bridges described in chapter 3. Results of simulated damage scenarios are shown in chapter 4.

1 METHODOLOGY

Temperature load represents a static load case. Structural response to a given temperature loading is determined by structural stiffness matrix *K*, thermal expansion properties and the boundary conditions. For the success of damage detection using the proposed method it is vital that the damage to be identified affects the stiffness matrix. Typical examples of damage cases affecting the stiffness are development of cracks in concrete and change of boundary condition's stiffness. Furthermore, bridge deformation can also influence the stiffness matrix due to second-order effects.

If the vector of structural response *X* (which includes displacements, strains and inclinations) to a given temperature loading ΔT is dependent on structural parameters influencing the stiffness matrix *K*, then identification of structural parameters based on measured structural response *X* is possible. The identification can be accomplished by model updating, which is defined by solving an optimization problem. The procedure implies evaluation of an objective function *J* that quantifies the discrepancy between measured and calculated responses. Mathematical optimization procedures are then used to adjust structural parameters *p* in such way as to minimize the objective function *J*.

In the proposed method, the objective function is defined as:

$$J(p)=\sum_i \frac{1}{\sigma\left(\frac{\partial X_{m,i}}{\partial T}\right)}\cdot\left(\frac{\partial X_{c,i}(p)}{\partial T}-\overline{\frac{\partial X_{m,i}}{\partial T}}\right) \tag{1}$$

with $\frac{\partial X_{c,i}(p)}{\partial T}$ and $\overline{\frac{\partial X_{m,i}}{\partial T}}$ being the calculated structural sensitivity of quantity *i* (displacement, strain or inclination at a given point on a structure) to temperature change and the mean of its measured counterpart, respectively. The difference between the calculated and measured sensitivities is then divided by standard deviation σ of the measured sensitivity, which ensures normalization of different quantities, as well as their weighting by measurement uncertainty.

2 PRESENTATION OF SIMPLE EXAMPLES

The use of changes in structural sensitivity to temperature loading is presented in this chapter using simple numerical examples. Structural damage, which produces stiffness reduction, changes structural response to temperature loading, as it is shown in the example of continuous girder below. On structures that have pronounced second-order effects, the structural response changes with deformation state, which is shown below using an example of single span girder. The examples presented in this chapter are solely results of numerical calculations.

2.1 Effect of damage on structural response to uniform temperature

The following example is a three-span continuous girder, on which stiffness reductions were introduced and their effects analysed. The girder has span lengths of 6 m, 8 m and 6 m. Its cross-section comprises 30 x 60 cm and the utilised material has elasticity modulus of 30 GPa. Beam elements were used for the model. Introduced damages are listed in Table 1. The damage extent was varied between 0% (no stiffness reduction) and 100%, which represents damage extents as listed in Table 1.

Table 1: Damage cases simulated on three-span girder

Damage case	Location, affected length, max. extent
dc1	At support S2, L = 0.7 m, $K_{red,max}$ = 70%
dc2	Midspan of central span 2, L = 0.8 m, $K_{red,max}$ = 50%
dc3	At support S3, L = 0.7 m, $K_{red,max}$ = 70%
dc4	Midspan of right span 3, L = 0.8 m, $K_{red,max}$ = 50%

The girder was assumed to have permanent deformation of max. 3 cm (Figure 1(a)). Stiffness of horizontal springs is 512 MN/m. Damages were introduced to the model and it was loaded with uniform temperature change of ΔT=+1 K. Strain response to the temperature load was calculated at the midspan of the left and central span's bottom fiber, in different damage states. Structural strain response to unit temperature loading, referred in the following as strain sensitivity [µm/m/K], exhibited changes dependent on introduced damage. Thus it can be concluded that strain sensitivity could be used in this case as indicator of damage.

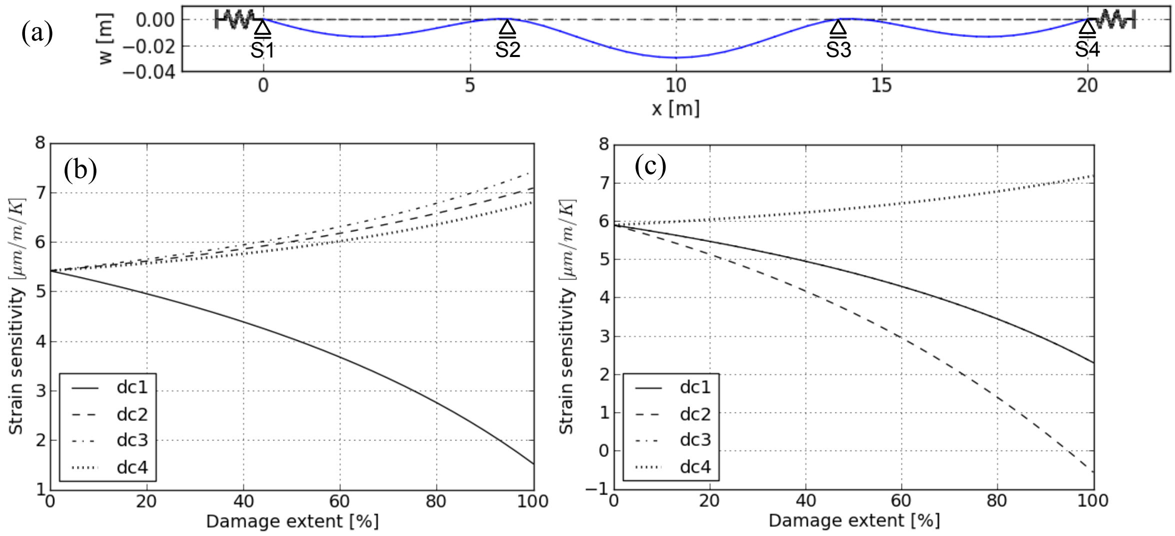

Figure 1: Three-span girder. (a) Model with permanent deformation, (b) Strain response in midspan of left span, (c) Strain response in midspan of central span.

2.2 Effect of permanent deformation on structural response to uniform temperature

This example of single-span girder is presented to illustrate second-order effects on structural sensitivity to temperature loading. A simple single span girder with 9 m span was modelled using the same cross-section and material as in previous example. Amplitude of permanent deformation was varied (0 – 34 cm), as well as stiffness of horizontal springs (8 – 8192 MN/m). In these different configurations, sensitivity of strain and deflection in midspan to uniform temperature change of ΔT=+1 K was calculated. It was found that both strain and deflection sensitivity (Figure 2) exhibit distinct changes at higher values of horizontal stiffness of boundary condition.

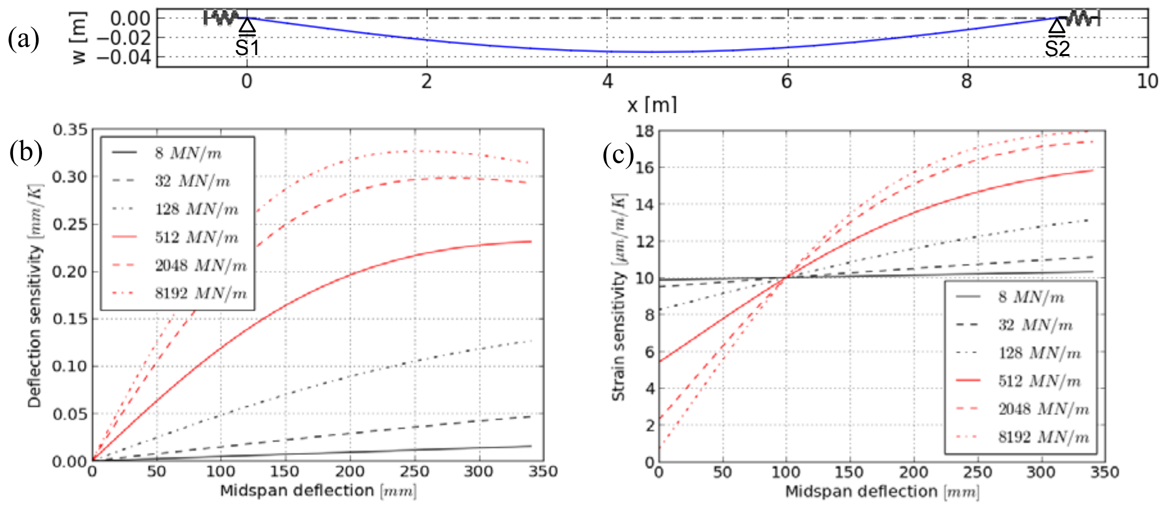

Figure 2: Single-span girder. (a) Deformed model, (b) Deflection response in midspan and (c) Strain response in midspan in 6 cases of horizontal spring stiffness ranging from 8 to 8192 MN/m.

The value pairs of strain and deflection sensitivity can be uniquely linked to value pairs of permanent deformation and spring stiffness. Thus, measurements of strain and deflection sensitivity can be used for identification of permanent deformation and horizontal spring stiffness. However, at low values of horizontal spring stiffness, the sensitivity is radically reduced, which would render identification of permanent deformation unfeasible.

3 BRIDGE MONITORING SYSTEMS

Two bridges with running monitoring installations were analysed as case studies. The bridge structures and monitoring installations are described in this chapter, while simulated damage scenarios and evaluated identification capabilities are shown in chapter 4.

3.1 Prestressed concrete box-girder bridge with tie bars

The prestressed concrete box-girder "Teufenbach" Bridge, located in Austria, was erected in 1959. The main span is 64 m long, with additional cantilevers of 4.8 m on each side. Both of them are connected to the foundation by prestressed concrete tie bars. Visual inspection of these bars' structural condition is not possible, as they are entirely covered with soil. The box-girder cross-section has a variable height ranging from 1.3m to 3.7m, which results in a slenderness of 1/50 in the central mid span. The superstructure is fixed to the concrete abutment. The bridge has a shallow foundation with diameters 3 to 10m, i.e. with no piles.
The Structural Health Monitoring (SHM) system consists of the following sensors (Figure 3): 2 electronic water level gauges in the mid span, 6 strain gauges, 4 structural temperature sensors inside each wall of the box-girder and 2 acceleration sensors (Reiterer [10]).

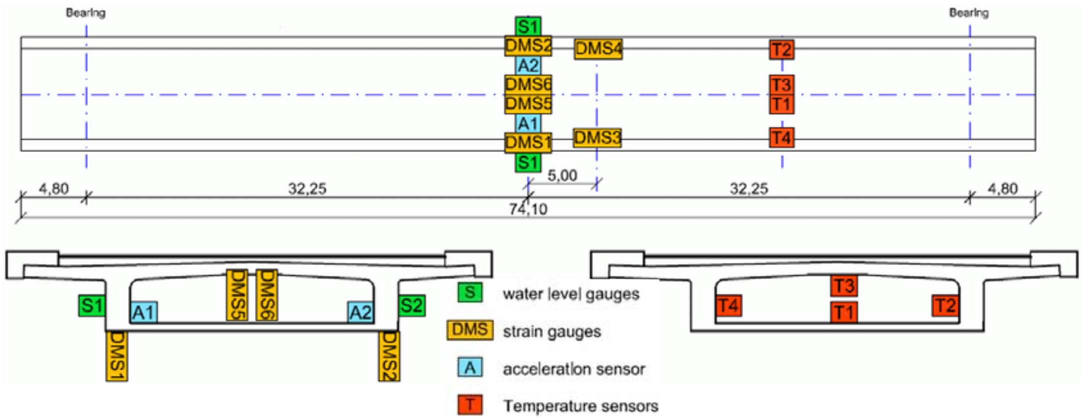

Figure 3: Monitoring system installed on Teufenbach Bridge

3.2 Cable-stayed bridge

The International Bridge over River Guadiana is a cable stayed bridge located in the South West of the Iberian Peninsula and was opened to traffic in 1991. The bridge has a central span of 324 m and two lateral and transition spans of 135 m and 36 m, respectively. The deck is a prestressed concrete box girder 18m wide and 2.5m high, suspended by one hundred and twenty-eight stay cables. They are composed by individually sheathed mono strands, varying from 22 to 55, with the length from 48m to 167m, and are spaced every 9.0 m on the deck and every 1.8 m on the pylons. The A-shaped pylons are 95m and 96 m high and consist of concrete hollow sections which, besides anchoring the cables, support the deck at a height of 35 m by means of hollow section transverse beam.
In 2010 an autonomous online SHM system was installed with the aim of carrying out early damage detection, thus contributing to increase of safety and to reduction of maintenance costs. Sensor positioning was based on the principle that any damage in the bridge would produce dead load redistribution and change in cable forces. Consequently, displacement and rotation changes close to cable anchorages were chosen as sensitive quantities that should be measured (Santos [11]).

Hydrostatic pressure cells (NL) and magnetostrictive transducers (DH) were used for measuring deck and joint displacements, respectively. Bi-axial inclinometers (CL) were installed on the top of the pylons. Differential displacements and rotations are also monitored using the same type of inclinometers (CL), installed in each foundation and abutment (Figure 4).

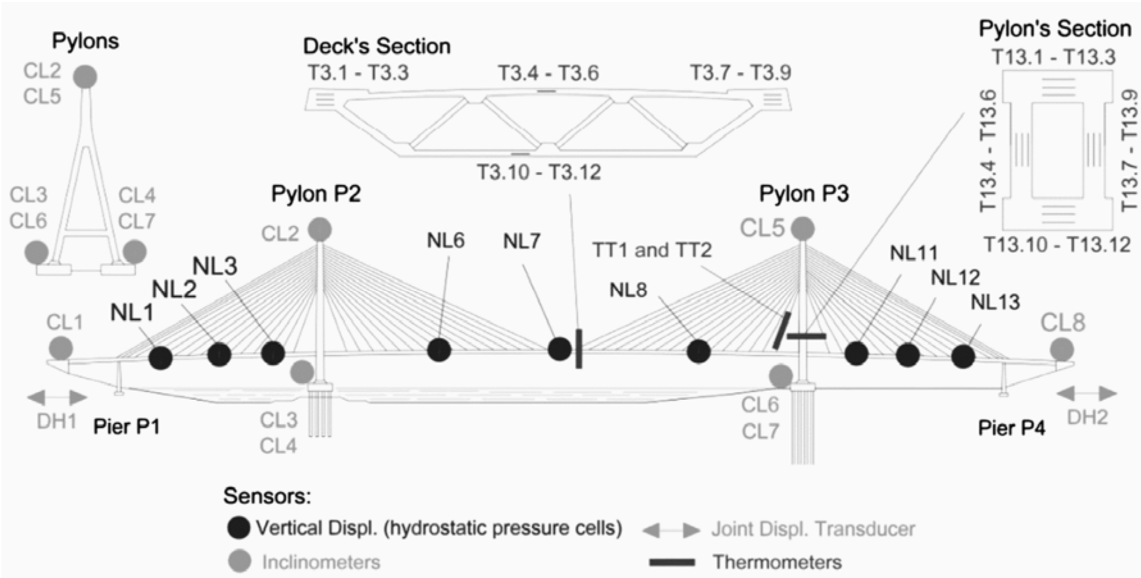

Figure 4: Monitoring system installed on Guadiana River bridge

3.3 Finite-Element Models

The Finite-Element models that were used in model updating (Figure 5) were constructed using commercial software packages. Soil behind the abutments of Teufenbach Bridge was modelled using horizontal and vertical springs, the stiffness of which was adjusted in model updating process.

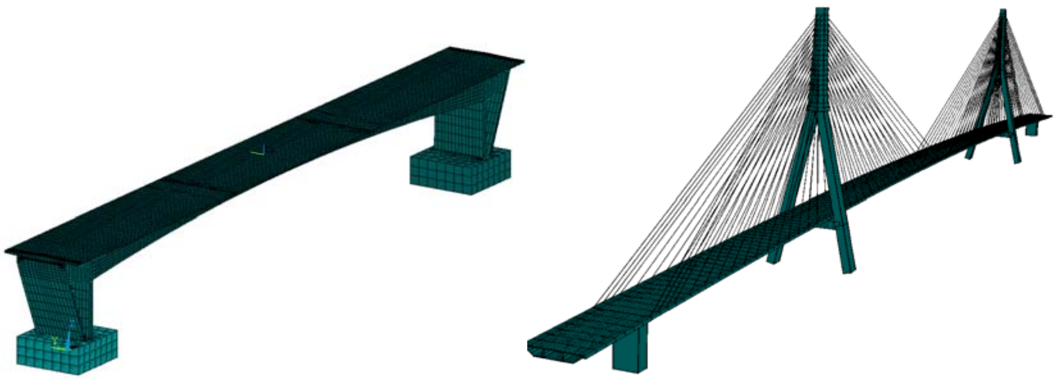

Figure 5: Finite-Element models. Left: Teufenbach Bridge, right: Guadiana River Bridge

4 SIMULATED DAMAGES AND MODEL UPDATING

Structural identification performed on these two bridges aimed at identification of the current state and evaluation of damage detection capability in particular simulated damage cases. Identification of current deformation state was performed on the Teufenbach Bridge that has a structural type suitable for such application. Relevant damage scenarios were simulated on both bridges.

4.1 Identification of deformation state and boundary stiffness

The Teufenbach Bridge is of a structural type sensitive to second order effects. The vertical deformation has a distinct effect on bridge response to temperature. Also the soil behind abutments affects significantly the structural response. Model updating using the two model parameters: boundary condition stiffness and permanent deflection at midspan was carried out. Model optimization was performed with the target of fitting to measured values of deflection sensitivity at midspan (0.82 mm/K), and strain sensitivity in the midspan at girder bottom (11.0 µm/m/K).
Figure 6 shows objective function values for different combinations of boundary stiffness *k* and midspan deflection *w*. The optimal solution was reached for parameter value pair of *k=15000 MN/m* and midspan deflection of *w=23 cm*.

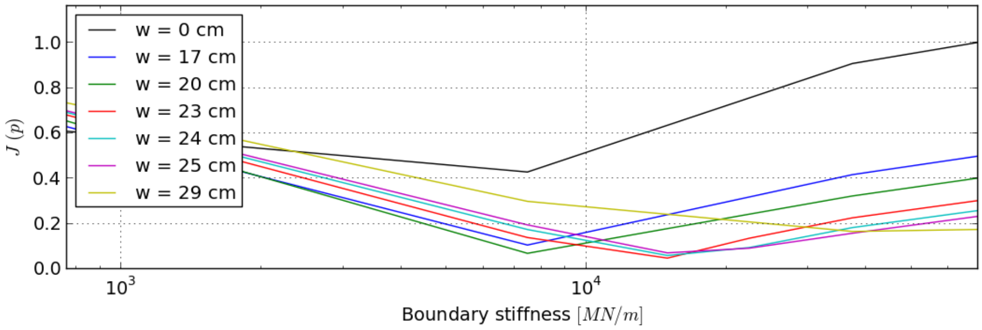

Figure 6: Objective function values during model updating

The updating results were verified by measurement of bridge geometry using precision total station. By comparison of measured bridge geometry with construction plans, a deformation of 23 cm at midspan could be determined. The displacement measured by total station matched exactly the value calculated by model updating.

4.2 Damage scenarios on Teufenbach Bridge

In order to investigate the capability of damage detection, several damage scenarios were simulated. Six types of damages were simulated (Table 2) and each was introduced using different extents. Damage types 3 and 4 produced change of bridge stiffness, whereas damage types 1,2,5,6 were assumed without change of structural stiffness.

Table 2: Introduced damage types

Damage type	Description
1	Failure of prestressing cable in tie bars, without crack formation
2	Failure of continuous prestressing cable, without crack formation in box-girder
3	Stiffness reduction of tie bar
4	Stiffness reduction of box-girder slab
5	Foundation movement
6	Failure of short prestressing cable at midspan, without crack formation

Table 3 shows the results of calculated structural response to various damage cases. Within the four columns of results, the first two represent change of deflection- and strain-sensitivity to temperature [%], whereas the last two columns represent the expected change of displacement [mm] and strain [µm/m] absolute values. The cells with values that are considered detectable are highlighted. As expected, the damage types that do not change structural stiffness have little effect on displacement- or strain-sensitivity to temperature, whereas damage types 3 and 4 are detectable. On the other hand, changes of absolute values of deflection or strain are sensitive also to damage types that do not change structural stiffness, but produce detectable deformations.

Table 3: Structural response to introduced damage scenarios

Introduced damage			Change of temperature sensitivity		Change due dead load effect	
Type	Extent		$\Delta(U_z/T)$ [%]	$\Delta(\varepsilon/T)$ [%]	$\Delta\ U_z$ [mm]	$\Delta\varepsilon$ [μm/m]
1	1	failed cables	-0.01	0.00	0.10	0.41
	2		-0.03	0.00	0.18	0.71
	6		-0.09	0.00	0.49	1.94
2	1	failed cables	-0.17	-0.09	0.67	8.27
	2		-0.15	-0.09	0.60	8.20
3	10	[%] stiffness reduction	-0.31	-0.09	0.07	0.28
	20		-0.65	-0.17	0.15	0.58
	50		-1.93	-0.52	0.45	1.72
4	1	[%] stiffness reduction	0.75	0.00	0.12	0.04
	5		3.83	-0.17	0.62	0.22
	10		7.87	-0.26	1.29	0.46
5	0.1	[cm] movement	0.10	0.00	-0.56	-2.10
	0.3		0.30	0.09	-1.68	-6.30
6	1	failed cables	-0.18	-0.09	0.68	5.32
	2		-0.37	-0.17	1.42	13.33
	6		-1.08	-0.52	4.14	40.66

4.3 Damage scenarios on the Guadiana River Bridge

FE-model of the cable stayed bridge was changed in 37 damage types and each damage type was simulated with different extents. Beside reduction of cable stay stiffness, damages in bridge deck and pylon were simulated. In this chapter, the scenarios with cable stay stiffness reductions are presented. In each scenario, the stiffness of one single cable was reduced starting from 0% up to 90% of its original stiffness. Figure 7 shows the structural response in terms of dead load effect (left) and temperature effect (right). Each line represents the effect of stiffness reduction of one particular cable stay.

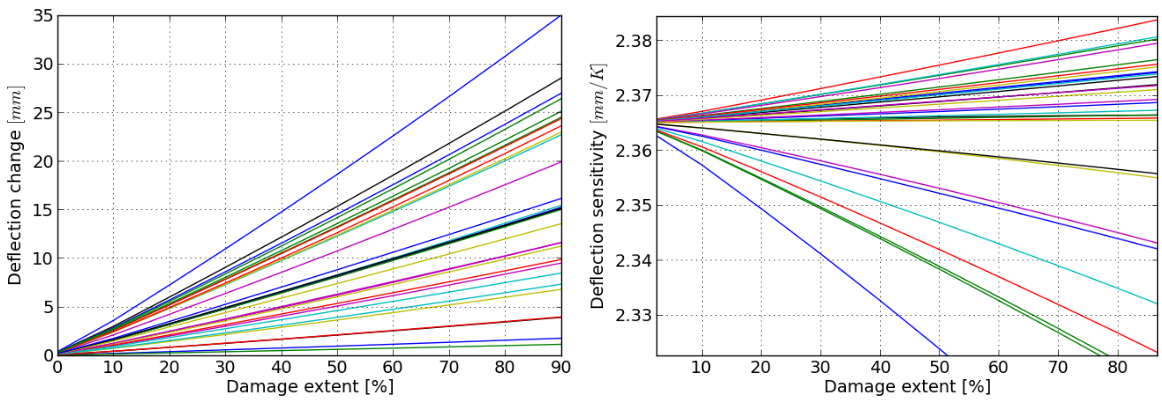

Figure 7: Effect of cable stay stiffness reduction. Left: absolute displacement change [mm], right: Deflection sensitivity to temperature [mm/K]

Considering the fact that deflection sensitivities of more than 0.01 mm/K are considered to be detectable, whereas the detectability limit of change in absolute displacement is 0.2 mm, it can be observed that using deflection sensitivities to temperature gives no advantage over using absolute displacement values in this case. The advantage would be present in the cases where no measurements of healthy state would be available. In such cases, the changes of absolute displacement would not be available, whereas the deflection sensitivity can still be determined.

CONCLUSION

The damage detection method presented in this paper shows the applicability of using structural response to temperature as indicator of structural changes and its use in the objective function of model updating algorithm. It was shown in the examples that the method was successful in identifying structural deformation state on a bridge that is sensitive to second order effects, where the calculated deformation value was also verified by measurements of bridge geometry. Furthermore, it was concluded that measured absolute change of structural deformation (displacement, strain, inclination) in different damage cases can be considered as a more sensitive indicator than change of sensitivity to temperature. However, in cases where measurements before occurrence of damage are not present, such data on change of deformation is not available. The sensitivity of structural response to temperature can still be measured, thus the presented method can still offer a possibility to identify the structural state.
It was shown in the presented calculations that deformation sensitivity to temperature varies between structural types and simulated damage cases. Therefore, performance of the method should be evaluated for each particular application. If available, it is recommended to use this identification procedure in addition to other detection methods, for example methods based on dead load redistribution, to achieve higher reliability and robustness of results.

ACKNOWLEDGEMENTS

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