



The Deformation Area Difference (DAD) method for condition assessment of reinforced structures

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ABSTRACT

The investigation and condition assessment of bridges have a very high priority in the construction industry today. Particularly, due to the fact that many bridge structures are getting old and partly reach the end of their useful life, the control and condition assessment of bridge structures have become very important and essential. The present research work introduces an efficient new method for condition assessment called the Deformation Area Difference (DAD) Method. This new method represents an attractive alternative to visual inspection and long-term monitoring. In this paper, the new method with its theoretical background is presented and explained by means of a laboratory experiment and some additional theoretical calculation examples. The experimental investigations have been realised on a reinforced concrete beam, which has been gradually loaded until failure. For each load step, the stiffness reduction and the apparent cracking have been monitored. High-precision measurements such as close-range photogrammetry, digital levelling and displacement sensors have been used for the determination of the deflection curve. The DAD method has been applied to identify the area of the crack pattern of the laboratory experiment. Furthermore, the method is discussed with regard to the load level and the precision of the deformation measurements. On the basis of the laboratory experiment, the applicability of the DAD method for damage detection could be proven. Furthermore, the sensitivity of the method with regard to the damage degree, the static system, the damage position and the impact of temperature variation were analysed.

1. Introduction

Owners of civil engineering structures have to ensure the load bearing capacity, safety and durability of structures under consideration of the economic efficiency. This requires regular and competent inspections of these structures, such as bridges, which are key elements of a long-term economic conservation strategy. In future years, the biggest challenge years will be to develop procedures, which allow an easy and cost-effective condition assessment of bridge structures. Many of the existing bridge structures worldwide were built using reinforced and prestressed concrete design concepts. The origins of most of the damages of reinforced or prestressed concrete structures, which lead to a stiffness reduction, occur inside of the structures. Currently the control and the condition assessment tasks are carried out by visual inspection [1,2], leading to a detailed damage analysis if required. Today bridge structures are subjected to increasing traffic volumes while vehicles are also becoming heavier. Furthermore, with increasing age the bridges are also exposed to chemical attacks caused by, e.g., water or oil penetration [3]. Techniques such as radar detection [4], infrared

thermography [5], ultra-sonic measurement [6], half-cell method [7], impact-echo method [8], chain dragging and hammer sounding [9] are suited for large-scale non-destructive investigations. The functionality of these methods is commonly similar, whereby each inspection technique is specialised for detection of certain damage types and has its own characteristics and limitations. Most of them are used for condition assessment of bridge decks, but the applicability is limited, e.g., to a certain range of temperature, to a certain depth where damage can still be localised, to a certain moisture content or due to challenging data interpretation, etc. [10]. In contrast, methods like the calcium-carbide method, endoscopy [11], rebounding hammer (Schmidt hammer) [12] or laboratory investigations are deployed to further locate and characterise an identified damage after a visual inspection of the bridge structure. Using the given inspection methods stiffness reducing damages cannot reliably be detected.

A global or local stiffness reduction such as, e.g., concrete cracking, failure of tendon or reinforcement and a deficit of tendon coupler can influence the load bearing capacity of a structure, which can be analysed by a load-deflection experiment. Several research projects are

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focused on the identification of stiffness reducing damages by means of dynamic and static analysis of bridge structures [13–15]. It has been proven [16] that the dynamic and static experimental data include decisive information about the stiffness reduction and the respective damages. Boumechra [17] presented in his work a new approach for damage identification for bridge structures using numerical calculations. Particularly, he took into consideration the static load deflection behaviour of two models with generated damage. The analysing factor was the deflection value at one position, but with variable position for an applied static force. Using his algorithm, he ascertained an approximate correspondence of damage detection. However, his method for damage detection requires the analysis of the initially undamaged beam, which poses a problem for existing bridge structures. A definitive and exact conclusion of the condition of a structure cannot be provided using the results of one single load deflection analysis when no reference measurements for a known reference condition are given. Stöhr et al. [18] investigated in their work the influence line of a laboratory beam and a real bridge with local damages. Thereby, an inclinometer was installed at the support, measuring the change of the inclination angle with load increase. A discontinuity in the curve of the influence line could be identified due to damage of the structure. However, by considering only the measured influence line the localisation of damage turned out to be difficult because of its unsteady curve and some noise in the measurement. The measured influence line could be used for the identification of structural damage, but the measurements were affected by noise. In their further research, it is planned to use measured static influence lines for finite element model updates in order to allow a reference-free condition assessment. Li [19] investigates the issue of damage identification of structures using optical measurements. In the extended literature review, the dynamic damage identification methods and static response data-based damage identification methods are compared. In comparison to dynamic-based identification methods, the literature of static methods is limited. Static load deflection methods often require continuous displacement measurements along the beam, which is unrealistic depending on the size of structure, on the local conditions or on the measurement techniques. Therefore, some research develop methods such as spline interpolation or moving load analysis to generate a complete set of displacement data [20,21].

He et al. [22] present a damage detection method for beam structures which are loaded by a quasi-static moving load. This deflection method is based on displacement measurements where the relationship between damage parameters and displacement influence lines is investigated. The theoretical background of the method which is based on Euler-Bernoulli [23], is presented numerically and experimentally. First the localisation of the damage is achieved by using the displacement influence line (DIL). Then the damage index is quantified. The changes in the DIL point to damages whereas the damage localisation index is calculated. The damage quantification requires a certain number of displacement sensors, which should measure close to damage. The results from the calculation and from the experiment show peaks in the area of damages. However, the author mentions difficulties for damage detection due to measurement noise effects. Further corresponding damage detection methods using moving loads are presented in [24–26]. Sun et al. [27] develop a method based on curvature to detect damages in structures such as bridges. The damage respectively the discontinuity of the curvature curve is identified by using static load deflection measurement with variable load positions. The measurement takes place using a displacement sensor at mid-span of the beam. Furthermore, they investigate the influence of measurement noise and try to reduce noise effects. The localisation of discontinuities was possible depending on the ratio of noise and damage.

Another approach for condition assessment of bridges is given by dynamic excitation of the structures and analysis of their response. These methods need reference measurements and thus, require long-term structural health monitoring over a longer time period [28]. Therefore, a large volume of observation data has to be processed and

reliably evaluated. The installation of a bridge monitoring system is often linked to the issue of establishing and guaranteeing a fixed reference point for the measurement techniques. Moreover, the dynamic data analysis requires broad experience and practice to differentiate all external influences such as environmental impacts, various traffic loads, the stiffness influencing asphalt layer [29] and temperature effects [30]. By using dynamic analysis the global excitation of a structure is provoked, which includes all global effects of structure such as the influence of the support conditions, the stiffness change due to temperature variation of asphalt etc. In order to identify all influencing factors, several in-situ bridge tests have been carried out. Sung et al. [31] tried to determine the initial static and dynamic behaviours using a series of experiments on a newly constructed bridge to determine its initial condition. Using the finite element model updating the evaluation of bridge safety thresholds could be defined. Tracking and monitoring of the bridge will continue into the future to make appropriate decisions in case of threshold exceedance. The further optimisation of dynamic analysis could show potentials for condition assessment of bridges, but the related efforts and time consumption should be taken into account.

Lee [32] used the ambient vibration data for the damage diagnostic of steel girder bridges. The potentially damaged member of the bridge is screened with the mode shape curvature, which is calculated from the second derivatives of the identified mode shapes. The localisation of damage was realised, but there were some false damage alarms at several locations in the screening process. With an increasing number of influencing factors on the data set, the probability of errors and ambiguous results increases.

So, it has been proven by several studies that the dynamic and static experimental data included decisive information about the stiffness reduction and respective damages. However, the fundamental problem of the condition assessment of bridges and damage detection remains the unknown initial condition of existing bridge structures as well as their sensitivity to global effects such as changes of temperature/humidity and changes of support conditions. The existing inspection techniques are specialised for certain damage types and have limited usability as well as reliability. Furthermore, the noise in the measurements of static or dynamic tests complicates a meaningful and clear condition assessment of existing bridges.

In order to solve these problems, a method, which does not depend on global effects, was developed within this research work to analyse and evaluate the remaining load-bearing capacity and reliability of existing structures. The proposed Deformation Area Difference (DAD) method requires data from a reference system this could be as well measurements from the initial condition of the structure as a numerical non-linear finite element model of the structure. The method is able to indicate all kind of stiffness reduction due to local damage of a bridge structure, independent of the degree of damage. The method is based on a simple load-deflection experiment using a specific data processing. The DAD-method is the first method which is able to detect all kind of damage by comparing a reference state to a modified damage state, being able to identify large damaged areas as well as small local damage. Furthermore, this is the first time that the deflection line over the whole length of a structure is measured and investigated as the current methods are too sensitive to noise effects. This is different for the DAD-method when combined to innovative measurement technologies where point clouds with minimum standard deviation are generated. Using the data of the load-deflection experiment, the inclination angle and the curvature were determined, whereby the discontinuity of the structure along the longitudinal axis can be detected and localised in case of local damage-induced discontinuities. However, the inclination angle and the curvature are calculated from the first, respectively second derivation of the deflection line, which behave unsteady without a precise measurement. Therefore, to validate the method on real structural elements, laboratory experiments on a gradually loaded reinforced concrete beam have been realised and several measurement

techniques were applied and compared. The damaged area of the concrete experimental beam was detected by applying the DAD-method. The applied measurement techniques were close-range photogrammetry, digital levelling, displacement sensors and strain gauges. The manuscript starts with the theoretical description of the DAD-method. Subsequently, the laboratory experiment will be presented in consideration of boundary conditions such as application of the measurement techniques, possible errors and the characteristic of the load deflection behaviour. Furthermore, the variability of each static system and the insensibility of the DAD-method to thermal effects will be presented using numerical finite element calculations.

2. The Deformation Area Difference-method

2.1. Relation between deflection line, inclination angle, curvature and stiffness

The deflection of a structure is generally calculated according to Eq. (1) and consists of deflections due to bending moments, shear forces, axial forces and temperature expansion/retraction. Therefore, it is obvious that the deflection of a structure depends on e.g. the loading, the temperature conditions, the material properties, respectively the stiffness of the structure. The control of the loading and the measurement of the deflection and temperature conditions allow to calculate the real stiffness of the structure. Considering bridge structures, the deflection due to shear is generally small compared to the deflection due to bending. Furthermore, the temperature conditions have a global influence on the deformation behaviour of bridge structures and will not impact the deflection line locally.

$$w(x) = \int_0^l \frac{M}{EI} \overline{M} dx + \int_0^l \frac{V}{GA_V} \overline{V} dx + \int_0^l \frac{N}{EA} \overline{N} dx + \int_0^l \frac{T_u - T_o}{h} \alpha_T \overline{M} dx + \int_0^l T_S \alpha_T \overline{N} dx \quad (1)$$

w – Deflection,
M – Bending moment,
V – Shear force,
N – Axial force,
EI – Bending stiffness,
GA_V – Shear stiffness,
EA – Axial stiffness,
T_u – Temperature at lower edge,
T_o – Temperature at upper edge,
α_T – Coefficient of thermal expansion,
T_S – Temperature along the longitudinal axis.

The first derivation of the deflection line (Eq. (2)) enables the calculation of the inclination angle:

$$w'(x) = \frac{\delta w(x)}{\delta(x)} = \varphi(x) \quad (2)$$

φ – inclination angle

The value of the inclination continually changes along the longitudinal axis of the structure if there is no change in stiffness. The second derivation of the deflection line allows computing the curvature, which depends on the stiffness and the bending moment of the considered static system (Eq. (3)). The term in the denominator $\sqrt{[1 + w'(x)^2]^3}$ amounts to almost 1 for small deflection values. So, for the given application, a real bridge loaded at serviceability limit state, the prerequisite to make this simplification is fulfilled. On the other side, the curvature can also be calculated from the relation of the strain states of reinforcement and concrete (Eq. (4)).

$$k(x) = \frac{-w''(x)}{\sqrt{[1 + w'(x)^2]^3}} = \frac{M(x)}{EI(x)} \quad (3)$$

$$k(x) = -w''(x) = \frac{1}{r} = \frac{-\varepsilon_{c2}}{x} = \frac{\varepsilon_{s1} - \varepsilon_{c2}}{d} = \frac{\varepsilon_{s1}}{d-x} = \frac{M(x)}{EI(x)} \quad (4)$$

κ – Curvature,
ε_{c2} – Compressive strain of concrete,
ε_{s1} – Tensile strain of steel reinforcement,
h – Height of the cross-section,
r – Curvature radius,
x – Height of the concrete compression zone,
d – Static height.

2.2. The principles of the DAD-method

The deflection curve of a structure, which has been measured during a static loading test, depends on the stiffness of the structure. Unfortunately, this curve is also influenced by global effects such as the boundary conditions at support or the temperature conditions. Especially the condition of non-structural layers such as the asphalt layer could lead to a huge impact: with changing thermal conditions, the stiffness of the asphalt layer changes considerably and thus, influences the global deflection behaviour of the structure. In case of an assessment of the structure by comparison to reference measurements of the initially intact structures no clear analysis can be done. To avoid this problem of global effects, the analysis of the structure must be done locally by identifying discontinuities. Therefore, the DAD-method has been developed. It provides the capability to detect and localise a stiffness reducing damage without the need of an exact reference measurement of the structure in undamaged condition. However, a theoretical model of the structure is needed as reference system. The theoretical background of the DAD-method will be explained in the following by using an example: a finite element calculation of a 30 m long bridge using the software SOFiSTiK (Figs. 1 and 2). The structure is modelled as a single span bridge and loaded at the middle of the span. The chosen loading of 1200 kN corresponds to two trucks each weighting 600 kN. In this case a linear calculation is performed with concrete type C40/50. The FE-mesh corresponds to an equivalent density of the measurement points of the deflection line (Fig. 3). The curves of the deflection, the inclination angle and the curvature at an undamaged state (blue line) and at a damaged state (red line) are represented in Fig. 1. When applying the method on real structures, the curves of the damaged state would correspond to experimentally measured values. Here, the damage is simulated by a reduction of the bending stiffness of the section at a quarter of the beam length by about 60% what is realised within the model by a reduction of the Young's Modulus of 60%. The large stiffness reduction is chosen in order to clearly present the method. However, an example with very small damage will be presented in Section 6.1. As a result of the locally reduced bending stiffness, a higher deflection and a re-localisation of the maximum deflection towards the location of damage can be identified. However, the maximum deflection is not located at the exact position of the damage. In order to detect a local discontinuity, the measured curves of the deflection line, the inclination angle and the curvature are compared to the expected theoretical curves. The damaged system shows a kink point in the inclination angle curve, respectively a considerable peak in the curvature curve (Fig. 1). However, a precise interpretation and an accurate reproduction of these curves requires a high number of measurement points. Furthermore, the detection is influenced by the degree of damage. The curves of the deflection, the inclination angle and the curvature do not allow to detect small damages associated with a small reduction of the bending stiffness. In this case, the curves for the damaged and undamaged state are approximately superposed (Section 6.1).

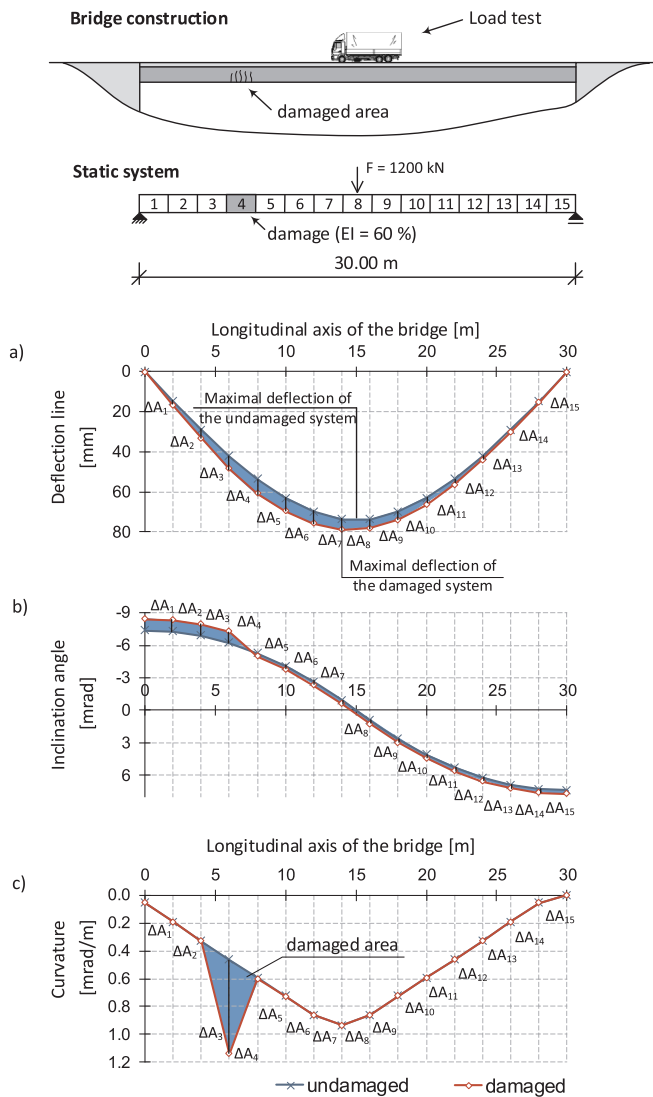


Fig. 1. Principle of DAD-Method: (a) deflection line, (b) angle of inclination and (c) curvature over the length of the single-span bridge [33].

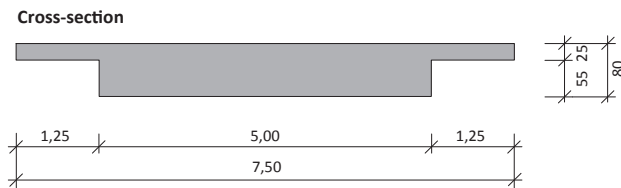


Fig. 2. Cross-section of the bridge example.

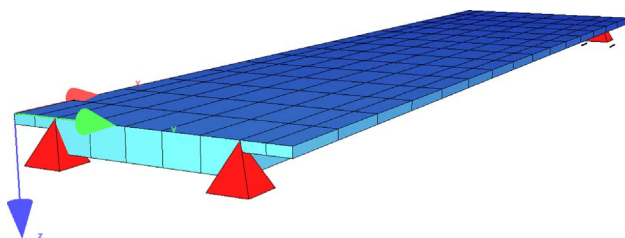


Fig. 3. FE-model of the bridge example.

For the application of the DAD-method and the calculation of the DAD-values, the area between the undamaged and damaged curves is considered (blue area in Fig. 1). The undamaged curves of a reference

system can be taken from an initial measurement or from a numerical model. In principle, the total area between the undamaged and damaged curves are divided in a number of the finite mesh units accordingly to the measurement grid (ΔA_i in Fig. 1). The DAD-value is the square of each area ΔA_i^2 divided by the square of the total area $\Sigma \Delta A_s^2$ (Eq. (5)). The squaring of the values allows to increase the damage detection sensitivity and the detection of still smallest discontinuities. The reduction of global effects such as global stiffness variations or thermal influences is enabled through the normalization of each area ΔA_i^2 by dividing it by the sum of total area differences $\Sigma \Delta A_s^2$.

The basic precondition of the method is a load-deflection experiment on a real structure with high precision measurements of the deflection line. This is particularly important if the curve of the curvature will be used to detect and localise a damage as it is calculated from the second derivative of the deflection line. Therefore, a poor recording of the measurements leads to increased noise in the curve of the curvature.

Possible stiffness irregularities of the undamaged structure are part of a measured reference system and must also be considered in case of modelling the reference system so that the localisation of discontinuities can only be attributed to damage. A case study which takes into account a planned stiffness change is carried out in Section 6.4.

In Eq. (5) the general formula of the DAD-values is given. The DAD-values are the squared values of the area difference between the curves of deflection line, inclination angle and curvature generated with the analysed structure state and the respective curves generated with the undamaged reference system. The explicit description of the equation for deflection, inclination and curvature are given in Eqs. (9)–(11). All individual steps for the calculation of the different area values are explained in detail in the following.

$$DAD_i(x) = \frac{[\int_{i-1}^i f_{d,i}(x)dx - \int_{i-1}^i f_{t,i}(x)dx]^2}{\sum_{i=1}^n [\int_{i-1}^i f_{d,i}(x)dx - \int_{i-1}^i f_{t,i}(x)dx]^2} = \frac{\Delta A_i^2}{\sum_{i=1}^n \Delta A_i^2} \quad (5)$$

DAD – Deformation Area Difference value,

f_d – Function (deflection, inclination or curvature) of the damaged curve,

f_t – Function (deflection, inclination or curvature) of the theoretical, undamaged curve,

ΔA_i^2 – Area difference from section i ,

$\sum_1^n \Delta A_i^2$ – Total area difference over the whole structure length,

To calculate the curves of the inclination angle and the curvature, the coordinates of the calculated and measured deflection line are required. The derivation of the deflection line can be calculated by using the Eqs. (2) and (4) (Fig. 4).

As already mentioned, the DAD-values will be determined by the area differences between the damaged and the undamaged curves. The area difference of the deflection lines can be directly calculated by using the measured values (Fig. 4 and Eq. (6)). The area difference within the inclination angle diagram is the integral of the function the inclination function $\varphi(x)$ (Eq. (7)) respectively the area difference within curvature diagram is the integral of the curvature function $\kappa(x)$ (Eq. (8)). According to the relation between the deflection line and the inclination angle as well as according to the relation between the inclination angle and the curvature (Eqs. (2) and (4)), the respective subtraction of the integral functions are carried out. Therefore, considering the inclination angle, the area difference between the two curves is calculated by the subtraction of the integral functions between damaged and undamaged curves (Eq. (7)). Based on the relation between the deflection line and the inclination angle (Eq. (2)), it becomes evident that the area difference of the two curves of the inclination angle can be calculated directly from the deflection values (Eq. (7)). The same applies to the area difference of curvature curves. The derivation of the inclination angle corresponds to the curvature according

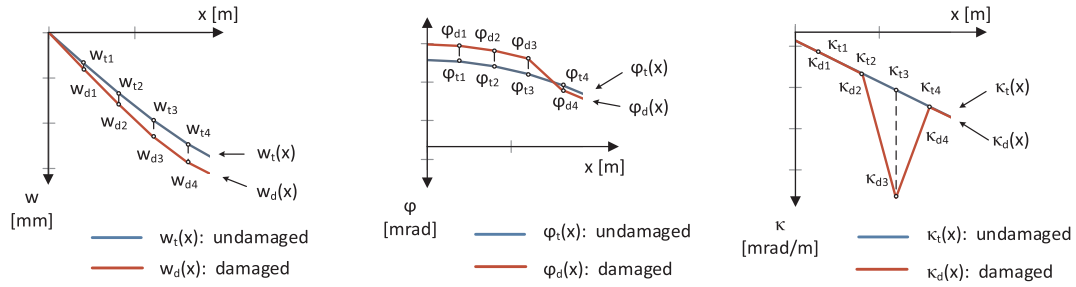


Fig. 4. Individual values from the deflection line, the inclination angle and the curvature.

to Eq. (4). Therefore, it can also be calculated directly from the inclination angle values (Eq. (8)). Thus, the double derivation of the deflection line is not needed anymore for the determination of the DAD-values from curvature, which reduces the noise related to multiple derivations.

$$\sum_{i=1}^n \Delta A_{w,i}^2 = \sum_{i=1}^n \left[\int_{x_{i-1}}^i w_{d,i}(x) dx - \int_{x_{i-1}}^i w_{t,i}(x) dx \right]^2 = \sum_{i=1}^n \left[\frac{(x_i - x_{i-1})(w_d(x_i) - w_t(x_i) - w_t(x_{i-1}) + w_d(x_{i-1}))}{2} \right]^2 \quad (6)$$

$$\sum_{i=1}^n \Delta A_{\varphi,i}^2 = \sum_{i=1}^n \left[\int_{x_{i-1}}^i \varphi_{d,i}(x) dx - \int_{x_{i-1}}^i \varphi_{t,i}(x) dx \right]^2 = \sum_{i=1}^n [w_d(x_i) - w_d(x_{i-1}) - w_t(x_i) + w_t(x_{i-1})]^2 \quad (7)$$

$$\sum_{i=1}^n \Delta A_{\kappa,i}^2 = \sum_{i=1}^n \left[\int_{x_{i-1}}^i k_{d,i}(x) dx - \int_{x_{i-1}}^i k_{t,i}(x) dx \right]^2 = \sum_{i=1}^n [\varphi_d(x_i) - \varphi_d(x_{i-1}) - \varphi_t(x_i) + \varphi_t(x_{i-1})]^2 \quad (8)$$

Then the DAD-values can be determined as described in Eq. (5) by using the Eqs. (6)–(8) for the determination of the DAD-value of the deflection line (Eq. (9)), for the determination of the DAD-value of the inclination angle (Eq. (10)) and for the determination of the DAD-value of the curvature (Eq. (11))

$$DAD_{w,i}(x) = \frac{\Delta A_{w,i}^2}{\sum_{i=1}^n \Delta A_{w,i}^2} = \frac{[w_d(x_i) - w_t(x_i) - w_t(x_{i-1}) + w_d(x_{i-1})]^2}{\sum_{i=1}^n [w_d(x_i) - w_t(x_i) - w_t(x_{i-1}) + w_d(x_{i-1})]^2} \quad (9)$$

$$DAD_{\varphi,i}(x) = \frac{\Delta A_{\varphi,i}^2}{\sum_{i=1}^n \Delta A_{\varphi,i}^2} = \frac{[w_d(x_i) - w_d(x_{i-1}) - w_t(x_i) + w_t(x_{i-1})]^2}{\sum_{i=1}^n [w_d(x_i) - w_d(x_{i-1}) - w_t(x_i) + w_t(x_{i-1})]^2} \quad (10)$$

$$DAD_{\kappa,i}(x) = \frac{\Delta A_{\kappa,i}^2}{\sum_{i=1}^n \Delta A_{\kappa,i}^2} = \frac{[\varphi_d(x_i) - \varphi_d(x_{i-1}) - \varphi_t(x_i) + \varphi_t(x_{i-1})]^2}{\sum_{i=1}^n [\varphi_d(x_i) - \varphi_d(x_{i-1}) - \varphi_t(x_i) + \varphi_t(x_{i-1})]^2} \quad (11)$$

- $DAD_{w,i}$ – Area difference value from section i resulting from the deflection line
- $\Delta A_{w,i}^2$ – Area difference of the segment i between the studied deflection lines
- $\sum_{i=1}^n \Delta A_{w,i}^2$ – Total area difference enclosed by the studied deflection lines over the whole structural length
- $DAD_{\varphi,i}$ – Area difference value from section i resulting from inclination angle
- $\Delta A_{\varphi,i}^2$ – Area difference of the segment i between the observed inclinations

- $\sum_{i=1}^n \Delta A_{\varphi,i}^2$ – Total area difference enclosed by the studied inclinations over the whole structural length
- $DAD_{\kappa,i}$ – Area difference value from section i resulting from curvature
- $\Delta A_{\kappa,i}^2$ – Area difference of the segment i between the studied curvatures
- $\sum_{i=1}^n \Delta A_{\kappa,i}^2$ – Total area difference enclosed by the studied curvatures over the whole structural length

In the following, Fig. 5 shows the calculated DAD-values for the structure given in Fig. 1. Concerning the DAD-values from the deflection curves, the curve has the tendency to move towards the damaged area (Fig. 5a). However, the most interesting information about the local damage can be taken out of the last two diagrams namely the DAD-values from inclination angle (Fig. 5b) and curvature (Fig. 5c). Particularly, the DAD-value from curvature highlights the exact position of the damage. Thus, the DAD-method allows a reliable recognition of the damages in these cases.

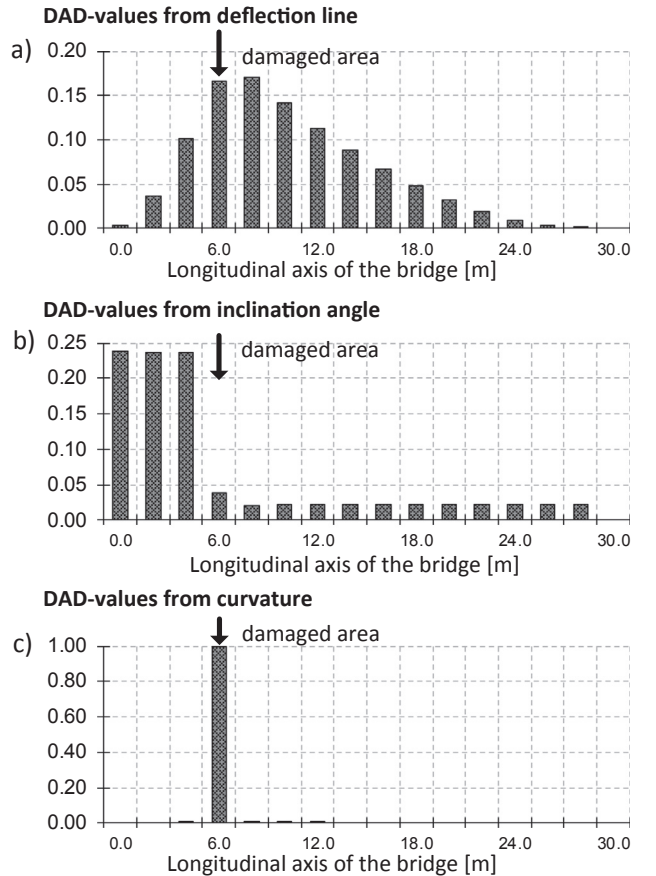


Fig. 5. DAD-values computed from the deflection line, the inclination angle and the curvature exemplified on a single-span bridge [33].

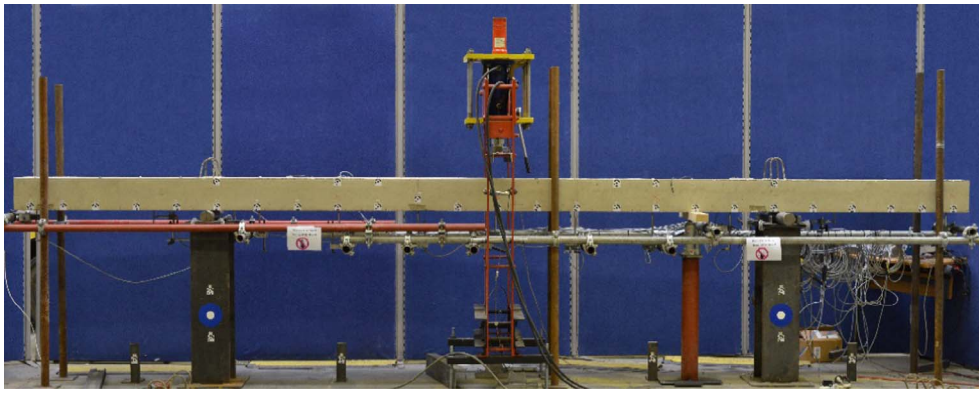


Fig. 6. Laboratory experiment with the reinforced concrete beam [33].

3. Description of the laboratory experiment

The applicability of the DAD-Method for condition assessment on a real structure was studied on an experimental beam in the laboratory. The beam consisted of reinforced concrete with a span length of 3.60 m (Fig. 6). The materials used were concrete class C40/50 and reinforcement of type B500B (Fig. 7). The loading of the beam was applied in the middle of the span length and was gradually increased. In total 9 load steps were performed, each one was carried out twice (Table 1). The duration of each loading amounts about two hours with path controlled loading. This duration was set to allow the measurement of the deflection and to control the creep effects of the reinforced beam. The plasticity of the beam was controlled by means of repetition of the load steps. The loading started at state I without cracking at 3.0 kN and is increased gradually with growth of the crack pattern (Table 1). The loading of the beam finished with the failure of concrete in the compression zone at 58.0 kN. The detection of cracks was carried out by visual inspection for each load step. The main objective of the experiment is to identify the area with reduced stiffness (crack pattern) using the DAD-values, which will be compared to the visually detected cracks. However, as already explained, the application of the DAD-method requires precise identification and localisation of damage using deflection measurements of high accuracy. Therefore, in the following several modern measurement techniques were applied and investigated regarding their accuracy and usability.

3.1. Load-deflection behaviour of the laboratory beam

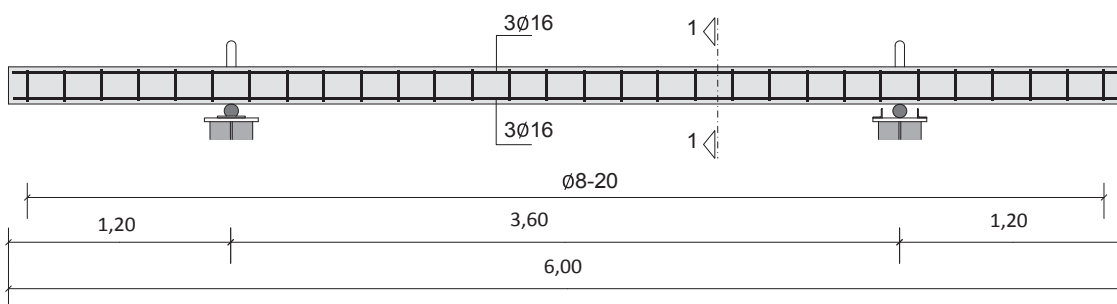
The measured load-deflection behaviour of the laboratory beam is investigated and compared to a FE-analysis (Fig. 8). The finite element model of the laboratory beam consists of a non-linear calculation also in consideration of non-linear material behaviour. The mesh size of the line-type model amounts 10 cm in longitudinal axis. With increasing loading, the reinforced concrete beam suffers different stiffness reductions due to the formation of cracks. The first load step of 3.0 kN was

Table 1
Load steps, crack pattern and measured deflection.

Load step Nr.	Load [kN]	Description of beam condition	Measured deflection [mm]
1	3.00	State I	0.50
2	5.23	First crack	1.00
3	10.0	Successive crack formation	4.50
4	15.0	Successive crack formation	7.90
5	20.0	Successive crack formation	10.6
6	30.0	Successive crack formation	17.0
7	40.0	Successive crack formation	22.6
8	50.0	Yielding of the reinforcement	28.7
9	58.0	Concrete failure in the compression zone	40.0

chosen to remain in an un-cracked state and did not generate any cracking. The second load step of 5.23 kN corresponded to the theoretical load of the expected first crack. The load steps from 10.0 kN to 40.0 kN generated a range of successive crack formation (Table 1 and Fig. 8). In Table 1 and Fig. 8, the load–deflection behaviour at state I represents the linear, undamaged condition of the beam without crack whereas the load–deflection behaviour at state II reflects the non-linear, cracked condition of the beam without contribution of concrete in tension between the cracks. Each load increase lead to the development of additional cracks, whereby the area with reduced stiffness in longitudinal direction of the beam increased. The successive cracking at state II considerably reduced the stiffness of the reinforced concrete beam. Generally, depending on the reinforcement content, the material properties and the dimensions of the cross section, the reduction of stiffness ranges between 30% and 80% [34,35]. For the experimental beam, the cracking of the concrete in the tensile zone reduced the stiffness by about 50%. This stiffness reduction has been calculated with Eq. (4) by using the curvature values from the double derivation of the deflection line and the corresponding bending moment. Due to a further increase of the load, a growth of the cracked area was observed while

Side view: reinforced concrete beam



Cross section 1-1

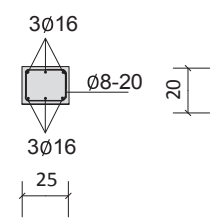


Fig. 7. Reinforcement of the laboratory beam [33].

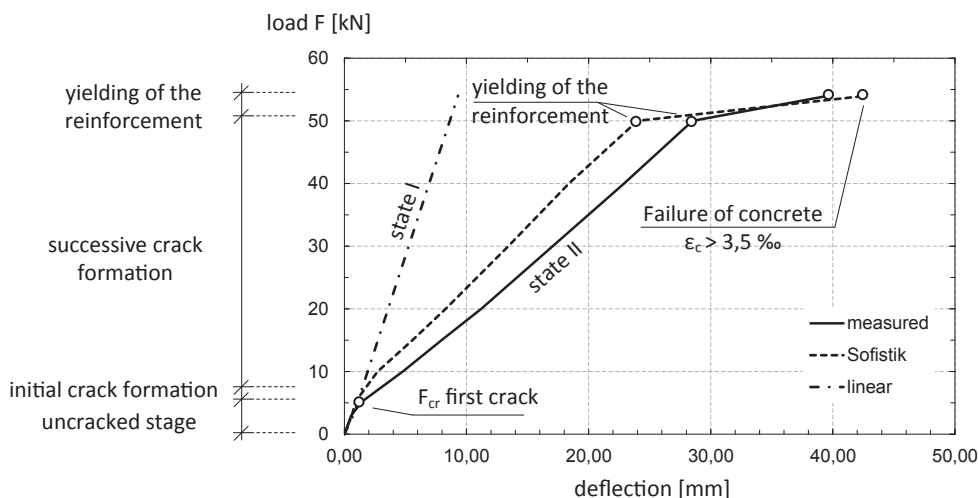


Fig. 8. Load-deflection behaviour of the laboratory beam [33].

no further stiffness reduction could be noticed. At load step of 50.0 kN, the yielding of the tensile reinforcement was reached due to which the deflection significantly increased (Fig. 8). The failure of the beam is defined here as failure of the concrete in the compression zone (Fig. 22). After the concrete failure, the remaining stiffness amounted to about 20% at mid-span.

4. Applied measurement techniques

The most decisive part to identify damaged areas of a structure using the DAD-method is the high-precision measurement of the deflection curve. Therefore, different measuring techniques have been compared in the following. The applied measurement techniques were (a) displacement sensors from HBM (Höttinger Baldwin Messtechnik) with a measuring length of 50 mm and 20 mm, (b) levelling using a high-precision digital level Leica DNA03 with bar-coded measurement staffs and (c) close-range photogrammetry using a calibrated full-frame digital SLR camera Nikon D800. The advantages of photogrammetry have already been discussed with respect to bridge inspection/monitoring. Within several research projects, photogrammetric methods were applied for deflection measurement or bridge monitoring under dynamic excitation [36–38]. Depending on the focus of the work, the considerable potential of photogrammetry for bridge monitoring has been shown. High-accuracy deflection measurement in range of 0.10 mm for laboratory condition [36] or depending on the type of cameras, temporal resolution from 1 Hz to 20 Hz can be achieved [38]. The displacement sensors needed an additional support structure, which was independent of the experimental beam (Figs. 6 and 9). The levelling took place on the top of the beam using prepared benchmarks and nearby reference benchmarks in the laboratory. While the levelling observations were analysed using standard office software, the captured imagery was analysed using Elcovision 10, a specialized photogrammetry suite. The application of special bar-coded targets, which were fitted to the side of the beam (Fig. 9), allowed the photogrammetry software to automatically find the targets in all images employed during the bundle adjustment. This produced very high precisions for the coordinates of the targets. Fig. 10 shows the point cloud from photogrammetry and the positions of the image captures as produced in Elcovision. It is noted here that prior to taking the imagery at the beam the camera was calibrated using the in-house camera calibration facility.

Following points about measurements can be summarized:

- The displacement sensors have a very high accuracy and can measure with high frequency which allows an interpretation of the performance of the structure over time. However, their in-situ

applicability on real structures is limited due to the need of a high number of sensors. Furthermore, all sensors must be able to measure the displacement relative to a rigid level. This is not always possible, especially in the case of long bridges as the accessibility from the bottom side is not necessarily given.

- Digital levelling is highly precise and easily performed but it requires additional man power and exactly defined benchmarks. The time required to perform the measurements increases depending on the number of measured points, which could lead to inaccuracies as the deflection curve usually changes due to temperature variations or due to creeping effects under loading. There is also a higher risk of measurement errors as for the displacement sensors as the accuracy could be influenced by human handling.
- The highly precise photogrammetric measurement requires pre-installed targets (Fig. 22) and shows high potential for the application to real bridge structures due to its high flexibility and accuracy. Furthermore, the flexibility of this method could be increased further with the help of modern imaging technology and from platforms such as unmanned aerial vehicles, ie. drones.

Before the load-deflection experiment was carried out, a small additional test was performed in order to compare the accuracy of the applied measurement techniques under laboratory conditions. In this experiment the beam was lifted on one side and set back to the original position. The comparison of the measurement techniques showed very high agreement and was generally at the sub-millimetre level. It was therefore decided that the application of the DAD-method would be based on the photogrammetric data. The localisation of the damage depends particularly on the density of measuring points. Furthermore, photogrammetry shows high potential for the application to the monitoring of real bridge structures.

5. Condition assessment of the beam using the DAD-Method

5.1. Detectable damage and influencing factors

The monitoring of the beam has been done in the area where cracks were expected: between the two supports on a span length of 3.60 m. As already stated all following discussions are based on the results obtained by close-range photogrammetry. The limit of the detectable damages is influenced by several factors. The data generated by each measurement technique during the experiments presented specific standard deviations. The standard deviation for the photogrammetric results amounted to about 0.17 mm under laboratory condition. Then, the loading of the beam with the hydraulic press was carried out path controlled. However, to hold the path for each load step small

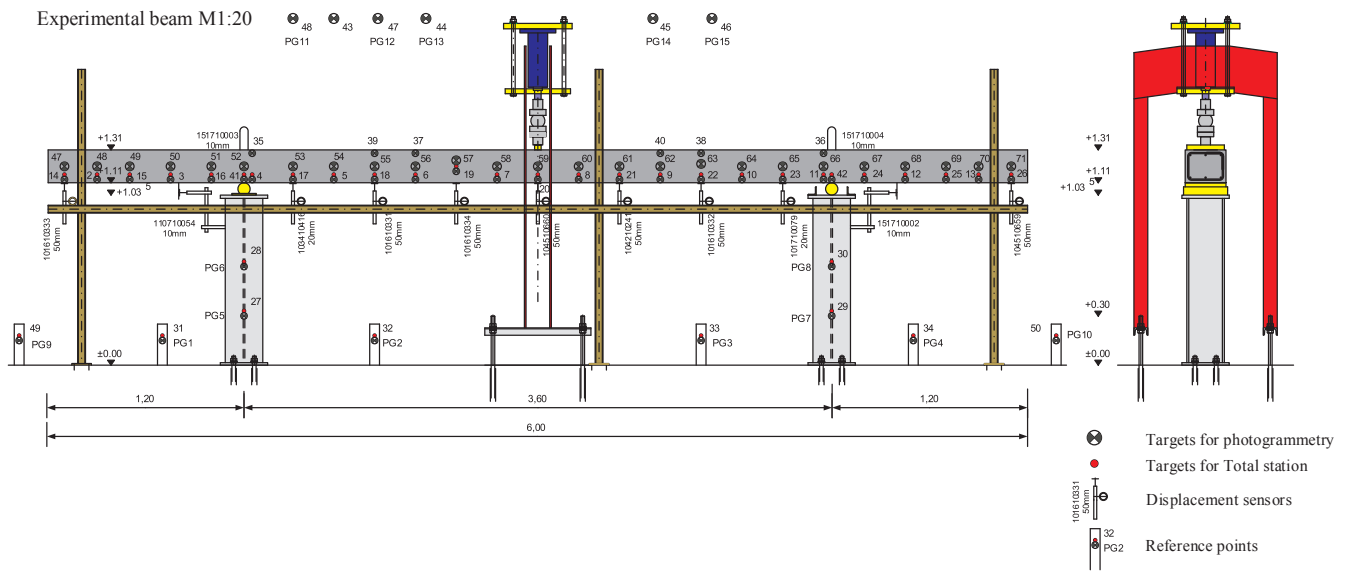


Fig. 9. Experimental beam with the targets for measurement techniques [3].

movements of the hydraulic press were to be expected due to force regulation. Therefore, the standard deviation for the path control amounted to about 0.07 mm. Furthermore, the survey of the crack pattern was done visually. Also, the density of the measuring grid had a high influence on the localisation of the cracked area. The impact of all these effects were studied on the load steps of 5.23 kN, which corresponds to 9% of the ultimate load, and 20 kN, which corresponds to 34% of the ultimate load. The measured deflection of the beam amounted to about 1.00 mm at mid-span for the load step of 5.23 kN (Table 1). Therefore, at this low loading the aforementioned effects have a high impact on the results and the deflection measurements could be influenced by about 24%. The deflection for the load step of 20 kN was 10.6 mm and here the relative influence remained much smaller with 3% (Fig. 11). Consequently, the influencing factors are proportionally higher for small deflections (Section 4). On the basis of these results the question arises how can the minimum load level, which is needed to generate accurate results, be determined. The recommendation is to remain within the serviceability limit state. For the laboratory beam with a span of 3.60 m, the maximum permitted deflection would be calculated from 1/250 according to EC 2, which leads to a deflection of 14 mm. The limitation of the maximum deflection for bridge structures under serviceability limit state is strictly regulated. The value generally varies from 1/400 to 1/1000 [39,40] of the span length. However, the large span of bridge structures would allow measurable deflection in the range of centimetres under the serviceability limit state. This proves that the method would still be applicable

for this large scale of real structures. Figs. 12 and 13 show the inclination angle and curvature for these two load steps, which are calculated from the first respectively second derivation of the deflection line.

5.2. DAD-values for the experimental beam

In the following, the results of the condition assessment of the beam in function of the different load steps will be discussed by using the DAD-Method. The DAD-values from the curvature were calculated using Eq. (11) and the results were presented in Figs. 14–21. Beneath each of these figures, the outcome of the DAD-Method from the analysis of the curvatures is compared to the depicted crack pattern of the experiment. As already mentioned, for small deflections (Fig. 14), the influence of measurement precision (Section 5.1) was proportionally high. The DAD-values for the load steps 5.23 kN (9% of ultimate load) and 10 kN (17% of ultimate load) showed no clear localisation of the cracked area. This was due to the fact that the measured deflections of 1.00 mm and 4.50 mm (Table 1) were too small and were highly influenced with up to 24% already only by the standard deviations of the measurement technique and of the hydraulic (Figs. 14 and 15). However, the localisation of the crack pattern area was successful for the load steps from 15 kN to 50 kN using the DAD-values (from Figs. 16–20). The serviceability limit state was first exceeded at 30 kN with a deflection of 17.0 mm. Thus, the applicability of the DAD-method as non-destructive method for damage localisation and

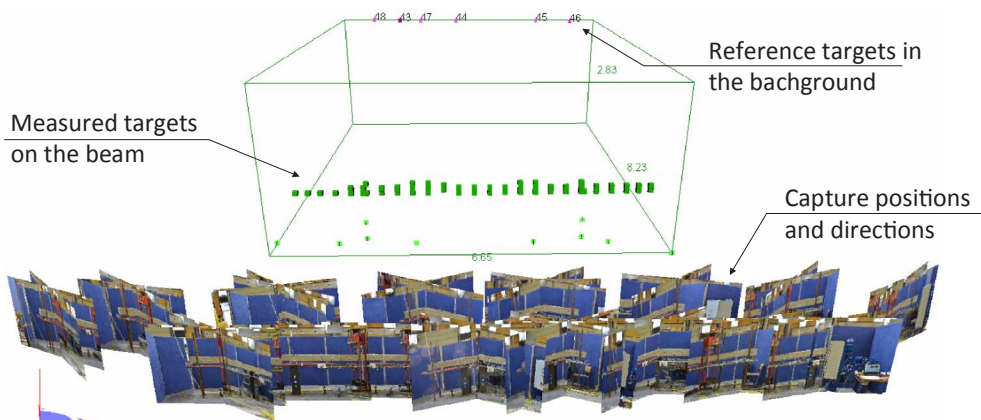


Fig. 10. Photogrammetry point cloud and capture positions.

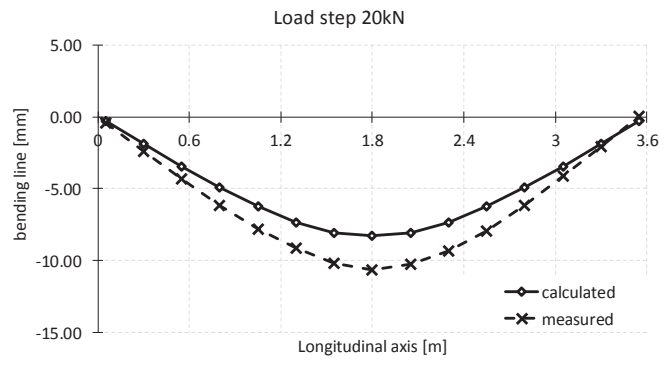
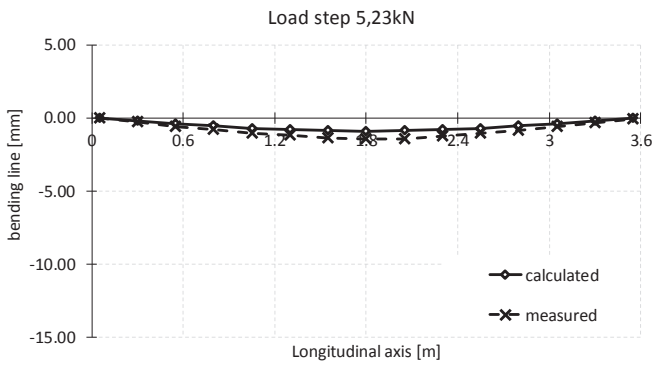


Fig. 11. Deflection values for (a) load step 5.23 kN and (b) load step 20 kN.

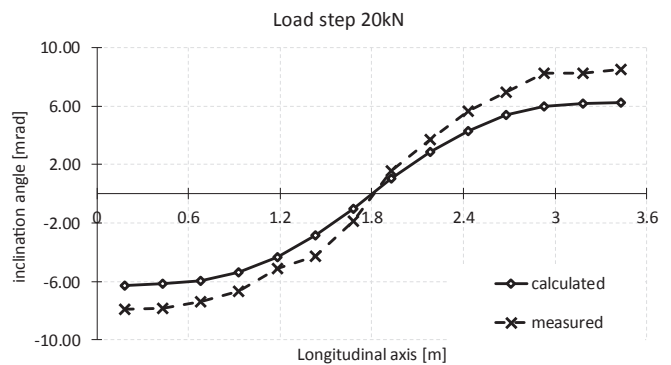
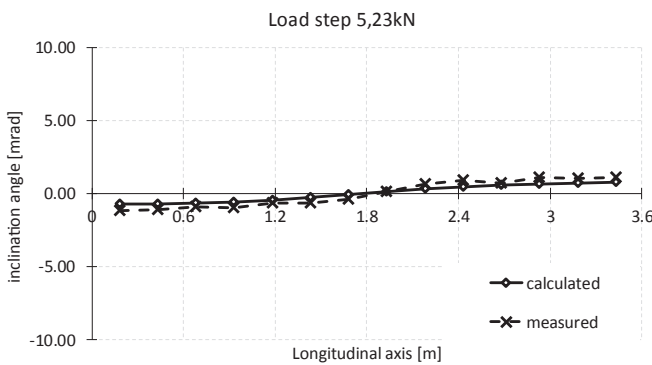


Fig. 12. Inclination angle for (a) load step 5.23 kN and (b) load step 20 kN.

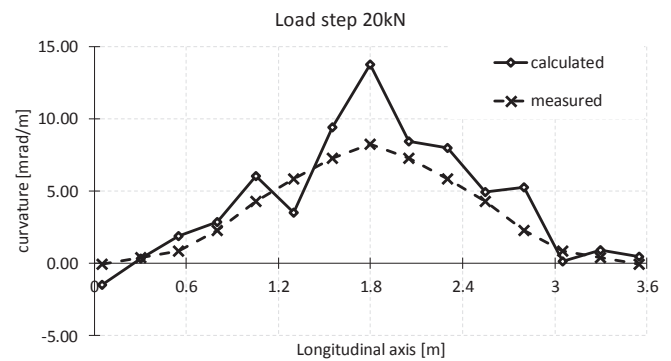
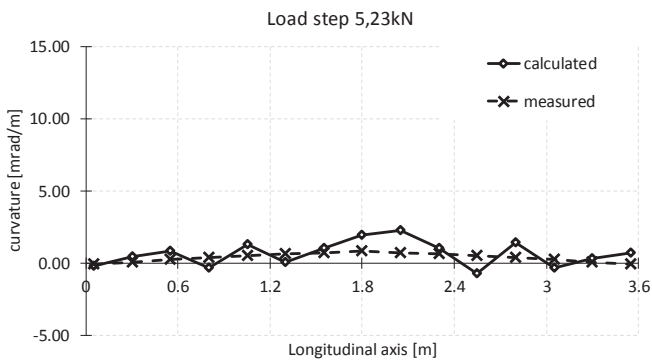


Fig. 13. Curvature for (a) load step 5.23 kN and (b) load step 20 kN.

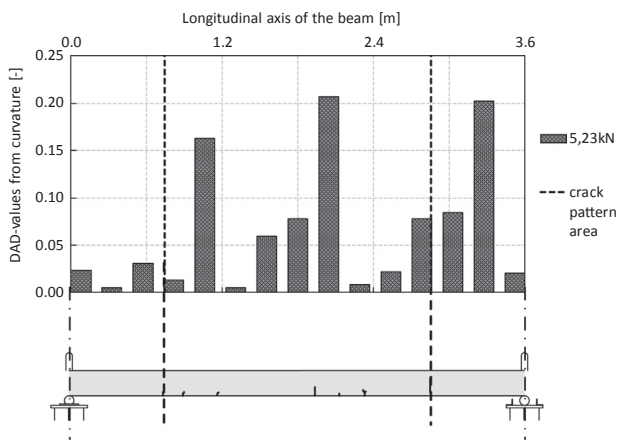


Fig. 14. DAD-values from curvature and detected crack pattern for the load step of 5.23 kN (9% of ultimate load).

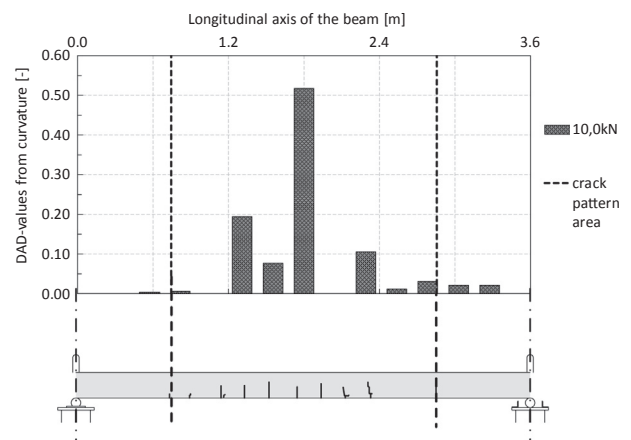


Fig. 15. DAD-values and detected crack pattern for the load step of 10.0 kN (17% of ultimate load).

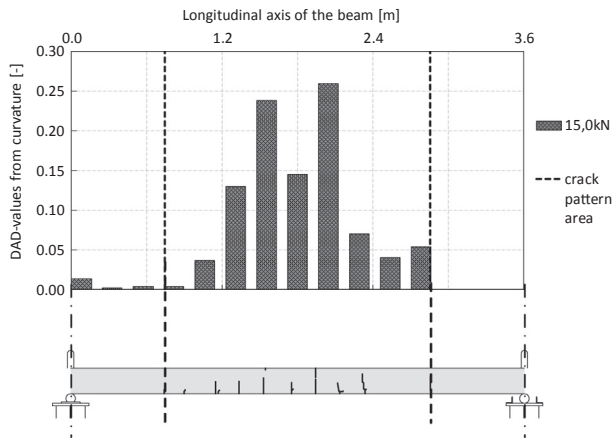


Fig. 16. DAD-values from curvature and detected crack pattern for the load step of 15.0 kN (26% of ultimate load).

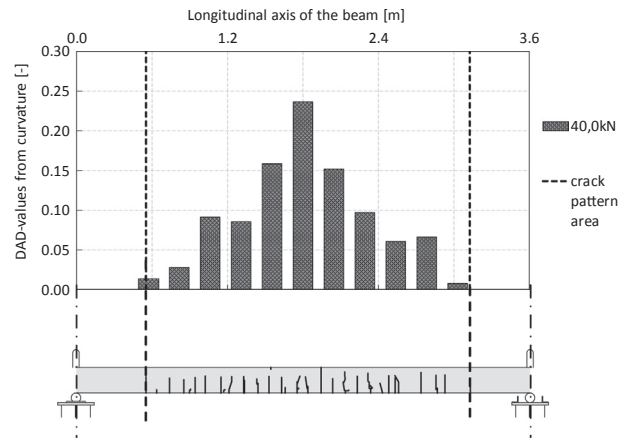


Fig. 19. DAD-values from curvature and detected crack pattern for the load step of 40.0 kN (69% of ultimate load).

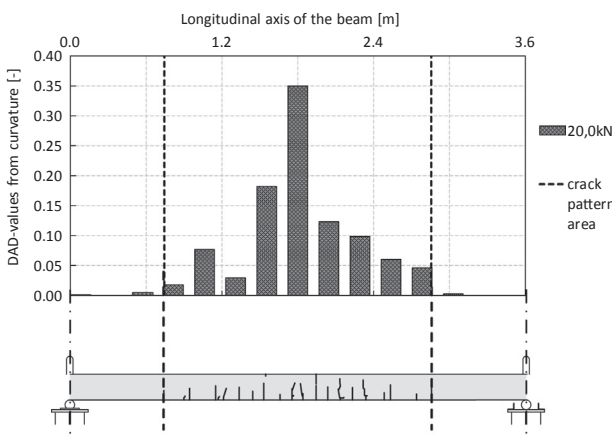


Fig. 17. DAD-values from curvature and detected crack pattern for the load step of 20.0 kN (34% of ultimate load).

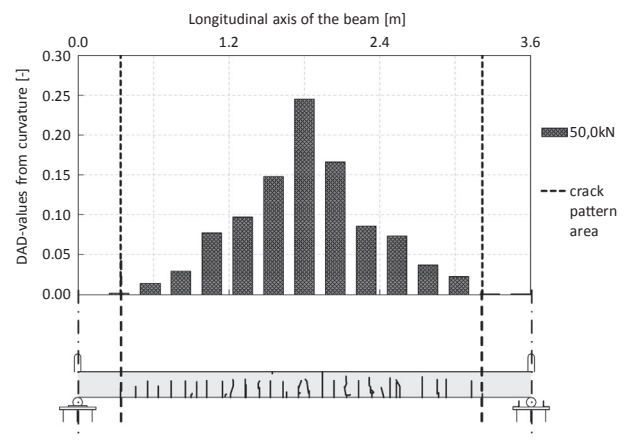


Fig. 20. DAD-values from curvature and detected crack pattern for the load step of 50.0 kN (86% of ultimate load).

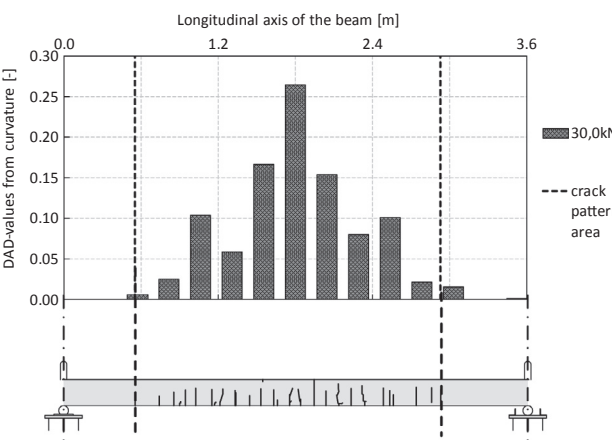


Fig. 18. DAD-values from curvature and detected crack pattern for the load step of 30.0 kN (52% of ultimate load).

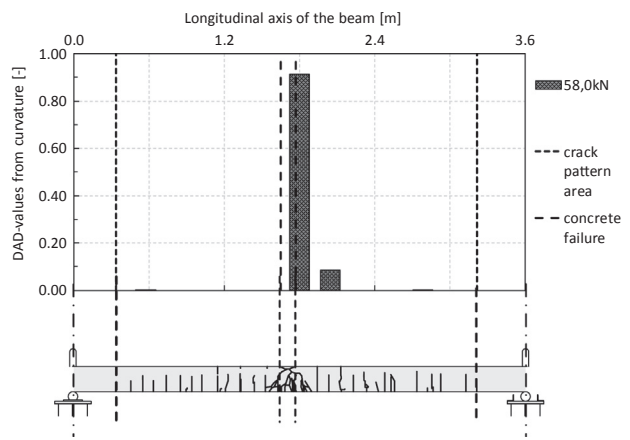


Fig. 21. DAD-values from curvature and detected crack pattern for the load step of 58.0 kN (ultimate load).

condition assessment has been proven. The localisation of the failure of the beam could also clearly be identified by using the DAD-values (Fig. 21). As a result, it can also be deduced that the accuracy of the applied measurement technique was sufficient for the damage detection of the analysed beam.

As already mentioned, the failure of the reinforced beam occurred at 58 kN due to a failure of the concrete in the compression zone (Fig. 22).

Fig. 23 shows the summary of the DAD-value diagrams for the load

steps 15 kN, 20 kN and 30 kN. These are the load steps in the area of the serviceability limit state.

6. Investigation based on theoretical calculations

As in the previous section, the applicability of the DAD-Method has been proven by the verification of an experimental test. In this section, some theoretical analysis will be carried out to further demonstrate the effectiveness of this method. Therefore, several different static systems

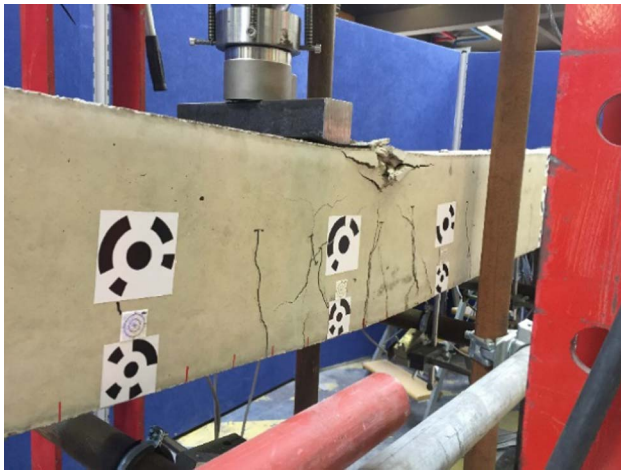


Fig. 22. Deflection failure of the beam.

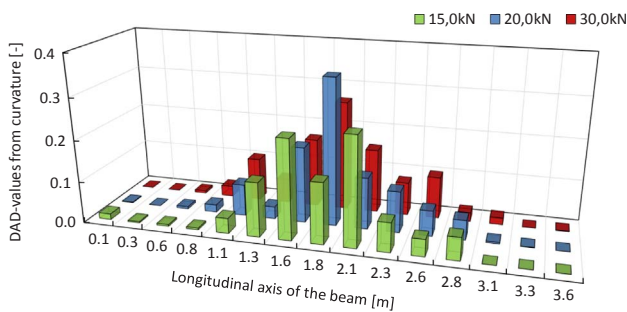


Fig. 23. DAD-values from load steps 15 kN, 20 kN and 30 kN.

will be investigated where the damage position, damage degree and load cases will be varied. For these cases, only the curvature DAD-values will be discussed.

6.1. Variation of the degree of damage

So far the analysed systems mostly showed relatively large damages with respectively large stiffness reduction. For the simulation of the example in the introduction e.g. a stiffness reduction of 60% has been chosen, whereas the stiffness reduction of the laboratory experiment due to cracking was about 50% for the crack pattern area, which increased to about 80% after yielding of the reinforcement. In the following, the bridge example is shown, but with a very small damage of 1% stiffness reduction at a quarter of the span (Fig. 24).

In Fig. 24, the first three curves show the resulting deflection, inclination and curvature lines which have been calculated by a FE-calculation for the undamaged as well as for the damaged beam. When comparing the undamaged curve to the damaged one, no visible changes in all these curves can be identified. The reduction of the bending stiffness by only 1% does not lead to any detectable change of the deflection line as well as of the curves of the inclination angle or the curvature.

Only the application of the DAD-Method permits a clear identification of the stiffness loss due to damage (Fig. 24). Thus, in this case, the DAD-Method is still valid as long as the performed measurements are precise enough.

6.2. Application of the DAD-method on different static systems and for different damage positions

The demonstration in the previous section has shown that the DAD-Method is applicable for single-span girders independent of the degree of damage as far as the measurement precision is sufficient enough.

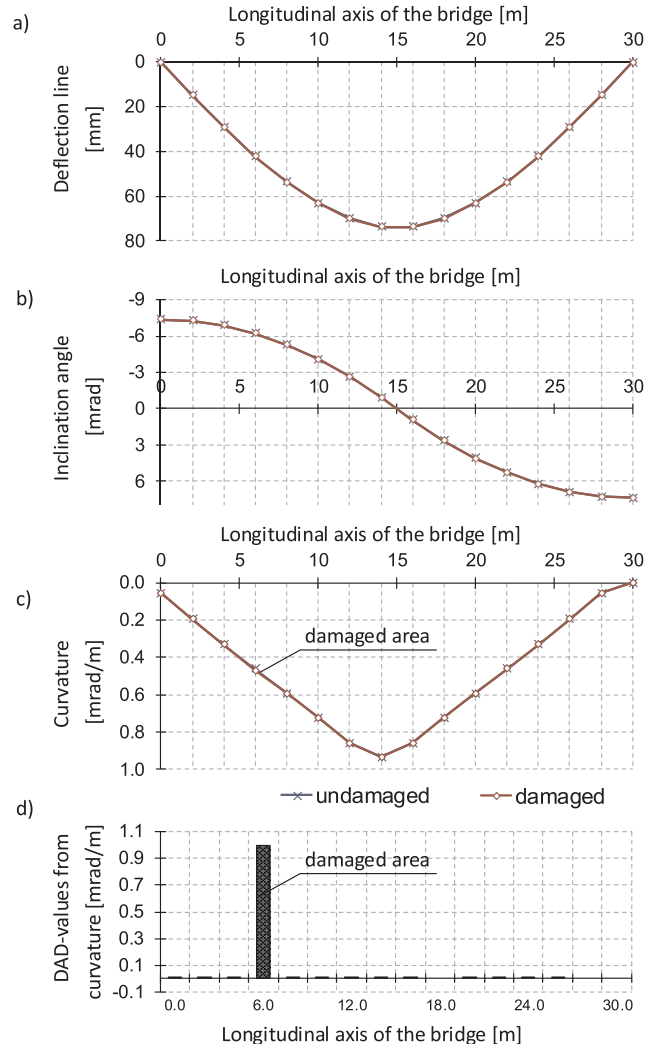
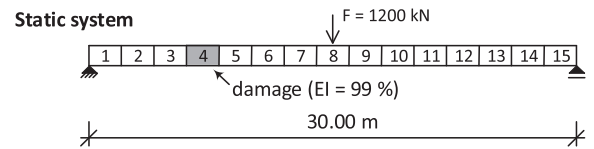


Fig. 24. Example of a beam with 1% stiffness reduction.

However, within the laboratory experiment it has been shown that the damage detection was successful in the range of the serviceability limit state. In the following, the damage assessment will be realised on two other static systems: a single-span girder with cantilever (Fig. 25) and a two-span girder (Fig. 26). In the first example, the damage is generated at the position of the left support by a 50% reduction of the bending stiffness. In the second example, the damage is located at the middle of the first span. The position of the load is insignificant for the localisation of damage using the DAD-values. In the diagram of the DAD-values, shown in Figs. 25 and 26, the localisation of damage corresponds to the generated damage in the FE model.

6.3. Effect of temperature changes on the DAD-Method

It is known that temperature changes affect the deformation behaviour of structural elements which makes it difficult to interpret the results of commonly used assessment techniques: if a data set generated during a first condition assessment test is compared to a data set of a second test after several months, the temperature conditions have changed between the tests and thus, even if the condition of the

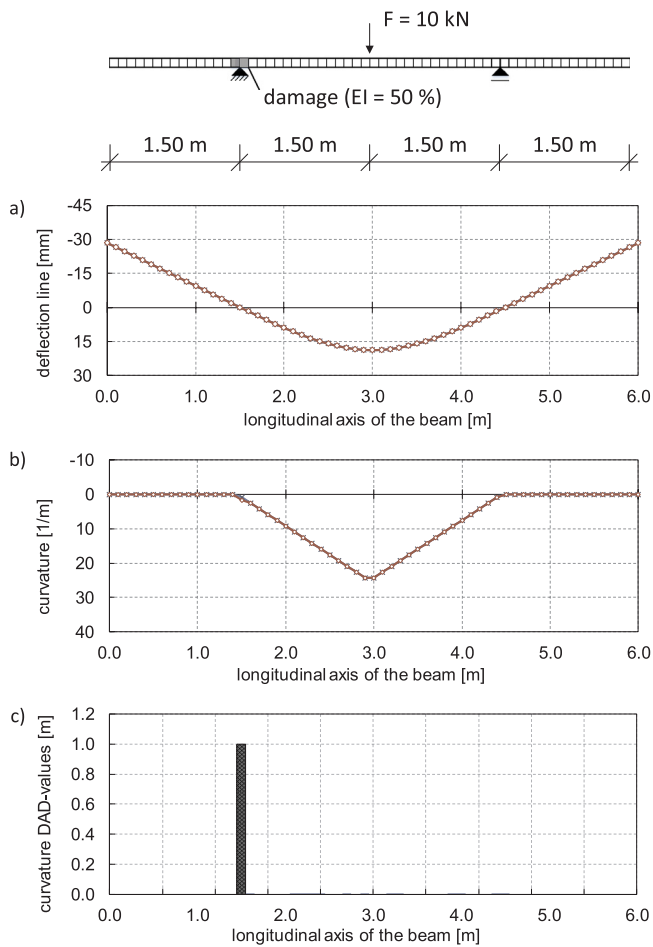


Fig. 25. Single-span beam with cantilever damaged at the first support.

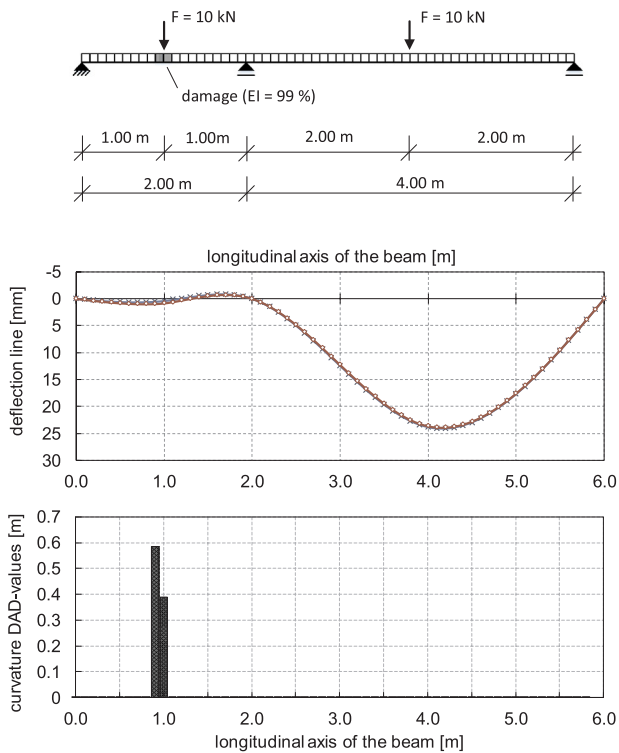


Fig. 26. Two-span beam with a local damage at the middle of the first span.

structure has not changed, the interpretation of the stiffness of the structural elements out of the measured data will not be the same for both measurements. This is particularly true for bridge structures which are still in use and covered on their top-layer by an asphalt layer. The rigidity of asphalt increases with a decrease in temperature and vice versa. Therefore, for commonly used assessment techniques, this effect should be known before any assessment of the structure can be performed by comparing the deflection line, the inclination angle or the curvature. This is difficult as the bonding behaviour between the asphalt and the load bearing structure must be known. However, as the bonding behaviour also depends on the temperature, it is nearly impossible to take into account this effect in all its dimensions.

In the following, this problem will be analysed using FE-calculation with the DAD-Method applied to a single span girder, damaged at mid-span. The damage is simulated by a 1% decrease of the bending stiffness at mid-span. Two data sets have been generated by the FE model. The first data set was calculated for a given temperature without any damage at mid-span and by further loading the section over its whole length by a temperature gradient of $\Delta T = 30^\circ\text{C}$. This temperature gradient could be the result of a heating of the top side of the structure by the sun. As the top side of a structural element is heated more than the bottom side, an upward directed deformation occurs and counteracts against the downwards directed deformation due to loading or due to damage. The calculated curvature DAD-values are depicted in Fig. 27. It shows a discontinuity in the range of the damage and also indicates the effect of the temperature gradient over the whole length of the structure. In this way it clearly identifies the damage as well as the temperature change. This identification of damage is related to the fact that the DAD-Method analyses local effects which are normalized over the whole length of the structure and it is, therefore, insensible to the global effects due a temperature variation for the detection and localisation of damage.

6.4. Case study with planned stiffness change

The previous examples as well as the experimental beam are based on simple linear elements. However, real structures or main girders of real bridges often have planned discontinuities such as surfaces curvatures, cross members, variable heights etc. The following example

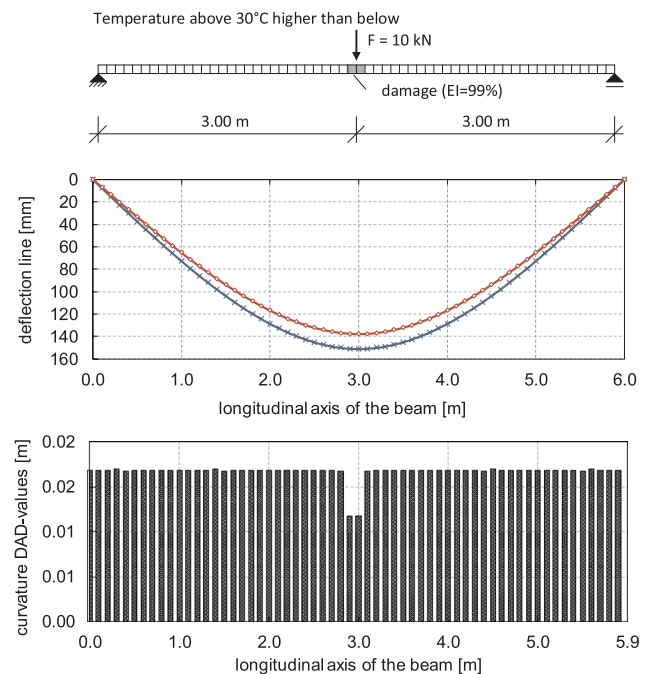


Fig. 27. Example of a girder with temperature influence and local damage.

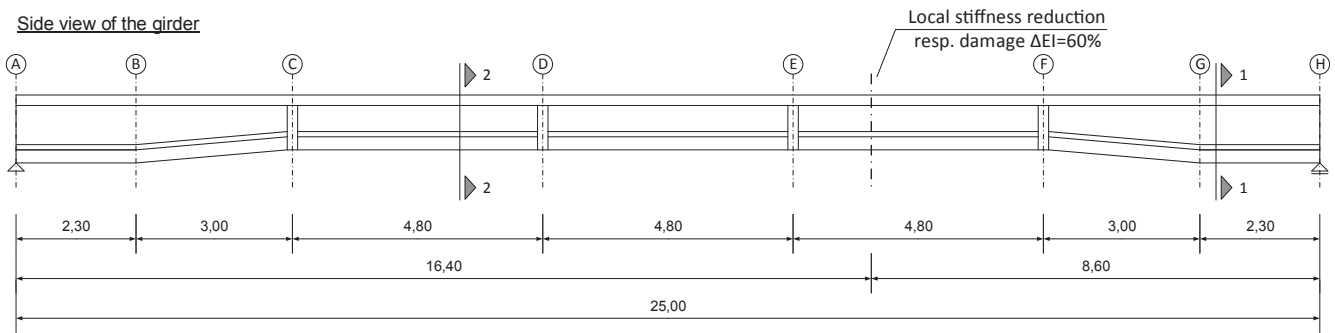
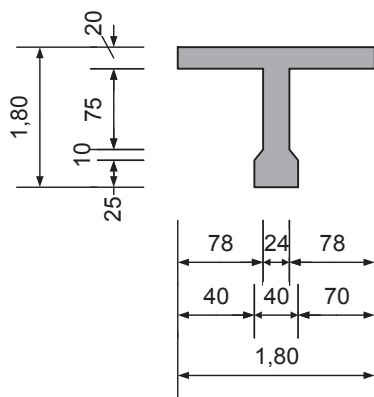


Fig. 28. Example of a girder with stiffness discontinuities at undamaged state.

Cross-section 1-1



Cross-section 2-2

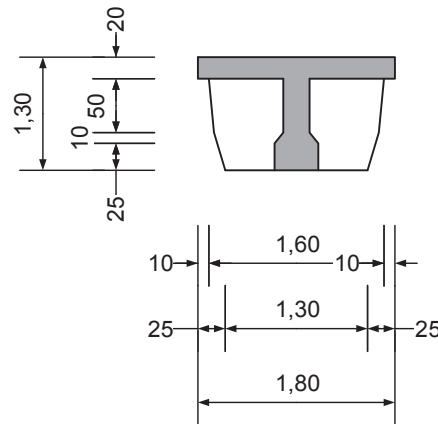


Fig. 29. Cross-sections of the girder.

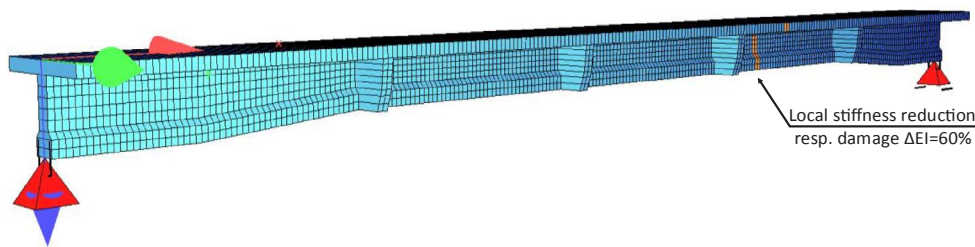


Fig. 30. Non-linear Finite element model of the girder.

shows the application of the DAD-method on a structure with variable stiffness along the beam. The example consists of a 25 m long girder (Fig. 28) of a bridge structure where the cross-section is shown in Fig. 29. The girder has a haunch next to the supports areas (between the axis B and C and between the axis F and G) as well as local transverse stiffening elements in the axis C, D, E and F (Fig. 28). The girder is loaded at mid-plan. The stiffness of the girder is reduced to 60% at 16.40 m from the left support. For the application of the DAD-method, a finite element model is created with a local stiffness reduction of 60% at this section by reducing the Young's Modulus again by this amount (Fig. 30).

Two models have been generated: a first model without damage serving as reference system and a second damaged model. In Fig. 31, the black lines indicate the results for the reference system without damage and the red lines the results for the system with local damage. The black dashed lines indicate the damage position. Considering the deflection curves respectively the inclination angles (Fig. 31(a) and (b)) the discontinuity due to damage is not clearly visible. Indeed, the curvature curve (Fig. 31(c)) shows discontinuities at the section of the structural discontinuities as well as on the section of the damage. So with the simple deflection, inclination and curvature curves the

discontinuity due to local damage could not be distinguished from the structural discontinuities. However, when considering the DAD-values in (Fig. 32), only the discontinuity due to damage becomes apparent as this constitutes the only difference between the reference system and the damaged system. The DAD-values from curvature (part c in Fig. 32) definitely localise the damage even if the structure has planned stiffness discontinuities, as these are already present in the reference system.

7. Summary

The reliable identification of damages to bridge structures is important as a precise damage evaluation allows an early and efficient bridge preservation. Until now, visual inspections or laborious long-term monitoring systems were the principal methods used for damage detections. The current state-of-the-art allows the detection of damages only at the surface or close to the surface of the structures.

In this article, a method is presented which allows the identification and localisation of stiffness reducing damages based on a single measurement of the deflection line during a load test. The deflection line along the total structure length is recorded using modern geodetic techniques to analyse the condition of the structures. The new DAD-

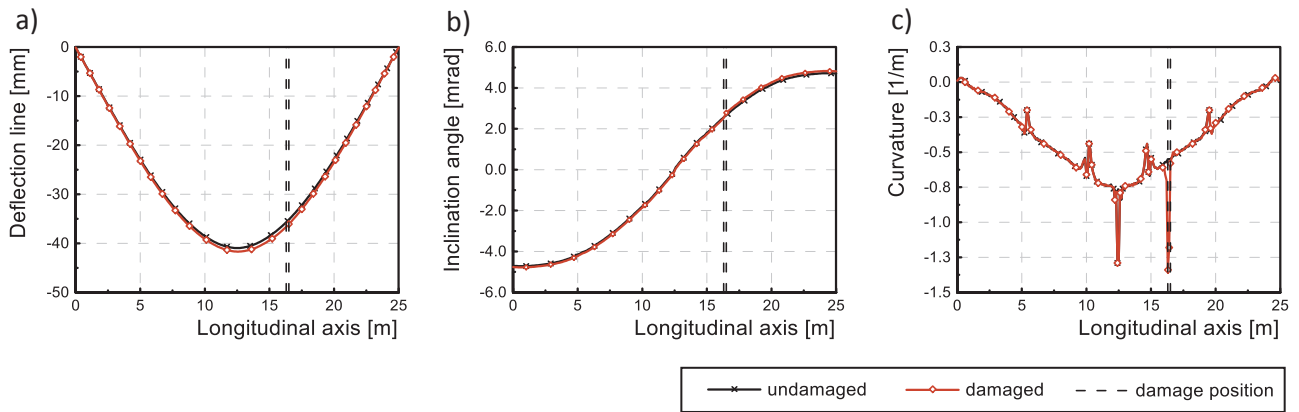


Fig. 31. Deflection line, inclination angle and curvature for the bridge girder.

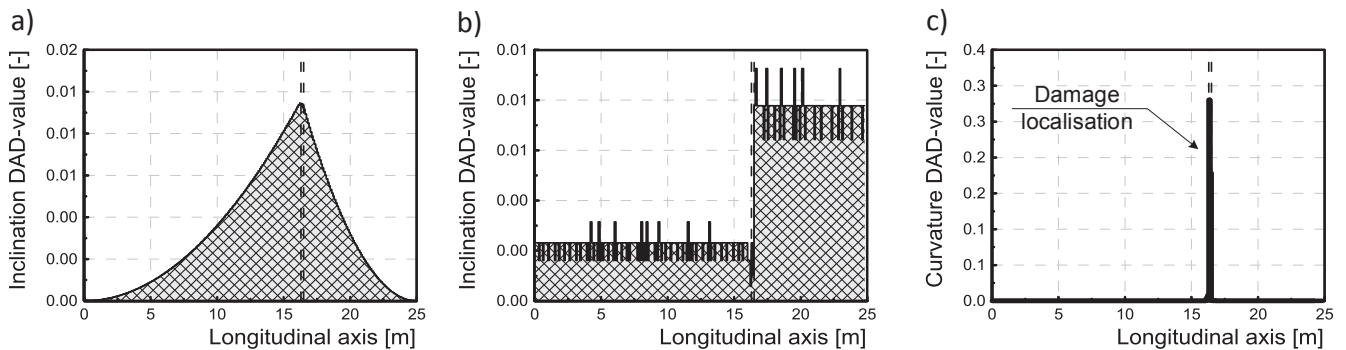


Fig. 32. DAD values from deflection line, inclination angle and curvature for the bridge girder.

Method is presented and discussed based on theoretical calculations and on a laboratory experiment. It was found that the DAD-values from the simple deflection line show changes of the system, but are not suitable and able to localise the damage. For this the evaluation of the DAD-values from the analysis of the inclination angle and the curvature were shown to be more suitable. Within the laboratory experiment, a single span reinforced concrete beam was prepared. The beam was gradually loaded at mid-span and several stiffness reductions occurred due to a progressive cracking. The main objective of the experiment was to enable the identification and the localisation of the cracked area using the deflection values. The curvature DAD-values, which were calculated by double derivation of the deflection line, allowed a reliable localisation of the crack pattern. The identification of the cracked area was even possible at the load steps under the serviceability limit state, thus the DAD-Method could be applied as non-destructive inspection method for condition assessment of bridge structures. Also for the shown examples with a single local damage, the DAD-values indicate discontinuities at the damaged areas. So, within this paper it could be demonstrated that this method is able to identify large spread damage zones as well as very local damage for different damage levels. In case of structures with planned stiffness changes along the longitudinal axis, the curve of the curvature includes several discontinuities, which has to be distinguished from local damages. A theoretical example shows an analysis of such a damaged system with the DAD-method. Again, the localisation of the damage was clearly possible despite the unsteady curvature curve. However, the essential prerequisite of the method application is the highly precise measurement of the deflection line along the bridge structure. Therefore, several modern measurement techniques such as close-range photogrammetry, levelling and displacement sensors were applied and compared within the laboratory experiment. From this photogrammetry showed high potential for the application of the DAD-Method with its standard deviation amounting

to 0.17 mm while the measured deflection amounted to 9.3 mm for the load step of 20 kN under serviceability limit state. Thus, the influence due to the accuracy of the measurement technique is relatively small with 2%.

The investigation of different static systems has shown that the method is applicable and constitutes a real supplement to commonly used condition assessment methods. Theoretical examples have shown that the method is even able to provide clear results in the case of small damages (1% stiffness reduction). The insensibility of the method in case of a temperature gradient in the structure has also been demonstrated by the means of a theoretical calculation. For the further research, the DAD-method will be extended to the analysis of slab and shell structures as these are common structural bridge element. Thus, it has been shown that the DAD-Method is nearly independent of the damage degree, no exact reference system in initial condition is needed and that it is nearly insensitive to global influences such as temperature effects. In a next step the application of the DAD-Method will be proven on real bridge structures.

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