Structural monitoring and modal properties of a real time bridge and lab tests

Veerman, R.P.
Faculty of Civil Engineering and Geosciences, Delft University of Technology, Delft, the Netherlands

Koenders, E.A.B
Faculty of Civil Engineering and Geosciences, Delft University of Technology, Delft, the Netherlands
COPPE-UFRJ Programa de Engenharia Civil, Federal University of Rio de Janeiro, Brazil

Abstract

For understanding the actual state of durability of concrete structures, it is important to know its response due to external loading. This response mainly depends on the actual structural and material properties of the structure, while loaded. Mechanical and chemical degradation are the main mechanisms that may affect these properties with time. Strain sensors can be used to measure elongations, deflections and accelerations, which monitor the combined effect of temperature, pre-stressing forces, loading forces and local strain variation, while vibration sensors can be used to measure the dynamic response of a structure. Natural frequencies and mode properties can be calculated from monitored data and show the critical dynamic response behaviour of structures. However, this response behaviour can also be calculated using structural finite element models. A full-scale sensor network is installed underneath the first span of a concrete bridge in The Netherlands. Natural frequencies and mode shapes are calculated using both monitored data and a structural finite element model. Calculation results from both methods show good agreement. Understanding the impact of chemical degradation on the mode shape properties is considered experimentally. A test-setup is currently under development in the Stevin II laboratory of Delft University of Technology and is capable of testing the response of a dynamically loaded four-point-bending beam with a chloride solution bath mounted on top of it. During dynamic excitation, the chlorides penetrate into the cracks and accelerate corrosion of the reinforcement. Strain, deflection, crackwidth, and corrosion rate will be measured during this test.

Keywords: chemical degradation, concrete bridge, corrosion, cracks, modal analysis, structural health monitoring.

1 Introduction

Traffic intensity at infrastructures has increased rapidly in the past decades. Structural and chemical degradation mechanisms have emerged and are threatening the reliability of our mobility network. It is of National interest to ensure that the reliability of infrastructural networks will remain at the highest level. The National STW perspective program called “Integral Solution for Sustainable Construction”[1] (IS2C) is developed to understand, to measure, and to predict the long term behaviour of infrastructures. Within this National research program the interactions between material degradation, monitoring and sensing, and material and structures are examined. One of the key-projects of the IS2C program is the project called “InfraWatch”, which is a joint research project between Leiden University and Delft University of Technology. Researchers from Leiden University focus on data mining and researchers of Delft University of Technology are considering the data interpretation and the matching with structural finite element models.

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[1] www.is2c.nl

In recent years, Structural Health Monitoring (SHM) systems have become more common and more frequently used in civil infrastructures. For example, the “Hollandse Brug” is a concrete highway bridge in the centre of The Netherlands where a SHM system is installed. Modal parameters of this bridge are calculated from the measured data as well as from a structural Finite Element (FE) analysis. This paper reports the explanation and the results of both calculation methods. In addition to this, the measured real-time bridge respond data is used for analysing the long term performance. However, in order to include explicit information about impact of degradation mechanisms mode properties are calculated explicitly, and compared with simulation results using FE models. Furthermore, an experimental lab-scale monitoring system is developed to register the structural and material degradation of two dynamically loaded beams which are exposed to a chloride solution. The setup and loading plan of this test-setup are discussed in this paper as well.

2 Bridge description

The Hollandse Brug is a concrete highway bridge in the Netherlands, built in the late sixties of the last century and opened for traffic in 1969. The Hollandse Brug is the main highway connection between Amsterdam and the north-east of The Netherlands and was originally built with two lanes in both directions, with a separate lane for bus and bicycle traffic. In the past few decades traffic intensity has increased dramatically, while the load bearing capacity of the bridge has reduced due to aging effects. In 2007, the Dutch National Research Institute TNO concluded that the remaining service life of the bridge had reduced dramatically, and that measures had to be taken in order to guarantee future safety for traffic passing the bridge. In 2008, based on these findings, the government decided to close the bridge for heavy traffic and only allowed normal (light weight) cars to cross the bridge. After this decision, the bridge was strengthened and extended by the two extra lanes to increase the traffic capacity of the highway. In 2009, the retrofitting works where finished, and the bridge was reopened again for all traffic.

Fig. 1 The Hollandse Brug

The Hollandse Brug originally consisted of 63 pre-stressed prefab girders distributed over seven spans of 50.55 meters each, leading to a total length of about 350 meters, and a width of 34 meters. During the reconstruction works of 2008, another 14 extra girders were added (7 on each side) in order to realize the necessary support for two extra lanes. Dilatation joints are installed between the girders in longitudinal direction to avoid the additional deformations induced by temperature or settlements to affect the internal stresses in the girders. However, this simply supported beam system is most unfavourable for the distribution of the internal bending moments in the bridge and leads to high stresses due to traffic. During reconstruction of the bridge in 2009, a full scale SHM system was installed underneath the first span of the Hollandse Brug. Sensors are located at three different locations in longitudinal direction. A total amount of 145 sensors, including 34 geophones, 91 strain gauges, and 20 temperature sensors were placed. With a measuring frequency of 100 Hertz, the sensor network generates to a total amount of data of about 5 GB per day.

Fig. 2 SHM system

3 Stochastic Subspace Identification (SSI)

Traditional identification technics can be used to analyse the output response of SHM systems for a given input load. While for a regular bridge the input load, which is the traffic, is undetermined, traditional identification technics turned out not to be the best solution to analyse monitored bridge response data. Therefore, for undetermined and uncontrollable input, identification algorithms extracting model parameters from structural response data called Operational Modal Analysis (OMA), has been developed by (Zhang, Tang and Tang, 2012). There are several identification methods for OMA that can be applied. One of the most commonly used method is the SSI method (Peeters and Roeck, 1999) (Brinker, Zhang and Anderson, 2001), which has, therefore, also been used for the identification of the mode properties for the Hollandse Brug. The SSI method is based on force equilibrium, formula (1), which is commonly applied for structural dynamics calculations. In this formula, $M \in R^{n*n}$, $C \in R^{n*n}$, $K \in R^{n*n}$ represent the mass, damping and stiffness matrices of a structure. $F(t) \in R^n$ is the vector containing the nodal forces, $u(t) \in R^n$ is the vector with nodal displacements and 't' the time.

$$M \cdot \frac{d^2}{dx^2}u + C \cdot \frac{d}{dx}u + K \cdot u := F(t) \tag{1}$$

The linear second order differential equation as given by formula (1) can be converted into matrix notation by applying simple math leading to formula (2), with (3), (4), and (5) as input.

$$\frac{d}{dt}x := A_c \cdot x(t) + B_c \cdot f(t) \tag{2}$$

$$x(t) := \begin{pmatrix} u(t) \\ \frac{d}{dt}u(t) \end{pmatrix} \tag{3}$$

$$A_c := \begin{pmatrix} 0 & I \\ -M^{-1} \cdot K & -M^{-1} \cdot C \end{pmatrix} \tag{4}$$

$$B_c := \begin{pmatrix} 0 \\ M^{-1} \end{pmatrix} \tag{5}$$

While not all degrees of freedom of a structure are measured and the measurements also include noise, a discretization in time is necessary. For this, formula (2) is converted into the formulas (6) and (7), which represent the driving forces in the SSI calculation method.

$$\chi_{k+1} := A \cdot x_k + B \cdot f_k + \omega_k \tag{6}$$

$$y_k := C \cdot \chi_k + D \cdot f_k \tag{7}$$

Noise is an unstable factor in the data analysis. While the amplitude of the higher order modes is limited, it is more complex to separate the noise from the actual response. An average value of multiple data sections can be determined to eliminate the measuring error of the higher order frequencies. After a mathematical analysis, the natural frequencies, the damping ratios, and the displacements can be calculated relatively straightforward. The mode shapes are not the primary output of the SSI calculation, but can be derived from the results by applying a secondary calculation using the sensor locations.

4 Finite Element Method (FEM) of the Hollandse Brug

A structural FEM calculation has been performed to predict the natural frequencies of the bridge. The geometry of the bridge used for the calculations is mainly based on historical design drawings made during the design stage of the Hollandse Brug, which was in the mid-sixties of the last century. The structural analysis is done with the computer software Scia Engineer (Nemetschek_Scia, 2011). The FEM model is divided into two sections where surface-elements are used for the deck and line-elements for the girders. The cross-sectional properties of the elements are calculated automatically by the software and are checked manually. Scia Engineer uses the plate theory of Mindlin-Reissner for their plate elements (Durban, Givoli and Simmonds, 2002). This theory is based on internal stress and deformation equilibrium for both bending moments and shear forces. The Mindlin-Reissner is a first order shear deformation theory with a linear displacement variation through the thickness of an element. The relative error of the FEM calculation depends on the size of the elements; smaller element sizes lead to a better approximation. In the FEM model for the Hollandse Brug, elements with an average size of 1.0 meter are used, which results in four elements between the main girders in transverse direction and 50 elements in longitudinal direction.

For natural frequency calculations, mass is an important parameter (Nemetschek_Scia, 2011). The bridge mass is calculated using the cross-sectional area and the unit mass of concrete, and an additional mass that represents heavy vehicles was applied on the structure as well. Both mass portions may influence the mode properties of the bridge. While the mass of a vehicle is relatively small compared with the mass of the bridge, the influence of the vehicle on the first order natural frequencies are limited. However, due to the non-symmetrical behaviour of the applied mass, it turned out to have a major influence on the mode shape of higher order natural frequencies and cannot be ignored in the calculation.

5 Modal properties of the Hollandse Brug

Two approaches are followed to estimate the mode parameters of the Hollandse Brug. The first approach is based on the measured data achieved from the sensor network and using the SSI method, and the second approach is based on the cross-sectional properties of the bridge using a FEM analysis. Table 1 shows the results of the natural frequencies for both calculation principles.

Table 1 – Natural frequencies of the Hollandse Brug

Mode	Frequency Order	Mode shape	Frequency (SSI calculation)	Frequency (FEM calculation)
1	1	Bending	2.51 Hz	2.63 Hz
2	1	Torsion	2.81 Hz	2.74 Hz
3	1	Bending & Torsion	5.74 Hz	6.25 Hz
4	2	Bending	10.09 Hz	8.56 Hz

5	2	Torsion	11.47 Hz	9.05 Hz
6	2	Bending & Torsion	11.99 Hz	11.68 Hz

The first three frequencies in table 1 show good agreement, while the fourth and the fifth frequency show a relatively high difference between the two approaches. Three reasons for this difference are reported in this paper. The first reason is that the three frequencies in table 1 are first order frequencies. The total mass, including the applied mass, is strongly correlated with these frequencies. However, since applied mass is much smaller than the dead weight of the bridge, the applied mass and other mass imperfections have a minor influence on the first order natural frequency. Higher order modes, however, are more sensitive to these mass changes and distributions of the total mass. Non-equally distributed mass may cause larger differences in the higher order frequencies, which are represented by the 4th, 5th and 6th frequencies in table 1. The second reason is that, the uncertainties in the behaviour of the connections and the exact mass of the cross girders, will increase the uncertainties in the calculation as reflected by the differences in the higher order frequencies. The last reason is that the higher order frequencies are more difficult to measure, while the amplitude is small and the amount of measurement points per cycle is limited. These arguments explain the higher error for the second order natural frequencies.

Another method to compare the results of different calculation methods is the mode shape of the frequencies. Figure 3 shows the mode shape of the first order bending frequency for both the SSI and the FEM calculation methods. Figure 4 shows the mode shape of the second order bending frequency. In contrast to the error in the second order natural frequency, both mode shapes show good agreement.

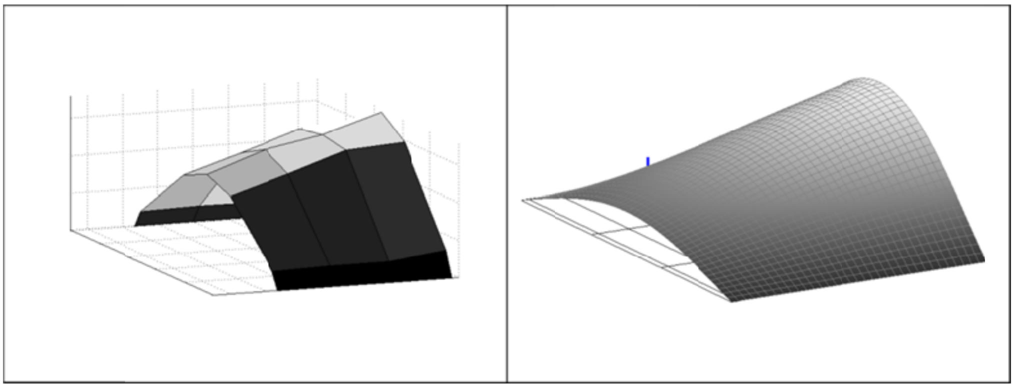

Fig. 3 First order bending frequency for SSI (left) and FEM (right)

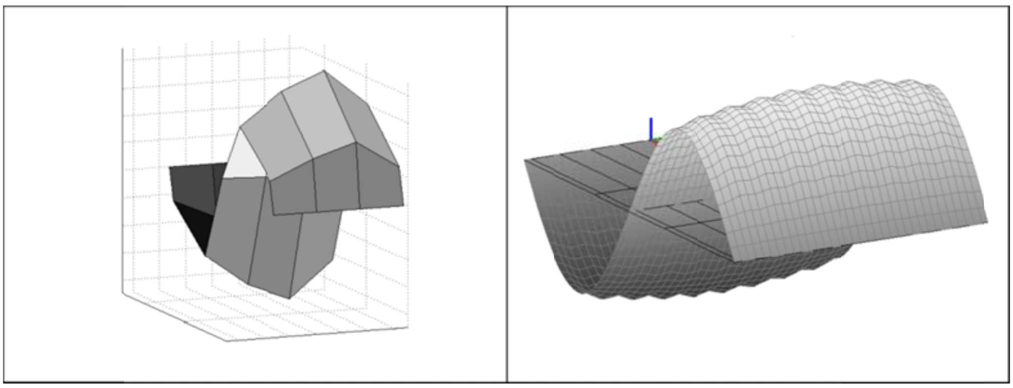

Fig. 4 Second order bending frequency for SSI (left) and FEM (right)

6 Understanding the impact of degradation

The data received from the Hollandse Brug monitoring system is not directly suitable to understand the impact of degradation on the structural performance. The first explanation is the complexity of the bridge; the bridge contains prestressed girders in both longitudinal and transverse direction. The force distribution between all girders causes large variations in parameters and uncertainties in the interpretation of the data, what leads to a reduced reliability of the calculation results. Second, the data is based on measurements under uncontrolled situations. The bridge is exposed to climate conditions, such as temperature, wind, and rain, which cause noise in the measurements. For a proper calculation, all noise should be neglected from the data, which is a complex process to conduct. The third reason is that the Hollandse Brug was recently renovated and is in good condition. According to some preliminary durability calculations, it may take many years before any significant change in bridge response would be gaugeable because of the impact of degradation. In addition, the instruments installed on the Hollandse Brug do not explicitly account for the impact degradation has on the structural and dynamic response of a bridge. Therefore, in order to develop more knowledge on this issue, a test setup has been developed in the Stevin II laboratory of Delft University of Technology. The test setup contains a beam which is dynamically loaded according to a four-point-bending test. Two beams are loaded simultaneously under the same conditions (fig. 5). To generate corrosion of the reinforcement, a bath with a chloride solution will be mounted on top of one of these beams, while the other beam is exposed to a bath with taped water. Due to a wet/dry cycle both water and oxide will penetrate into the beam, and since also the chloride solution penetrates into the concrete beam through the cracks, corrosion of the reinforcement bars will accelerate. The dynamic load results in opening and closing of the cracks, which will increase the rate of chloride penetration in comparison with a statically loaded situation. The hypothesis is that the corrosion rate of a dynamically loaded test develops in a shorter time span than it develops under statically loaded conditions.

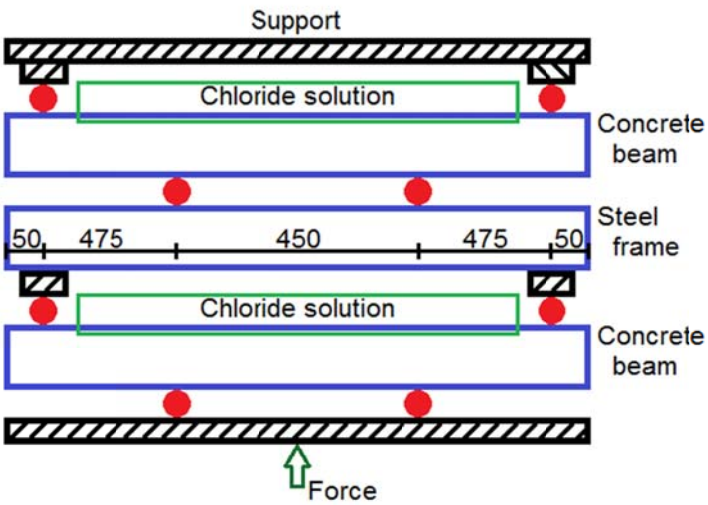

Fig. 5 Draft of the load setup

Beam layout

The beams which are used for the lab-test are the same beams as used in another IS2C project (www.is2c.nl), but, on which the corrosion rate of a loaded four-point-bending test is measured statically. The main purpose of using the same beams is the possibility of comparing the test results measured under both static and dynamic loading conditions. While using the same beam samples, the test results can be used in both projects and help to develop a more thorough conclusion about the mechanism that affects the corrosion rate and the mode properties. The properties of the beam layout are listed in table 2.

Table 2 – Properties beam layout

Concrete		Steel	
Quality	C20/25	Quality	FeB 500
Width sample	100 mm	Diameter	12 mm
Height sample	150 mm	Reinforcement ratio	0.76 %
Length sample	1500 mm	Cover	30 mm
Load distances	See fig. 5		

Measured data

The test setup is developed to measure the effect of chemical degradation on the structural performance, to analyse the monitored data, and to evaluate the overall performance of the beam monitoring system. For these purposes it is necessary to measure both the degradation rate and the structural response. Degradation is measured from the amount of cracks, the crackwidth, and the corrosion rate while the structural response is measured from the deflection of the beam. The acceleration of the beam can be calculated form the second derivative of the deflection with time. The stiffness of the beam can be calculated with formula (1) using measurement data. Table 3 shows a list of all measurement activities.

Table 3 – Measurements

Deflection by Linear Variable Differential Transformer (LVDT)
Acceleration by derivate the deflection data
Crackwidth by Linear Variable Differential Transformer (LVDT)
Corrosion rate by Half-Cell Potential (HCP)

Load specimen

Four beams with the same concrete composition will be cast from one batch and harden under the same controlled conditions. Due to this, the material properties of all four beams are statistically comparable. The first beam will be loaded until failure, and will be used as a reference for the other beams, and also to determine the magnitude of the dynamic load. As shown in fig. 5 the second and the third beam will be loaded at the same age. One of the beams will be affected by a chloride-water solution, while the other beam is only affected by water, which is the reference. The main difference between both beams is the presence of chloride ions. The fourth beam is a backup and can be used in case of unexpected failures.

Load schedule

Loading of the first beam by a static load will be done according to a prescribed loading rate which leads to a slow increase of the load with time. To minimize dynamic effects, the load increase will be 7 Newton per second. Since the beam is reinforced by one steel bar, cracking will be part of the internal force equilibrium. Yielding of the reinforcement bar is the situation that is considered as failure and determines the failure load of the beam.

As explained before, the second and the third beams will be loaded simultaneously at the same age as the first beam was loaded statically. A bath with a water-chloride solution will be mounted on top of one of the beams to accelerate the corrosion process. After loading the first beam with a static load, the next two beams will be loaded dynamically with a frequency of 2.5 Hz and an impact of 70 per cent of the failure load of the statically loaded first beam. This load will hold on for two weeks to measure the response of the beams in the cracked stage. After these two weeks, the bath which is mounted on top of the beams will be filled with a water-chloride and water solution to introduce chemical degradation in the RC sample actively. The load schedule of the dynamically loaded beams subjected to chlorides has two stages. The first stage is a dynamic load with a constant frequency and a constant amplitude of the load, and takes one week. The second stage is a dynamic load with a random frequency and a random amplitude, and takes one month. After both stages, the test will be evaluated and decided if the load schedule is still appropriate or if it needs to be adapted for the following stages. The complete load schedule is given in table 4.

Table 4 – Load schedule

Test number	Load type	Load	Duration	Note
Test 1	Failure	$F = 7\ N\ s^{-1}$ Till failure	Till failure (appr. 30 minutes)	Note the failure load
Test 2a	Static	$F = 1\ kN\ s^{-1}$ 70% of Fy	20 minutes	Measure the static displacement
Test 2b	Dynamic	F = 70% of Fy f = 2.5 Hz	2 weeks	Reference load
Test 2c (*)	Dynamic	F = 70% of Fy f = 2.5 Hz	1 week	Apply the chloride solution
Test 2d (*)	Dynamic	F > 60% of Fy F < 80% of Fy f > 0.5 Hz f < 5.0 Hz	1 month	Measure the response, including chloride penetration

(*) Test 2c and test 2d will be repeated. After test 2d, the test will be evaluated to decide if the frequency and impact are still representative.

Testing plan

In relation to the load schedule, the period that the beams are submitted to the water-chlorides has two stages as well. During the first stage of dynamic loading with a constant frequency of 2.5 Hz and a constant amplitude of 70% of the failure load, the beam will not be submitted to a water-chloride solution. This stage is necessary to calculate the stiffness of the beam. Figure 6 shows an example of the load schedule applied during the first loading stage.

Since in daily practice, a bridge is loaded randomly load, by vehicles having a different mass and a different velocity with which they cross the bridge, a random loading pattern will also be applied to the beams. For more realistic results, the applied load will be adapted to a random load with a random frequency ranging between 0.5 Hz and 5.0 Hz, and a load that ranges between 60 and 80 % of the failure load of the statically loaded beam. Figure 7 gives an example of the irregular loading pattern foreseen during the second stage.

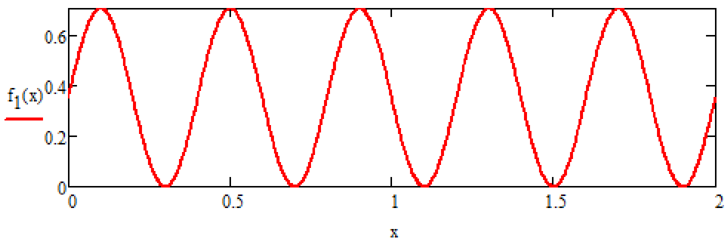

Fig. 6 Dynamic load distribution with a constant frequency and a constant impact

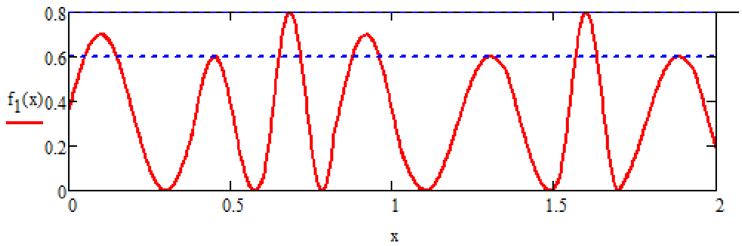

Fig. 7 Dynamic load distribution with a random frequency and a random impact

7 FEM calculation of the test samples.

In addition to the test results, the behaviour of the beams loaded in the test-setup will be predicted by structural FE calculations. While the cross-sectional properties of the test beams are known, the results of the FE calculation are comparable with the measured test results achieved from the reference beam (Test 1, 2a and 2b). Due to degradation, the response of the test beam will change with time. The structural FE calculation will be used to analyse the impact of the degradation on the structural performance of the beam. At the end of the experiment, the results of the test and the results of the structural calculation will be compared and discussed.

8 Conclusion and discussion

The modal properties of the Dutch concrete bridge called “Hollandse Brug” have been calculated using two different calculation methods. The first calculation is based on the monitored data received from an applied sensor network. The second calculation is based on a structural FEM analysis, using historic design drawings of the bridge. The results are compared and show good agreement. However, the relative error between the results of the two calculation methods is larger for the higher mode frequencies in comparison with the lower order mode frequencies. This relative error can be explained by uncertainties in both calculation methods. While the measured data is finite, the amplitudes of higher modes frequencies are small, and the amount of sensors is limited, the probability of error increases by measuring higher order frequencies. An error in the structural properties can be explained by the lack of knowledge about the material properties, and the uncertainty of the mass, the construction imperfections, and impact of these uncertainties on the properties of the girders. Both calculation methods show a larger error for the higher mode frequencies.

As discussed, the data of the Hollandse Brug is not directly suitable for understanding the impact of degradation on the structural performance. To understand this impact an experimental test-setup is currently under development at TU Delft. Results of the test-setup provide more information about the impact of the degradation on the response of the beam. The knowledge achieved from this test will be used to understand the behaviour of more realistic structures. Preliminary test results will be presented at this conference.

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