Damage assessment of a gradually damaged prestressed concrete bridge using static load tests and non-linear vibration characteristic

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Abstract

The detection of damage in bridges is mainly done by visual examination. However, defects as for instance partial rupture of a prestressing cable cannot be visually observed. It is well known that damage changes dynamic structural parameters like eigenfrequencies, modeshapes and damping. Since the sensitivity of changes in modal properties is sometimes low, this paper presents an approach that makes use of non-linear dynamic behaviour. The basic idea is that non-linear dynamic effects, i.e. the amplitude dependency of the modal parameters and the occurrence of Higher Harmonics, vary with changes in structural condition, due to failure of tendons or cracks for instance. This paper presents the experimental results of tests with a gradually damaged reinforced concrete beam and the application of this method to a prestressed concrete bridge. The results concerning damage assessment are compared to the results of static load tests as a classical and often used method to assess the state of a structure.

1 Introduction

The prime objective of dynamic based structural health monitoring and damage assessment is the determination of the modal characteristics. A lot of damage detection methods have been developed and applied in other fields of engineering like Mechanical Engineering, in aerospace sector and in Civil Engineering as well. Structural properties such as stiffness are closely related to structural vibration parameters. Most methods are based on the fact, that damage and thus change in stiffness influence the modal properties, although it is well known that the sensitivity of the modal properties is sometimes relatively low. Damage detection methods are classified as linear and non-linear methods. Linear methods are based on changes in modal properties that are a result of changes in geometry or material, but the system remains to act linear-elastic, in the damaged state as well as in the undamaged state. Non-linear methods make use of the non-linear dynamic behaviour of the structure like amplitude dependent modal properties or the occurrence of Higher Harmonics due to opening and closing cracks or non-linear material behaviour.

The results presented in this paper show that the non-linear dynamic characteristic could be another useful feature for structural health monitoring, because in some cases it might be more sensitive to damage than other indicators like changes in eigenfrequencies or modeshapes. The effects of non-linear behaviour on the Frequency Response Function are described in chapter 1.1. In chapter 1.2 it is explained by means of a simple example, why Higher Harmonic Components (HHCs) appear in the vibration response of non-linear behaving systems. Chapter 2 shows the results of a gradually damaged reinforced concrete beam. Changes in eigenfrequencies are opposed to changes in the non-linear dynamic behaviour. Chapter 3 presents the results of forced excitation tests that were performed on a progressively damaged prestressed concrete bridge in order to confirm the laboratory results and to investigate whether the observation of non-linear dynamic behaviour for damage assessment is useful in practice. Several classical damage assessment methods are applied in contrast to the investigation of the non-linear vibration characteristic.

1.1 Non-linear effects on the Frequency Response Function

Assuming linear behaviour of a harmonically excited structure, the differential equation (1) describes the vibration adequately, where *M* is the mass matrix, *C* the damping matrix, *K* the stiffness matrix and *F* the excitation force.

$$M\ddot{y}(t) + C\dot{y}(t) + Ky(t) = F(t) \tag{1}$$

The dynamic properties (eigenfrequencies, damping, mode shapes) are identified by means of the Frequency Response Function (FRF). The FRF according to equation (2) can be established by dividing the vibration response $Y(\omega)$ by the input signal $F(\omega)$ in the frequency domain.

$$H_{lk}(\omega) = \frac{Y_l(\omega)}{F_k(\omega)} \tag{2}$$

For a SDOF system with a mass *m*, a spring *k* and a damper *c* the FRF is defined:

$$H_{lk}(\omega) = \frac{Y_l(\omega)}{F_k(\omega)} = \frac{1}{(k - \omega^2 m) + i(\omega c)} \tag{3}$$

Exciting the system with its natural frequency $\omega = \omega_0$ the FRF takes maximum value. When the FRF is known or can be measured, the damping value and the eigenfrequency can directly be extracted.

In case of a linear behaving system, the FRF is independent of the excitation force and vibration amplitude. Disturbances in the structure like cracks lead to non-linear stiffness- and damping matrices. In this case the coefficients in the equation of motion (1) and the dynamic properties are dependent on the vibration amplitude, velocity and thus on the excitation force. Damage in reinforced concrete structures like friction in cracks, change in stiffness due to the alternately opening and closing of cracks under dynamic excitation or amplitude dependent material behaviour lead to particular distinct non-linear behaviour.

1.2 Higher Harmonics in the vibration response

It is well known that non-linear dynamic behaviour leads to the presence of Higher Harmonics in the response spectrum which could in theory be used to detect damage. A Higher Harmonic is a harmonic which is at a frequency that is an integer multiple of one of the modal frequencies and a result from the motion not being perfectly sinusoidal but taking the form of a distorted sinusoid at the same frequency. An often used example for the explanation of the occurrence of Higher Harmonics Components (HHCs) in the vibration response is the Duffing oscillator that characterizes a mass suspended to a spring with non-linear characteristic line:

$$m\ddot{y}(t) + d\dot{y}(t) + k_1 y(t) + k_2 y^2(t) + k_3 y^3(t) = F(t) \tag{4}$$

where m is the mass, d the damping, k_1 the coefficient for the linear term of the restoring force, k_2 and k_3 the coefficients for the non-linear terms of the restoring force and F the excitation force.

Assuming non-linear stiffness, which is symmetrical to the origin, k_2 is to zero, and neglecting damping results in the symmetric Duffing oscillator where the term $k_1y(t)+k_3y^3(t)$ represents the restoring force of the spring.

$$m\ddot{y}(t)+k_1y(t)+k_3y^3(t)=F(t) \tag{5}$$

A division by m results in

$$\ddot{y}(t)+\frac{k_1}{m}y(t)+\frac{k_3}{m}y^3(t)=\frac{F(t)}{m} \tag{6}$$

$$\ddot{y}(t)+\omega^2y(t)+\kappa y^3(t)=\hat{F}_m(t) \tag{7}$$

with

$$\omega^2=\frac{k_1}{m},\quad \kappa=\frac{k_3}{m},\quad \hat{F}_m(t)=\frac{F(t)}{m} \tag{8}$$

If the factor k_3 is positive, the stiffness takes a greater value compared to the linear case for the same displacement. This is referred to as hardening stiffness. Disturbances in the structure or cracks often lead to softening stiffness. In this case k_3 would take negative values. Considering only the restoring forces under sinusoidal excitation results in:

$$\begin{aligned}&\omega^2y(t)+\kappa y^3(t)\\&=\omega^2\sin(\Omega t+\varphi)+\kappa\sin^3(\Omega t+\varphi)\\&=\omega^2\sin(\Omega t+\varphi)+\kappa\left[\frac{1}{4}\{3\sin(\Omega t+\varphi)-\sin(3(\Omega t+\varphi))\}\right]\end{aligned} \tag{9}$$

Obviously, the response contains integer multiples of the excitation frequency. The amplitude of the Higher Harmonic Components (HHCs) in the response spectrum takes maximum values when the eigenfrequency of the system is excited.

For the use of many methods considering the occurrence of HHCs it is provided that the input is pure sinusoidal. As practice shows, this is often difficult and multiples of the excitation frequency are already contained in the input spectrum. With respect to the regular inspection of bridges it is of course difficult to apply an excitation with identical higher harmonic characteristic to make different measurements comparable. Higher Harmonics are excited at integer multiples of the excitation, even for linear systems. If one of these Higher Harmonics is near one of the system's eigenfrequencies then this frequency will resonate even if it is not being directly excited. It should be possible to minimize this effect by observing the ratio of the response spectrum and the input spectrum, provided that the input signal takes measurable values above noise in the range of the Higher Harmonics, to avoid division by zero (the Coherence Function should have adequate values in the investigated frequency range). For linear behaving structures this relation is nothing else but the Frequency Response Function that is invariant with respect to the type of input. So, exciting a mode sinusoidal (or the range of the eigenfrequency with stepped- or swept sine) and Higher Harmonics are contained in the excitation, the ratio of the input and output spectra should not have higher values compared to the conventional measured FRF in linear case (in the frequency range of the HHC as well as in the directly excited frequency range). The non-linear case creates a difference between the ratio of input and output spectra compared to the ordinary measured FRF in the range of HHCs, because the structural born non-linearity leads to additional vibration. As already mentioned, on the one hand it is necessary that the excitation force contains HHCs to avoid division by zero; on the other hand, large HHCs in the input could suppress the non-linear effects because the relation between the vibration response due to excitation and those non-linear effects that arise due to non-linear behaviour in the structure is to large.

2 Reinforced concrete beam

2.1 Damage loading

For the experimental verification a reinforced concrete beam is examined concerning its non-linear behaviour. The 6m long beam is made of concrete C40/50, stirrups 8/20, and six reinforcement bars of diameter 16mm are equally distributed over the tension and the compression zone. The beam has been loaded stepwise up to its calculated strength during a symmetrical three point bending test. The static test setup for the beam is illustrated in Figure 1. The loading sequence is presented in Table 1.

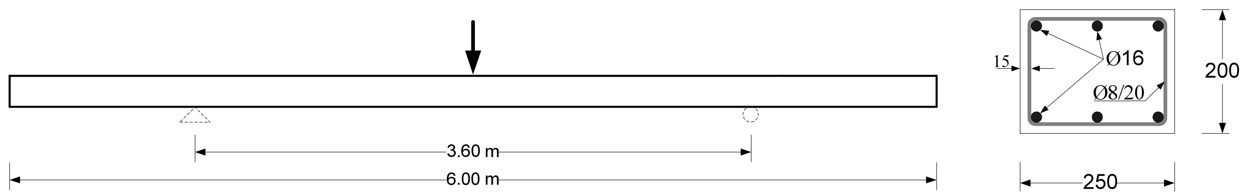

Figure 1: Static setup for the beam, cross-section, undamaged state #1

load step	load at 3 m [kN]
#1	0
#2	8
#3	16
#4	25
#5	40
#6	55

Table 1: Loading sequence

The first applied load of 8kN was around the appearance of the first bending crack. Failure was defined as the point where the beam could sustain no further increases in load (approx. 55kN). Figure 2 shows the deflections in the centre of the beam measured during the application of the static load. These results are concordant with results reported in [1]. The progressive cracking of the beam is represented in Figure 3. In undamaged state no cracks were visible. For the first applied load of 8kN two small cracks arise at mid-span. After release, these cracks close again and cannot be detected by visual inspection anymore. For increasing loading the cracked zone became wider and at a loading of 55kN the reinforcement began to yield and the concrete in the compression zone started to fail.

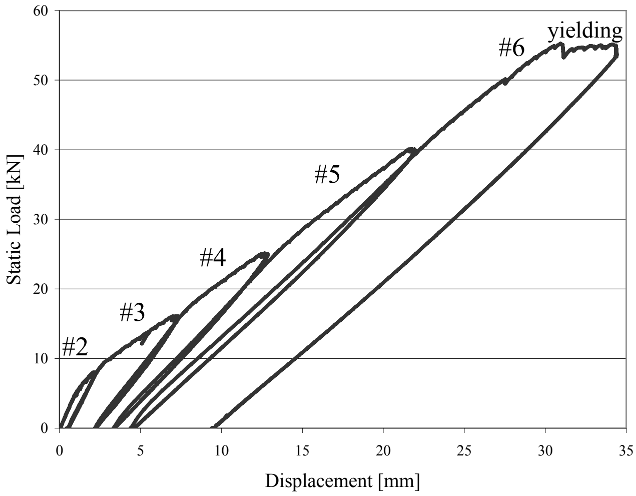

Figure 2: Force-Displacement diagram. The displacements at mid-section are shown

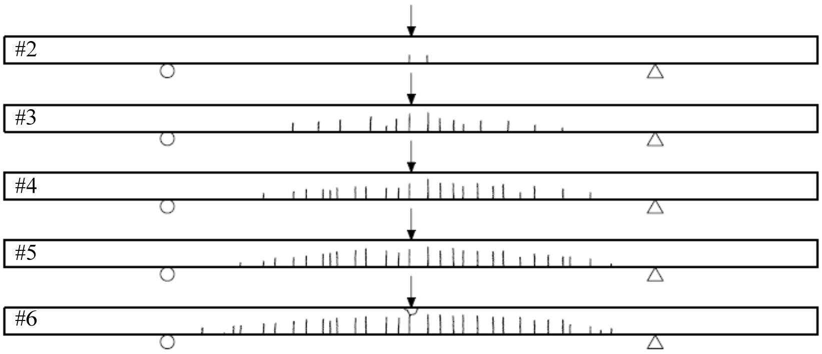

Figure 3: Progressive cracking

2.2 Dynamic Tests

For the dynamic tests the beam is suspended to flexible springs, in order to obtain a free-free setup. Initially the eigenfrequencies of the beam are measured using hammer impact. The energy associated with an individual frequency is small and non-linear effects do not appear clearly [2]. So only the quasi-linear dynamic properties of the system can be observed, when using hammer-impact method. As already mentioned in the introduction changes in eigenfrequencies are widely-used for damage assessment. Table 2 figures the first three measured eigenfrequencies (first three bending modes) for all load steps. As expected the eigenfrequencies decrease with increasing damage.

	eigenfrequency [Hz]		
load step	mode B1	mode B2	mode B3
#1	22.0	60.7	118
#2	21.5	60.0	117
#3	20.5	58.8	114
#4	20.0	57.3	113
#5	19.7	56.0	112
#6	18.8	54.9	108

Table 2: First three eigenfrequencies using hammer impact

Figure 4 plots the percentage decrease of the first three eigenfrequencies related to the undamaged state #1. The first eigenfrequency decreases about 2.2% for the first induced damage of small level. Self-evident, the first eigenfrequency is affected most. Depending on the size of the cracked zone, the second and third eigenfrequency are affected as well.

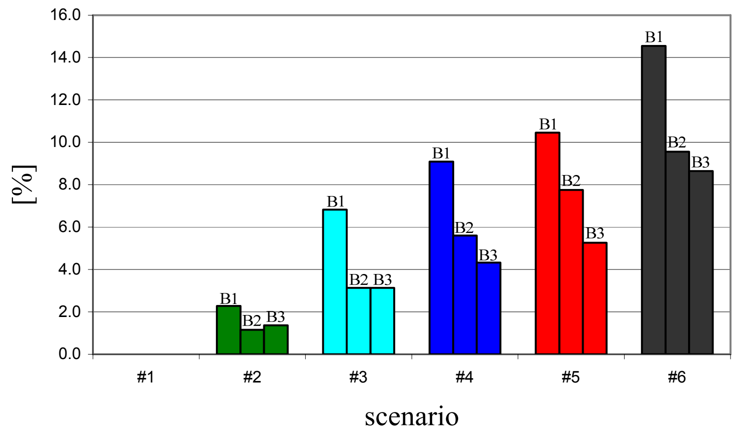

Figure 4: Percentage decrease of the first three eigenfrequencies related to the undamaged state #1 B1-mode 1, B2-mode 2, B3-mode 3

However, the change in eigenfrequencies from scenario #1 to scenario #2 is small because stiffness has been reduced only little due to small cracks. Thus, it would be interesting to know whether the non-linear vibration characteristic is more sensitive to small damage. For the investigation of the non-linear dynamic behaviour the beam has been excited harmonically by means of a swept sine excitation in a frequency range including the first three eigenfrequencies. In order to transfer as much as possible of the oscillation energy into the non-linear parts of the oscillation (to activate non-linear mechanisms like opening and closing of cracks) the beam has been excited with a swept sine using a low sweep-rate of 0.2Hz/s.

Figure 5 shows the FRFs measured for excitation forces of 5N and 100N in the frequency range of the first mode for all damage states. For the undamaged state #1 both, the eigenfrequency and the amplitude of the FRF decrease with increasing excitation force. This property can be explained by means of the non-linear hysteretic material behaviour of concrete and the existence of inevitable micro-cracks that strongly affects the damping characteristic of the beam. The percentage decrease of the first three resonance frequencies from an excitation force from 5N to 100N is shown in Figure 6. For the first resonance frequency it is about 3.2 %. The first mode is affected most. This is due to the distortion dependent non-linear material behaviour of concrete. As the deflection of mode 1 is always larger than the deflections of mode 2 and 3, the influence on the first mode is more significant.

It is obvious that the amplitude dependency of the first eigenfrequency increases for state #2 where two small cracks have been produced in the mid-span. Although the damage is small and the cracks closed again after unloading, the amplitude dependency increases from 3.2% in undamaged state to 5.6%. Again, the non-linear behaviour observed for mode 1 is affected most, because the damage is an area of high modal curvature of mode 1 (cp. Figure 7).

For state #3 the amplitude dependency of the three examined eigenfrequencies further increases. When investigating the states #4 and #5, the amplitude dependency of the first and third eigenfrequency keeps constant while the amplitude dependency of the second eigenfrequency increases. The reason for this is that the cracked zone is expanded into areas of high modal curvature of mode 2. For the last damage state #6 the amplitude dependency of the first eigenfrequency decreases again. In this state the cracks in the mid-span keep open even without static loading, frictional effects in the crack-surfaces are reduced and thus the non-linear behaviour decreases.

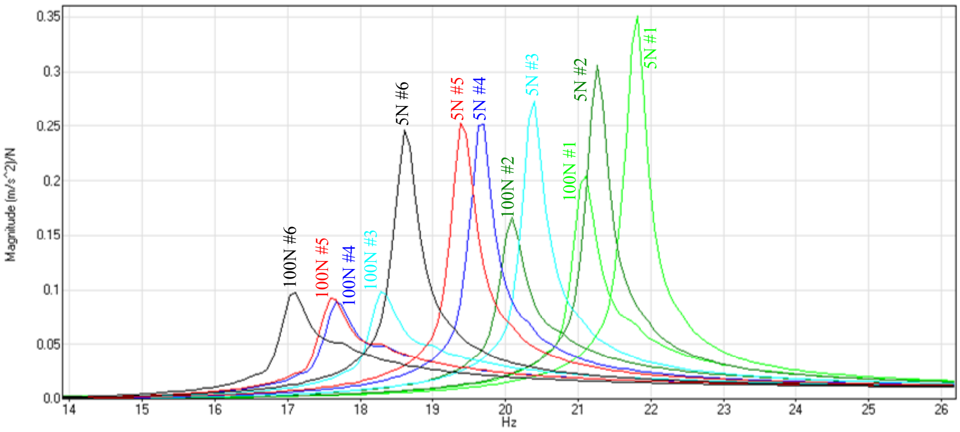

Figure 5: FRFs for excitation forces of 5N and 100N for all damage states, mode 1

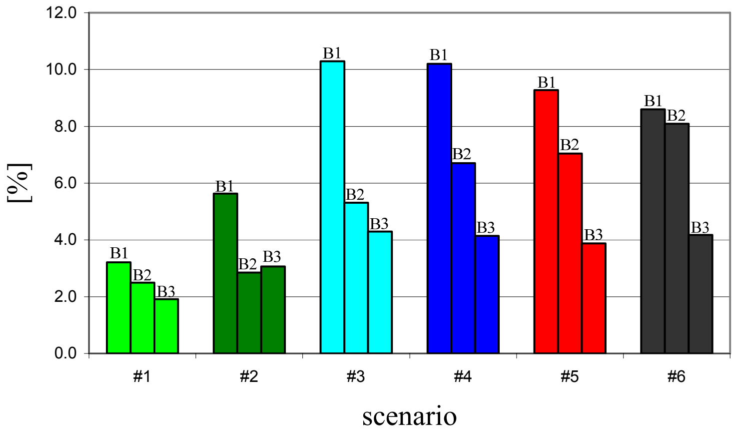

Figure 6: Percentage decrease of the first three eigenfrequencies from 5N to 100N B1-mode 1, B2-mode 2, B3-mode 3

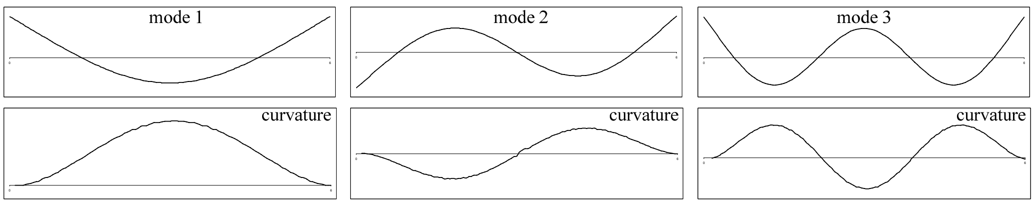

Figure 7: Modeshapes and corresponding modal curvature

For the observation of the Higher Harmonic Components (HHCs), the beam has been excited with swept sine in a small frequency range (between 14Hz and 27Hz) including the first eigenfrequency. Figure 8 shows the results of the measurements for the beam in undamaged state #1 with an excitation force of 100N. The autospectrum of the excitation force (INPUT) takes high values for the excited frequency range. As expected, the excitation force contains HHC and so does the response spectrum (OUTPUT). The horizontal line marks a free chosen limit of adequate excitation (above noise), for which the evaluation is done. The vertical lines border the frequency range of the first HHC. In this frequency range the Coherence Function has adequate values. The ratio of output and input (in the following the terminology FRF_{small} is defined as the ratio of the input spectrum by the output spectrum; small-small frequency range excited, between 14Hz and 27Hz) is plotted at the bottom of the figure. The ordinary measured FRF (also with an excitation force of 100N, exciting the frequency range between 10Hz and 120Hz) is plotted for comparison (dashed line).

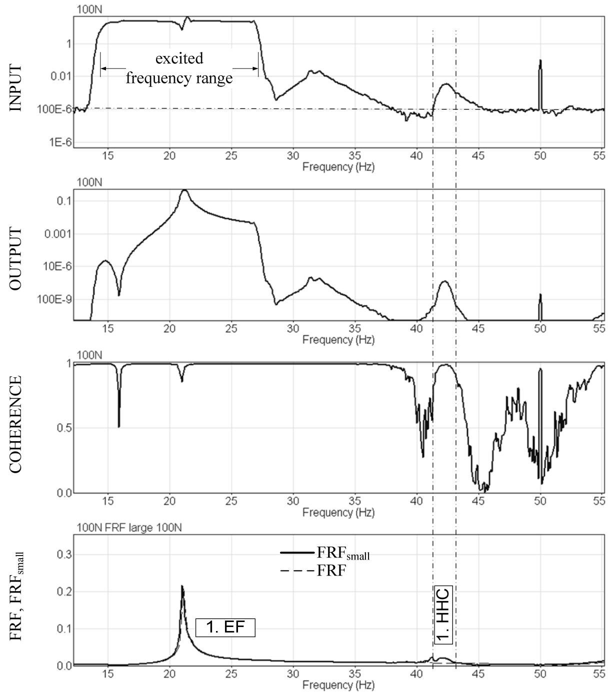

Figure 8: Input spectrum, output spectrum, Coherence, FRF and FRF_{small} for the beam in undamaged state #1, excitation force 100N

Although small Higher Harmonics of the first eigenfrequency are present in the response (OUTPUT), FRF_{small} is almost equal to the FRF, in the excited frequency range as well as in the non scheduled excited frequency range and in the range of the first HHC. The beam responds concerning the FRF. This means that the vibration response of the undamaged beam is approximately linear. Any non-linearity in material behaviour (cp. amplitude dependent eigenfrequencies) seems to be too small to create Higher Harmonics. In cracked state there are clear differences in the range of the first Higher Harmonic. The beams vibration response is larger than it would be expected by the FRF.

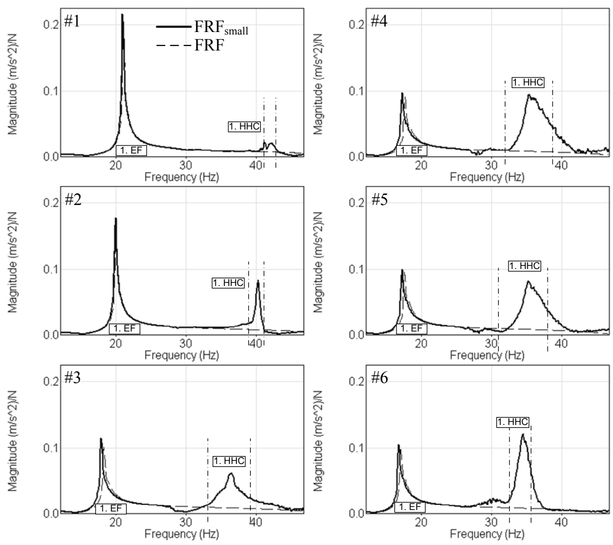

Figure 9: FRF_{small} and FRF for all damage states, excitation force 100N

Figure 9 shows the FRFs and the ratios of the input and output spectra (FRF_{small}) when exciting only the frequency range of the first eigenfrequency (FRF_{small}) for all damage states. There is a clear peak in the range of the first HHC of the first eigenfrequency for all load cases. It can be considered that these HHCs are based on non-linear behaviour of the beam vibrating in mode 1 and a result of non-linear behaviour bourn in the cracks. It is interesting that especially for small damage states (cp. damage state #2) the HHC is very clear an thus a sensitive indicator for the presence of cracks. It has to be noted, that, once the beam has been excited with an excitation force of 100N in cracked state, the first eigenfrequency decreases slightly (small difference between the FRFs in the range of the first eigenfrequency), because the bond between reinforcement and concrete has been reduced due to the dynamic load.

3 Prestressed concrete bridge

3.1 Introduction

In this chapter the results of forced excitation tests on a gradually damaged prestressed concrete bridge are presented. These tests are carried out in order to try out the use of non-linear dynamic behaviour of bridges and civil engineering structures in general for damage identification and damage assessment and to compare them with often applied conventional methods. The three span bridge was slightly skewed, curved with a radius of 300m and had a total length of 51m. The mid span had a length of 23m, the side spans were 13m and 15m long. The prestressed slab was supported on 16 elastomer bearings (Figure 10, axis A, B, D, E). The 29 tendons were arranged according to the bending moment, i.e. in the middle of the mid span the tendons were arranged in the lower part of the cross-section, above the column the tendons were arranged in the upper part of the cross-section. Figure 10 shows the dimensions and the cross-section of the bridge.

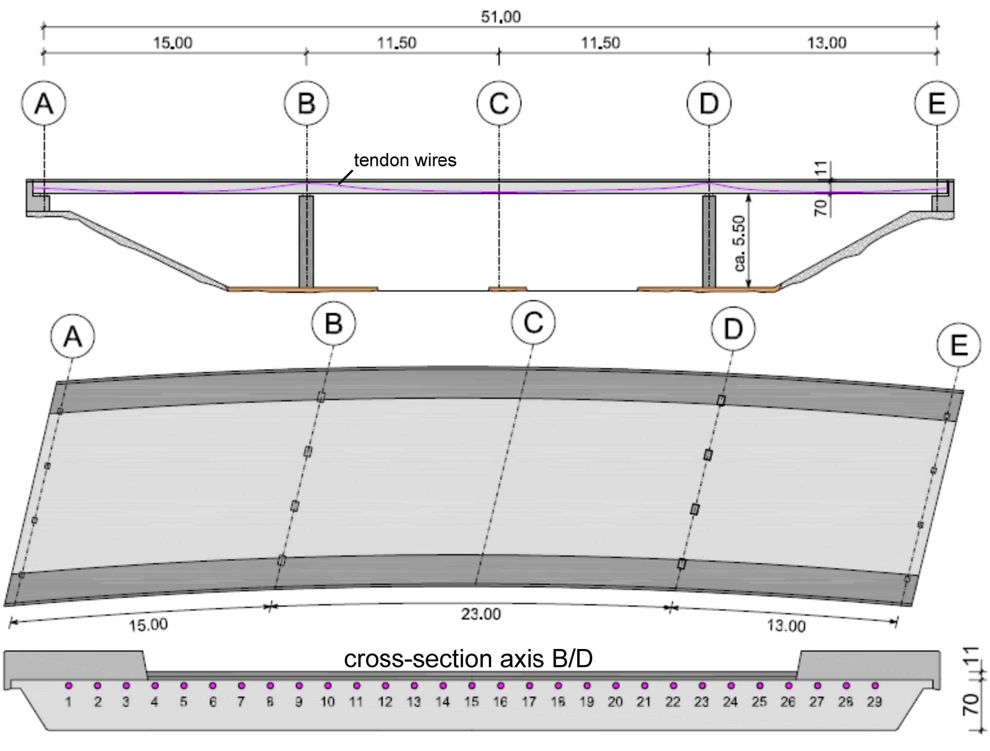

Figure 10: The investigated bridge, cross-section

The bridge was investigated in six different states. After the examination of the undamaged initial state, the roadbed was removed. In the next steps, it was possible to damage the bridge by cutting tendons. Table 3 lists the different scenarios. Figure 11 illustrates a cut tendon.

ID	scenario	location
#1	undamaged	
#2	undamaged, removed roadbed	
#3	failure of tendon 15	C
#4	failure of tendon 7, 13, 15, 17, 23	C
#5	failure of tendon 5, 7, 9, 13, 15, 17, 21, 23, 25	C
#6	failure of tendon 5, 7, 9, 13, 15, 17, 21, 23, 25	B, C, D

Table 3: Damage scenarios

Figure 11: Cut tendon

3.2 Static load tests

For each damage scenario, static load tests were performed in order to control the static deformation that is directly related to the bending stiffness of the bridge. This is a conventional method to assess the state of bridges. The loading has been applied using trailers for cement transportation. Figure 12 gives an overview of the situation. The deformation has been measured with displacement sensors that were fixed to a scaffold under the bridge. Figure 13 shows the deformation of one half of the bridge in case of four trailers in the mid-span. The deformation is nearly the same for all damage scenarios, about 6 mm in the centre of the mid-span. The small differences in deformation are difficult to analyse without an adequate FE-model. For neither of the scenarios cracks were visible. Every cross-section in the bridge remains under compression even for damage scenario #6. Thus, the stiffness of the bridge has not been decreased considerably, but only the pretensioning forces.

Figure 12: Trailers for static loading

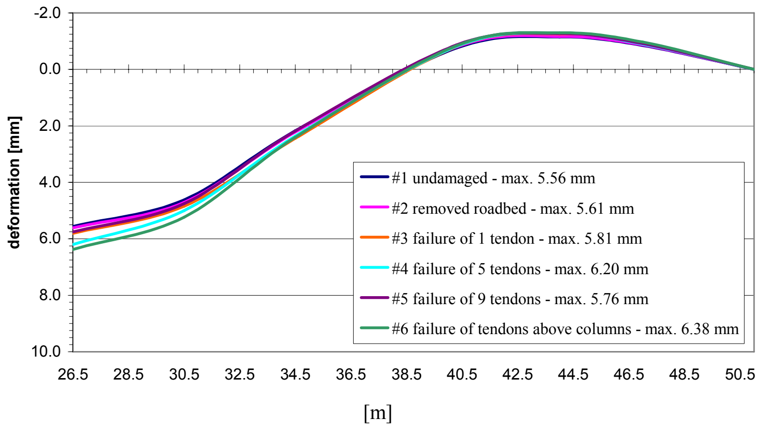

Figure 13: Vertical deformation along longitudinal direction

3.3 Dynamic tests

For the investigation of the dynamic behaviour, the bridge has been excited harmonically with an eccentric mass shaker by means of a swept sine in a frequency range including the first four eigenfrequencies. Figure 14 shows the first four mode shapes of the bridge. The sweep-rate was 0.02Hz/s. The tests were performed with excitation forces of 0.9kN, 1.8kN, 2.7kN, 5.4kN, 8.1kN and 9.9kN.

Changes in eigenfrequencies are one of the most popular damage detection methods. Figure 15 and Figure 16 show the FRFs in the frequency range of the first four modes for the six damage scenarios, measured with an excitation force of 900N. The first eigenfrequency in the initial state #1 (light green) has a value of approx. 4.0Hz. As expected, the eigenfrequencies increase when removing the roadbed due to the loss of mass, especially the eigenfrequencies of the bending modes (B1-4Hz, B2-11.6Hz).

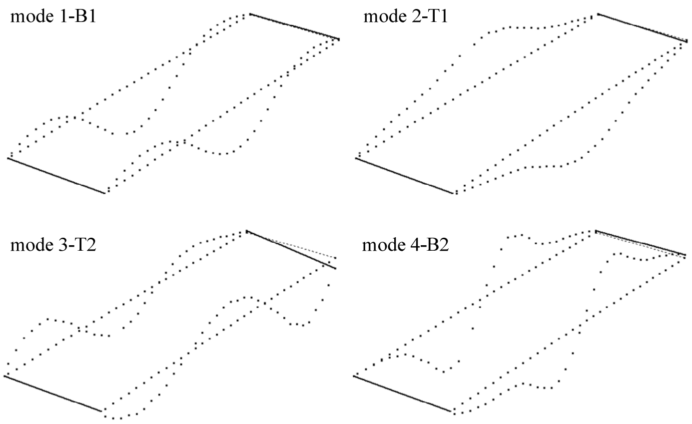

Figure 14: First four modeshapes, B-bending mode, T-torsional mode

It is also obvious that damping decreases when removing the roadbed and thus, the amplitude of the FRFs increase. Changes in the first four eigenfrequencies due to damage from scenario #2 to #5 are very small. For the second bending mode (B2) there is barely a difference visible. Thus, one can confirm the conclusion of the static tests that the influence of damage on the stiffness of the bridge is small (because the cross-section is still completely under compression). For scenario #6 only, the eigenfrequencies increase lightly.

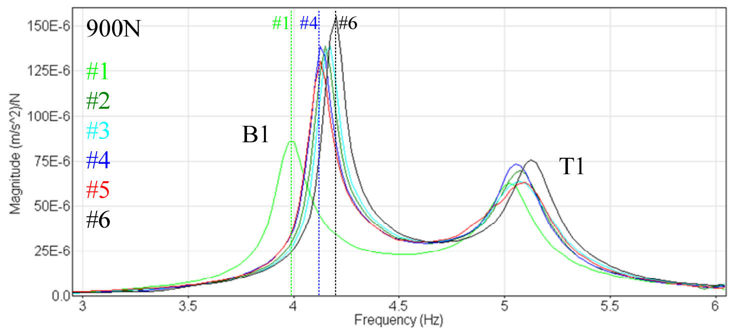

Figure 15: FRFs, modes 1 and 2, scenarios #1, #2, #3, #4, #5, #6, excitation force 900N

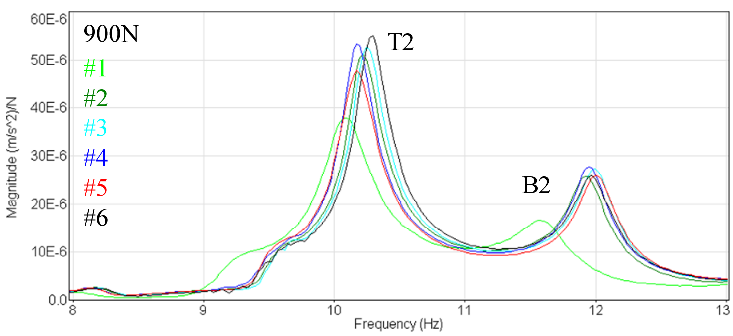

Figure 16: FRFs, modes 3 and 4 scenarios #1, #2, #3, #4, #5, #6, excitation force 900N

Another often applied method to assess the state of structures are changes in modeshapes. Figure 17 shows the first modeshape overlaid for all damage scenarios each normalized to the maximum in the mid-span. It is obvious that there are nearly no differences in the area of the induced damage in the mid-span. The first modeshape of #5 differs slightly from the first modeshape #2 and the other modeshapes in the area of the right nodal point (approx. 40m). The nodal point in this scenario moves slightly to the left. For the second modeshapes there is barely a difference visible. MAC, COMAC and Flexibility are insensitive to such small changes in modeshapes. The results are similar for the higher modes.

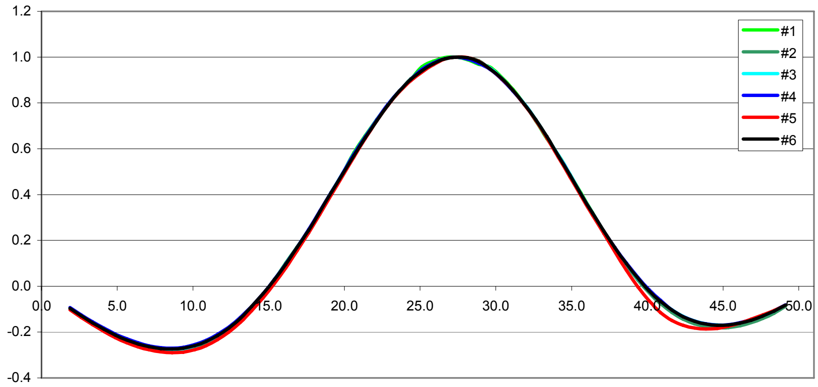

Figure 17: First modeshape, all scenarios

In a next step it is again interesting to know, whether the non-linear dynamic behaviour is more sensitive to damage. Figure 18 shows the FRFs in the frequency range of the first two modes for the six scenarios, measured with excitation forces of 900N and 9900N. For all scenarios, there is a small dependency of the eigenfrequencies on the vibration amplitude visible. Figure 19 contains the percentage decrease of the first two eigenfrequencies from an excitation force from 900N to 9900N. For scenario #1 the dependency is small. It has to be noted that the vibration amplitudes for this scenario are smaller compared to the other scenarios, due to the higher damping value. Therefore, a direct comparison is difficult. For the other scenarios, the dependency of the eigenfrequencies is more evident. Observing scenarios #2 to #5 the dependency of the first eigenfrequency varies. This is probably due to the change of the stress distribution in the cross-section of the bridge when cutting the tendons. The dependency of the second eigenfrequency is approx. constant. For scenario #6 the dependency is most evident. Altogether, the amplitude dependency of the eigenfrequencies is much smaller compared to the undamaged beam in chapter 2. This is supposed to be due to different material behaviour of reinforced- and prestressed concrete. However, the amplitude dependency is not negligible.

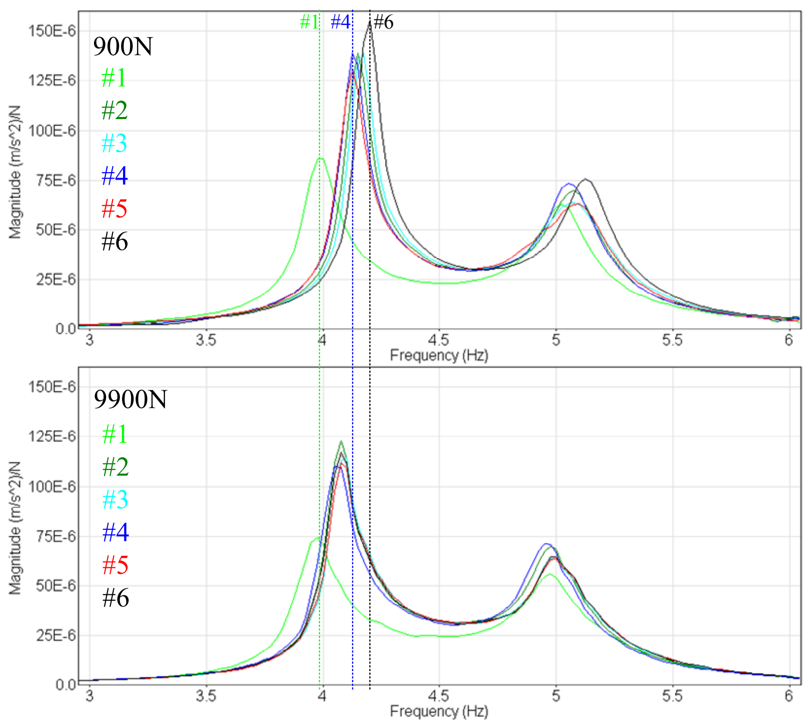

Figure 18: FRF, modes 1 and 2, scenarios #1, #2, #3, #4, #5, #6, excitation forces 900N and 9900N

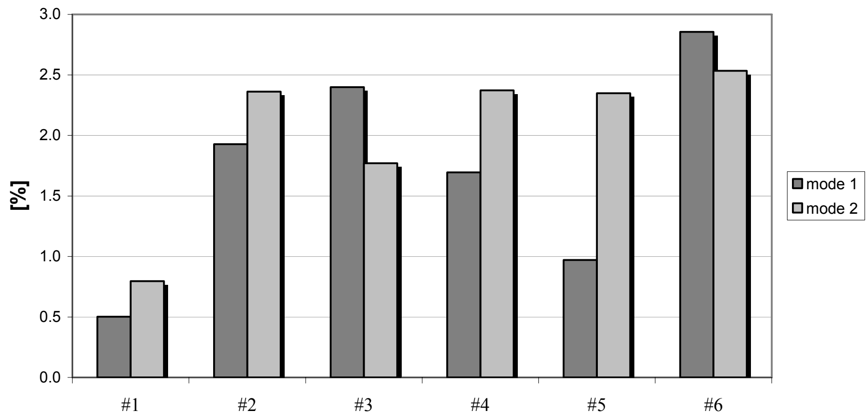

Figure 19. Percentage decrease of the first two eigenfrequencies from 900N to 9900N

Another question to answer is, whether Higher Harmonic Components can be detected in the response. According to the excitation of the beam the bridge has been excited in a small frequency range including the first and the second eigenfrequency (between 3.5Hz and 5.8Hz) with an excitation force of 2700N. Figure 20 shows the ratio of the input and output spectra (FRF_{small}) and the ordinary measured FRFs (excited frequency range between 3Hz and 35Hz) for all scenarios. For the undamaged state #1 FRF_{small} is nearly equal to the FRF, in the scheduled excited frequency range as well as in the range of the first HHC. That means that the bridge seems to respond linearly. Compared to scenario #1 where no HHC were visible the non-linear behaviour is different for scenario #2. There is a distinct peak at a frequency of approx. 8.2Hz which is the first HHC of the first eigenfrequency. For scenario #3 where one tendon has been cut the peak increases. While the peak decreases for scenarios #4 and #5 (it has to be mentioned that coherence is bad for these two scenarios), it increases again for damage scenario #6. It is interesting to note that this behaviour correlates with the amplitude dependent behaviour of the first eigenfrequency (cp. Figure 19). For scenario #6 there is a second remarkable peak at a frequency of about 6.7 Hz. It might also be visible for scenarios #3 and #4. But it has to be mentioned that for these scenarios and measurements the excitation force in this frequency range is very small and partially near noise.

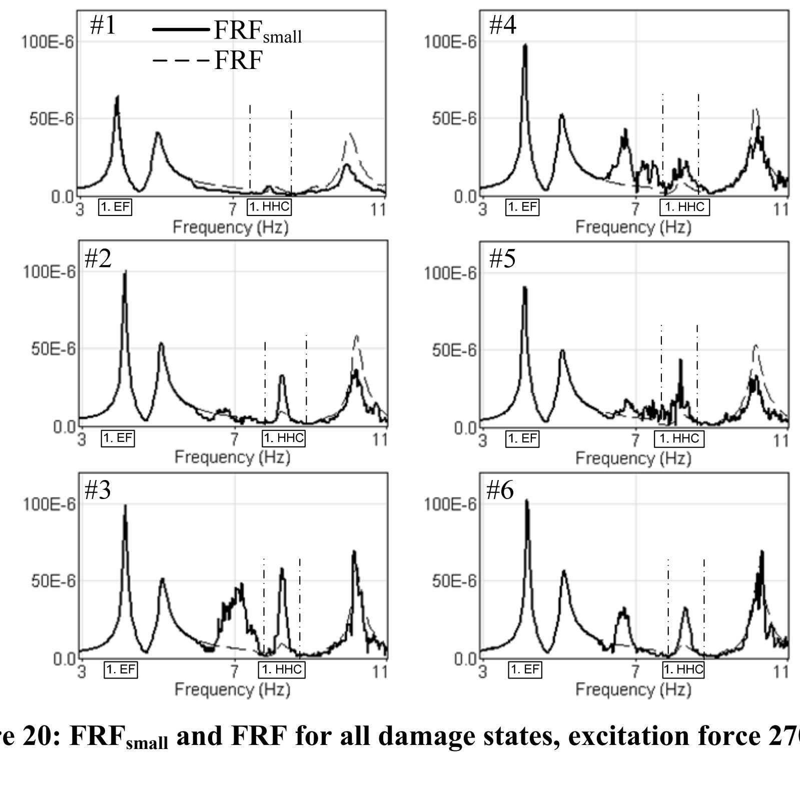

Figure 20: FRF_{small} and FRF for all damage states, excitation force 2700N

As no cracks could be discovered during the tests although a number of tendons were cut, the type of non-linearity is supposed to be of another type compared to the cracked reinforced concrete beam. The quantity of the non-linear behaviour changes with the number of cut tendons. Because every cross-section in the bridge is under compression for all scenarios, failure of the tendons is not a damage in terms of a reduction of stiffness, but rather a reduction of the pretensioning. This is also confirmed by the fact, that the deformation under applied static load was approximately equal for every scenario. Thus, it can be assumed that the occurrence of Higher Harmonics in this case depend on the pretensioning forces in the bridge and are a result of non-linear dynamic material behaviour of concrete under different compression conditions.

4 Conclusions

Using non-linear vibration analysis for detecting damage in civil engineering structures is still in the early stages. The presented experimental investigations show promising results. For the reinforced concrete beam there exists an obvious relationship between excitation force and eigenfrequencies. In undamaged case, the dependency of the eigenfrequencies on the excitation force is small but not negligible. In the cracked state the beam shows a very noticeable relationship between modal parameters and excitation force, especially for small levels of damage. It has also been shown that Higher Harmonics can easily been detected using the ratio of the response and the input spectra. The Higher Harmonics are a clear indicator of the presence of cracks.

The results for the investigated bridge are more difficult to interpret. The system of the bridge is more complex than a simple beam; furthermore, the system is prestressed. However, non-linear effects like amplitude dependent eigenfrequencies and damping are visible. Higher Harmonics were also detected as an indicator for non-linear behaviour. The idea that the non-linear behaviour of the bridge is not a result of cracks but a result of non-linear material behaviour of concrete under different compression conditions has to be further developed and confirmed in laboratory tests.

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