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Detection and localization of damage in civil engineering structures by using static and dynamic assessment methods

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ABSTRACT:

Visual inspections and static loading tests, both time consuming and cost intensive, were commonly used to reduce the costs of maintenance, to ensure the safety, the serviceability and to expand the durability of existing civil engineering structures. Thus, dynamic based condition assessment techniques of civil engineering structures have gained a lot of interest in the last years, as this method is an easy applicable and inexpensive technique to evaluate the mechanical behaviour of structures. Usually applied in the domain of mechanical engineering, these methods are more and more used to analyse civil engineering structures like buildings and bridges by interpreting the changes of dynamic parameters, such as eigenfrequencies and modeshapes depending on the structure's stiffness.

To permit an interpretation of measurable changes, the static and dynamic response of gradually loaded reinforced concrete and prestressed structures were analysed in function of their degradation. Some tests were executed under laboratory conditions and compared with an in-situ experiment of a real bridge. The observed variations in dynamic properties due to structural changes are quantified and compared with possible changes from environmental conditions, measured by a long-term monitoring system installed on a composite bridge. For all investigated objects, changes of measured dynamic parameters are analysed with regard to damage detection and localization.

1 INTRODUCTION

In the framework of a research project carried out at University of Luxembourg, common dynamic testing methods are used to describe the state of civil engineering structures and bridges made of reinforced and prestressed concrete as well as composites. For this purpose, changes in eigenfrequencies and modeshapes of structures tested in the laboratory of the University of Luxembourg are under investigation when damaging these types of structures stepwise. The so called modal properties change with damage state if the stiffness of the structure is affected (eigenfrequencies and modeshapes). In addition to that, typical changes in the nonlinear dynamic behaviour of structures with damage state are under investigation. These analyses focus on changes in the force amplitude dependency of eigenfrequencies and the appearance of higher harmonics if a nonlinear behaving system is excited harmonically. The excitation force amplitude dependency for harmonically excited systems is based on the nonlinear behaviour of the structure when oscillating. For instance, the oscillation behaviour and thus the dynamic answer of reinforced concrete depends on the opening and closing of micro cracks and, depending on the damage state, also of visible cracks in concrete. High force amplitudes induce a strong opening and closing of cracks which generally results in a lower dynamic stiffness and hence lower eigenfrequencies than small force amplitudes. The appearance of higher harmonics for harmonically excited systems is based on the non sinusoidal answer of a system itself. Detailed information of these studies as well as the basics concerning the dynamic condition assessment of civil engineering structures can be taken from Bungard et al. (2009), Waltering et al. (2009), Waltering (2009).

In comparison to laboratory tests, an in-situ object has been analysed which do not result from structural defects but from changing outer influences like changing temperature or boundary conditions. The main question is if these influences on the modal properties can be neglected or if they have to be considered to avoid misinterpretations. For this purpose, a long-term measurement of the dynamic behaviour of a composite bridge by means of the output-only method and forced excitation tests at different structural temperatures and different harmonic excitation force amplitudes have been conducted.

2 REINFORCED CONCRETE BEAM

The first presented specimen is an isostatic beam of reinforced concrete with a span of 3.60 m (figure 1). The beam has been loaded stepwise up to its ultimate load during a symmetrical three point bending test.

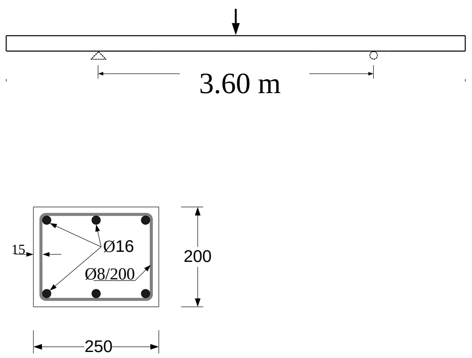

Figure 1. Static setup for the beam, cross-section, undamaged state #1.

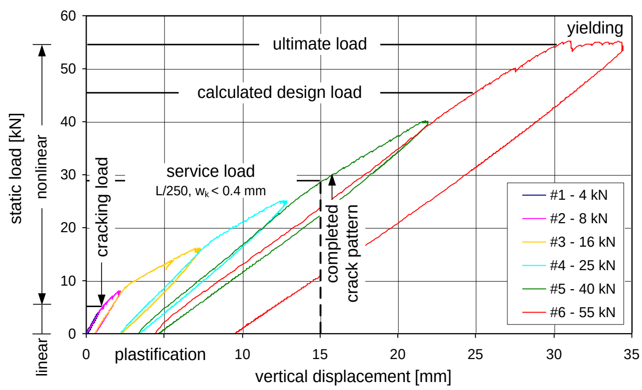

Figure 2. Force-Displacement diagram. The displacements at mid-section are shown.

The first applied load of 8kN was around the appearance of the first bending crack. Failure was defined at 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. In undamaged state no cracks were visible. For the first applied load of 8kN two small cracks arise at midspan. 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, while the concrete in the compression zone started to fail, too.

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, q.v. Waltering et al. (2008). Table 1 figures the first three measured eigenfrequencies (first three bending modes) for all load steps. As expected the eigenfrequencies decrease with increasing damage.

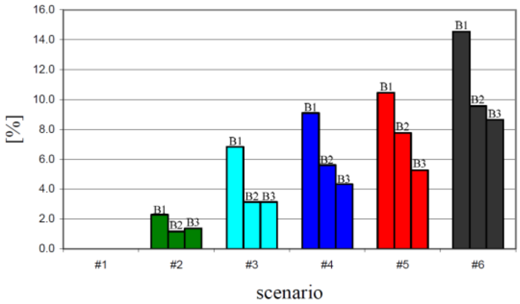

Figure 3. Percentage decrease of the first three eigenfrequencies related to the undamaged state #1 B1-mode 1, B2-mode 2, B3-mode 3

Table 1. First three eigenfrequencies (hammer impact)

load step	eigenfrequency [Hz] 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

Figure 3 plots the percentage decrease of the first three eigenfrequencies related to the undamaged state #1. The first eigenfrequency decreases about 2.3% for the first induced damage of small level. Selfevident, the first eigenfrequency is affected most. Depending on the size of the cracked zone, the second and third eigenfrequency are affected as well. . The evaluation of the decrease of the first eigenfrequency (figure 3 and table 2) shows that there is a maximum decrease of 14.5 % between the undamaged load step (#1) and the ultimate load step (#6). For the second and the third eigenfrequency the decreases are lower.

Table 2. Percentage decrease of the first tree eigenfrequencies according to the loading sequence

mode	#1	#2	#3	#4	#5	#6
B1	-	2.3	6.8	9.0	10.4	14.5
B2	-	1.2	3.1	5.6	7.7	9.6
B 3	-	0.8	3.3	4.2	5.1	8.5

3 DESCRIPTION OF THE PANEL

Two identical panels, one prestressed and one non-prestressed, both with identical reinforcement ratio, have been analysed. Figure 4 shows the cross-section of the panels. The measurements are divided in two parts: first, the static load test, with successively loading of weights in different damage scenarios for both slabs and, second, by dynamic measurements with a hammer impact.

The weights are arranged as shown on figure 5, in order to approximate the theoretical additional load and distribution for natural office conditions.

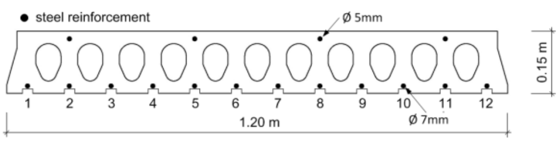

Figure 4. Cross-section of the panels

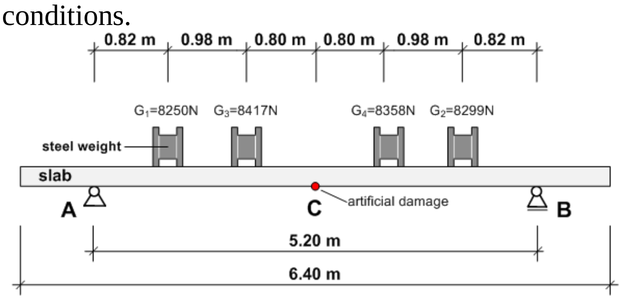

Figure 5. Sideview of the loaded slab

Table 3. Damage scenarios and load cases for the RC and PC slabs.

no.	damage scenario	location	Percentage of damage	additional loads
#0	initial state: undamaged	-	-	2, 4, 2* and 0* weights
#1	cutting of tendon no.: 6, 7	axis C	16.7 %	2, 4, 2* and 0* weights
#2	cutting of tendon no.: 6, 7, 2, 11	axis C	33.3 %	2, 4, 2* and 0* weights
#3	cutting of tendon no.: 6, 7, 2, 11, 4, 9	axis C	50.0 %	2, 4, 2* and 0* weights
#4	cutting of tendon no.: 6, 7, 2, 11, 4, 9, 3, 10	axis C	66.7 %	2, 4, 2* and 0* weights

(* represents the dynamic and static measurements after the slab is loaded with an additional load of 4 weights)

For the static load test, a first issue emphasises on the vertical deflection of the transducer in the middle of the slab for all damage scenarios and load steps according to table 3. As one can see in figure 6, the deformation for the RC slab is considerably higher for all damage scenarios than for the PC slab, which can be expected due to the early formation of a distributed crack pattern in the RC slab.

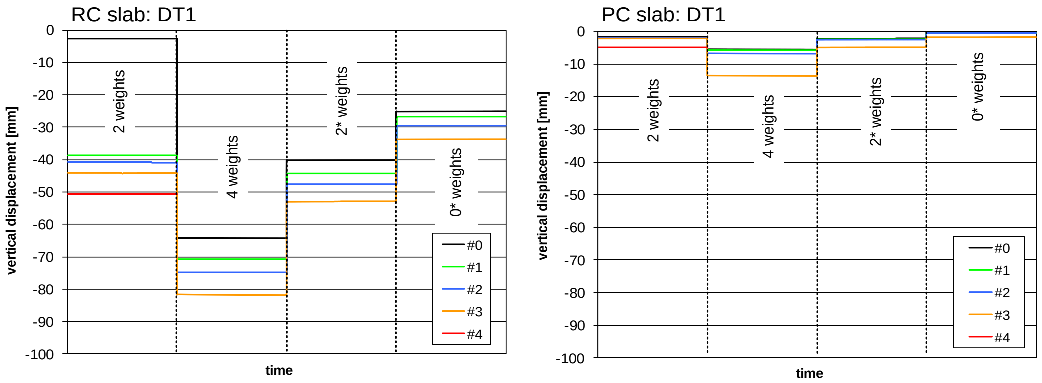

Figure 6. Changes in maximal vertical deflection for both types of slabs (left: RC slab; right: PC slab)

For the comparison on dynamical parameters, a Hammer-Impact (HI) test has been accomplished to see changes in eigenfrequencies for all testing scenarios (table 3).

Table 4. Measured 1st eigenfrequency in [Hz] for the RC slab with the Hammer Impact (HI) test and the percentage decrease of these eigenfrequency related to the initial state; on the right side the decrease is shown

damage scenario	#0	#0*	#1*	#2*	#3*	#4*
eigenfrequency (mode B1) [Hz]	11.00	9.18	8.07	7.85	7.69	failure
decrease [%]	-	16.5	26.6	28.6	30.1	-

(* unloaded, but after loading of 4 weights)

Regarding Table 5, which represents the value of the eigenfrequencies for the PC slab determined by the *Hammer Impact* method, one can see that these values are not as strongly depending on the damage scenarios as for the RC slab. Here, the formation of cracks just begins after the second damage state. In addition, the decrease is very small for the RC slab.

Table 5. Measured 1st eigenfrequency in [Hz] for the PC slab with the Hammer Impact (HI) test and the percentage decrease of these eigenfrequency related to the initial state; on the right side the decrease is shown

damage scenario	#0	#0*	#1*	#2*	#3*	#4*
eigenfrequency (mode B1) [Hz]	11.75	11.70	11.65	11.65	11.55	failure
decrease [%]	-	0.4	0.9	0.9	1.7	-

(* unloaded, but after loading of 4 weights)

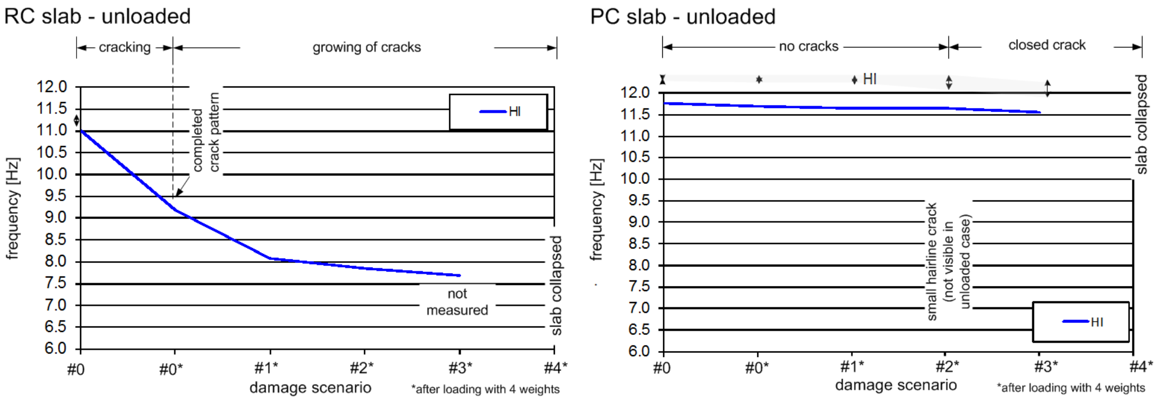

Figure 7. Changes of the first eigenfrequencies for both slab types (left: RC slab; right: PC slab)

To reinforce this effect, figure 7 illustrates the decrease of the first eigenfrequencies regarding both slabs. For the RC slab a clear decrease of the eigenfrequency up to 30.1% for the #3* damage state can be observed. Even after the crack pattern is completed, a decrease of 16.5% of the frequency can still be identified, probably due to the growing of the crack in the middle of the slab at axis C. In contrast the PC slab, which shows a decrease of around 1.7% only from the initial state #0 to the damage state #3*. This low value can be explained by the fact that the crack closes again after removing the 4 weights, and therefore, no changes in the testing can be recognised when no additional loads are applied.

4 BRIDGE DESCRIPTION

To compare the previous results with the variation due to climate changes, a two-lane road bridge located in Luxembourg has been investigated in situ. The composite bridge has an overall length of 37 m and consists of two spans with lengths of 24 m and 13 m (cp. figure 8).

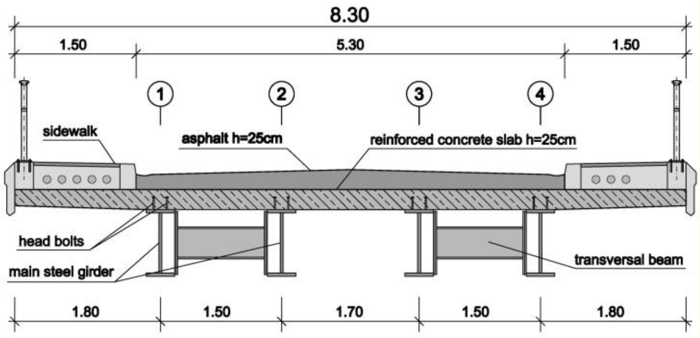

Figure 8. Side view of the bridge Figure 9. Cross section

The bridge consists of a reinforced concrete slab with a thickness of 25 cm and four main steel girders with heights ranging from 53 cm to 1.31 m according to the bending moment. Two steel girders are connected to each other with transversal ones at intervals of 4 m (cp. figure 9). The main steel girders and the slab are connected by headed studs and both ends of the girders are encased in reinforced concrete cross beams. The road surface is made of a huge roof-shaped asphalt layer (cp. figure 9) with a total thickness of also 25 cm.

For this bridge a special bearing situation is given as the smallest field is built-in between its two supports.

Since the beginning of 2007, the temperature behaviour of the concrete slab and the steel girders are measured continuously at different points. Furthermore, vertical accelerations of the superstructure are recorded at several points using ambient vibration (wind, traffic). The air temperature is measured by a meteorological station close to the bridge. The aim of this measurement is the investigation of the dependence of the eigenfrequencies of the temperature behaviour of the structure.

As result of this long term measurement a temperature difference of the structure between summer and winter is about 30 K. And in spring and summer, temperature differences between girders and slab of 10 K are possible. The temperature of the girders corresponds directly with the air temperature which has a large daily fluctuation in spring and summer. Daily changes in the temperature of the concrete slab are much slower and also less as the slab has a high thermal inertia.

By means of the installed acceleration sensors, the first four eigenfrequencies of the bridge have been determined by using ambient vibration.

The aim of the investigation of the eigenfrequencies was the determination of their behaviour with changing temperature conditions. It was possible to investigate the frequency behaviour for temperatures between -10°C and +30°C. In a first step, the analysis of the eigenfrequencies was conducted for equal temperatures of the girders and the slab. That means, only measurement data without any temperature gradient between girders and slab has been used.

Figure 10 presents the measurement results of the temperature behaviour of the first two eigenfrequencies. It is clearly visible that the investigated eigenfrequencies decrease significantly with increasing temperature. In relation to the highest measured eigenfrequencies of the investigated time period, the frequencies of the torsion modes decrease stronger than the frequencies of the bending modes. For the 1st torsion mode (T1) an annual percentage decrease of 38 % could be observed. The annual percentage decrease of the 1st bending mode (B1) was 22 %.

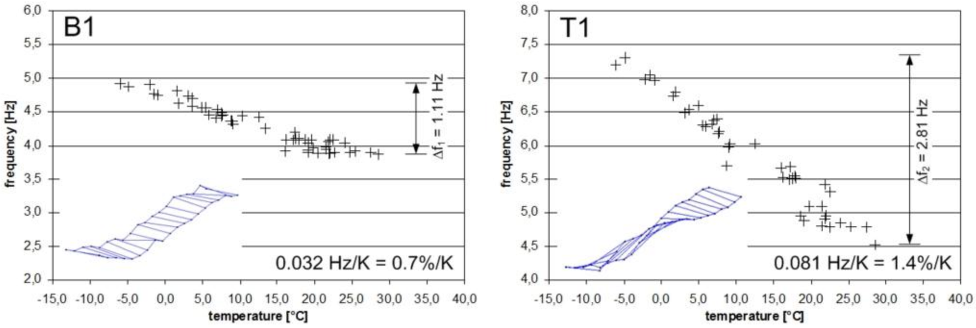

Figure 10. Temperature dependency of the 1st eigenfrequency (1st bending mode) and 2nd eigenfrequency (1st torsion mode) – identified from PSD by means of Peak Picking (PP)

In a second step, the influence of the temperature gradient between girders and slab has been investigated. It turned out that for a constant slab temperature and a varying girder temperature, the behaviour of the eigenfrequencies is comparable with that one without any gradient. For a constant girder temperature and a varying slab temperature none respectively very small changes have been observed. It can be assumed that the temperature dependency of this structure is based on changing bearing conditions and the strong stiffness changing of the huge asphalt layer with temperature. This is confirmed in literature and in laboratory tests, q.v. Bungard et al. (2010). A linear finite element simulation using the software ANSYS also confirms the assumption that the temperature dependency of the stiffness of asphalt causes the variation in the eigenfrequencies. A parameter study shows that a changing of the stiffness of asphalt in that way which was measured in laboratory for different temperature conditions results in the changes in the measured bending modes as well as torsion modes.

So by using a long-term measurement, it could be observed that the structure is influenced by large changes in temperatures and that, especially in spring and in summer, an additional influence of temperature gradients between steel girders and concrete slab is present. Furthermore it could be observed that the eigenfrequencies depend strongly on the temperature condition of the structure (cp. figures 10 as well as table 6). With respect on the interpretation of changes in modal properties concerning condition or damage assessment, the temperature effects on modal properties cannot be neglected.

Table 6. Annual percentage changing of eigenfrequencies in relation the highest measured value

mode	B1	T1
annual change	22 %	38 %

5 CONCLUSIONS

Static and dynamic tests have been realized on prestressed and non-prestressed concrete slabs. The decrease of the first eigenfrequency for the 50% damaged RC slab (damage scenario no. #3) is nearly 30% in correlation to the undamaged state. For the PC slab the decreasing is only 1.7 %. So it becomes obvious that damage assessment supported only on the basis of percental decrease of the eigenfrequencies can only be realised by a comparison to finite element calculations calibrated on initial measurements.

For the in-situ object the long-term measurement by means of the ambient vibration method has shown that eigenfrequencies change strongly with changing temperature conditions of the materials. For the eigenfrequencies, an annual changing up to 38 % is measured (cp. table 6). Concerning condition assessment by means of the interpretation of structural defects, these environmental changes cannot be neglected and have to be considered avoiding misinterpretations.

Therefore initial measurements during the first year of the construction to determine the sensitivity of the structure due to temperature variations have to be realized. In this way, later measurements could be interpreted on the basis of these sensitive results.

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