BUDAPEST UNIVERSITY OF TECHNOLOGY AND ECONOMICS

CRACK-RELATED DAMAGE ASSESSMENT OF CONCRETE BEAMS USING FREQUENCY MEASUREMENTS

New scientific results of the PhD thesis

Tamás KOVÁCS
Budapest University of Technology and Economics
Department of Structural Engineering

Supervisor:
György FARKAS, Dr. habil
Professor
Budapest University of Technology and Economics
Department of Structural Engineering

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1 INTRODUCTION

According to experience, civil engineering structures can be kept in service economically at the intended technical level with the specified structural reliability (i.e. safety and serviceability) during their design working life. During this period they are exposed to various deteriorating effects that unfavourably influence their durability. The recently published *fib* Model Code for Service Life Design proposed Condition Control Levels according to *Table 1.1* in order to preserve the structural condition during the design working life.

Table 1.1 Condition Control Levels according to the *fib* Model Code for Service Life Design

Condition Control Levels	Characteristics	Examples
CCL3	Extended inspection	Systematic inspection and monitoring of relevant parameters for the deterioration process(es) that is (are) critical
CCL2	Normal inspection	Regular visual inspection by qualified personnel
CCL1	Normal inspection	No systematic monitoring nor inspection
CCL0	No inspection	No possible inspection, for instance due to lack of access

Condition control systems for civil engineering structures are intended to identify damage. According to *Rytter (1993)*, a full damage identification procedure consists of the following four levels:

Level 1: *Detection*: Determination that damage is present somewhere in the structure;
Level 2: *Localisation*: Determination of the location of damage in the structure;
Level 3: *Quantification* of the severity of the damage;
Level 4: *Prediction* of the remaining service life of the structure.

This work makes a distinction between “damage identification” and “damage assessment”. In this context, damage assessment means an evaluation process (method) which provides information on the structural performance based on the analysis of the structural response. Damage identification is considered as a precondition to damage assessment which covers activities focusing on the first three of the above levels (Level 1-3).

Damage identification and assessment may be carried out on the basis of either static (conventional load test) or dynamic (vibration test) approach depending on the character of the investigated parameter addressed (measured, calculated) by the method. Vibration tests generally require less equipment and disturb the normal service of the structure in less degree than conventional load tests.

1.1 Existing vibration-based damage identification and assessment methods

The idea of dynamic-based damage identification for special civil engineering structures arose when the oil industry started to pay considerable attention to the assessment of offshore structures (oil platforms and light stations). Nowadays vibration-based damage identification is mainly used as part of a monitoring program for bridges.

The basic idea of these methods is that modal data of structures (natural frequencies, mode shapes, modal damping) are functions of physical (mass, stiffness, internal damping) as well as structural (geometry, support conditions) properties. Therefore, any change in the physical and/or structural properties induces a change in the modal data, which can be detected by suitable diagnostic techniques.

The existing methods are often categorized according to the dynamic characteristic that is analysed (measured and/or calculated) during the procedure (*Doebling et al. (1996, 1998)*), as follows:

- Methods based on changes in the natural frequencies;
- Methods based on changes in the mode shapes and in their derivatives;
- Methods based on changes in damping parameters;
- Methods based on dynamically measured flexibility;
- Matrix update methods;
- Non-linear methods.

The effectiveness of these methods in both experimental and practical applications is different depending on i) the information content on the structure gained through the analysed dynamic parameter, ii) the necessary computation effort and iii) their sensitivity to damage.

1.2 Motivation of the research

The existing vibration-based methods focus mainly on detection, localization and maybe quantification of damage but the structural performance-related issues remain generally untouched. The need for assessment methods, which are able not only to identify damage by dynamic methods but to provide information on the structural performance, is the main idea behind the proposal of Level 3a as a new intermediate level after Level 3 in the above damage identification system:

Level 3a: *Evaluation* of the structural performance based on damage assessment.

1.3 Goals of the research

The global purpose of this research was to fulfil Level 3a of crack-related damage assessment carried out on concrete beams under experimental conditions. To address Level 3a, the following goals were set out:

- Introduction of the achieved assessment level of existing dynamic-based damage assessment methods. Determination of tasks to achieve Level 3a under experimental conditions.
- Modelling crack-related damage under experimental conditions on various types of concrete beams.
- Elaborating signal processing tools to process numerically-recorded vibration data and then to provide statistical basis for the determined dynamic parameters (here the first two natural frequencies). Development of appropriate frequency evaluation techniques to reflect on different damage cases.
- Definition of damage indices, which are able to identify crack-related damage as well as to measure its extent (identification).
- Establishment of relationships between the measured dynamic parameters and the related structural performance-characterizing properties (assessment).
- Application of the method (or one of its part) on a real structure.

Recently executed, on-site load tests on structures are generally declared as successful if the difference between the calculated (model-based) and the measured (non-model-based) values of the investigated parameter remains below an accepted level. This level takes into account the effect of model uncertainties that are consequences of assumptions (regarding geometry, materials, loads, material law, etc.) of the applied model. Dynamic models require assumptions, additional to the static models, regarding the (often non-linear) behaviour of the deteriorated structure under movement, which results in further model uncertainties. Therefore, another goal of this research was to omit the development and use of dynamic models when defining damage indices and to base them on static models (static-based indices).

2 SUMMARY OF THE RESEARCH

2.1 Interpretation and modelling of damage

For experimental purposes, damage was modelled either by cracking for reinforced concrete beams or by the combination of cracking and tendon cuts for prestressed beams. Damage was measured by the extent of cracking and by the extent of reduction in the effective prestress due to tendon cuts. To model structural strengthening, post-tensioning was applied to a few, already deteriorated reinforced concrete beams. Cracking was caused and further intensified by pure flexure in steps within an experimental deterioration process. No primarily shear-related behaviour was considered and investigated; test parameters were chosen accordingly.

2.2 Experimental deterioration process

Reinforced concrete (non-prestressed) (P1, P2, R1, R2) and prestressed (P2p) model beams with the same concrete cross-section and a total height of 190 mm (*Fig. 2.1, Table 2.1*) as well as with different reinforcements were tested.

Table 2.1 Geometrical and reinforcement data of model beams

Beam type	No. of beams	Length: Total length L_b [m]	Length: Span L [m]	Reinforcement: Strength $f_{pk}/f_{p0,1k}$ or f_{tk}/f_{yk} [N/mm^2]	Type (surf.)	No. & ϕ [mm] of bars	Steel ratio, ρ [%]: $A_s/(b_w d)$	Steel ratio, ρ [%]: A_s/A_c	Initial prestress
P1	1	3.4	3.2	1770/1520	prestressing steel (tr.)	2ϕ5.34	0.527	0.284	unstressed
R1	4	4.4	3.8	600/500	reinforcing steel (sp.)	2ϕ8	1.183	0.636	-
P2	3	4.4	3.8	1770/1520	prestressing steel (tr.)	(7+1) ϕ4.7	1.436	0.769	unstressed
R2	3	4.4	3.8	600/500	reinforcing steel (sp.)	3ϕ8	1.774	0.955	-
P2p	3	4.4	3.8	1770/1520	prestressing steel (tr.)	(7+1) (5+1) (3+1) (2+1) ϕ4.7	1.436 1.040 0.645 0.449	0.769 0.549 0.329 0.220	$0.61 f_{pk}$

All beams were prismatic, both the concrete section and the embedded reinforcement were unchanged along the full length of each beam.

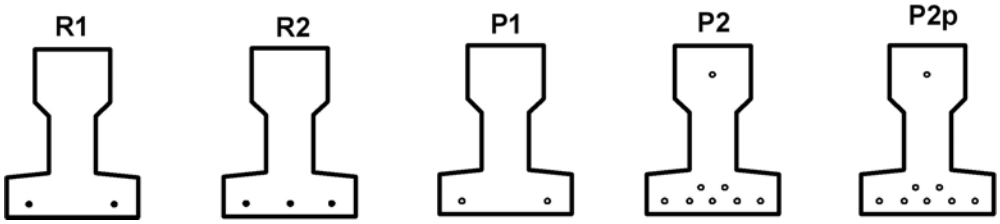


Fig. 2.1 Cross sections of model beams

For the P2p beams gradual, tendon breaks were simulated by sawing the intended number of tendons (wires) in the bottom flanges of beams through the concrete cover at selected cross-sections (cut points) along the beam length (*Fig. 2.2*).

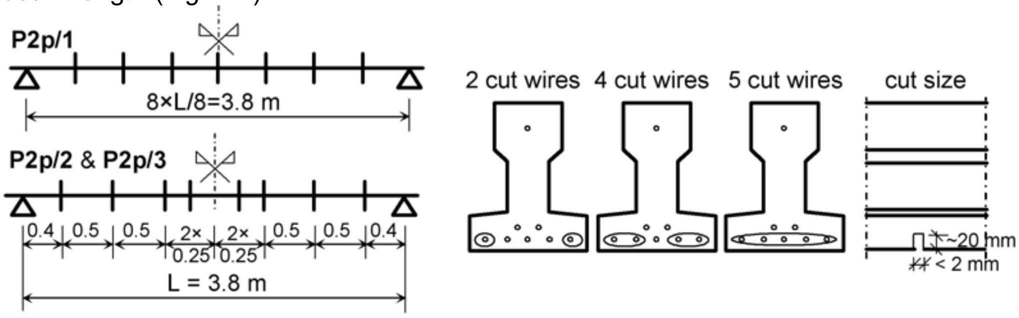


Fig. 2.2 Wire cuts for the P2p specimens

To simulate a possible strengthening effect, a few, reinforced concrete (non-prestressed) beams (P1, R1/4, R2/1, P2/1, P2/2), which had already undergone the complete deterioration process, were equipped with an external post-tensioning system (*Fig. 2.3*).

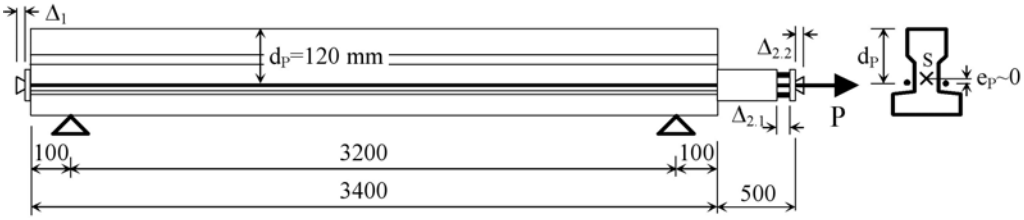


Fig. 2.3 Arrangement and geometry of post-tensioning for the P1 specimen

All beams underwent an artificially-produced *deterioration* process in several, exactly-defined steps (marked by states 1-5 (1-7 for P2/1 & P2/2)). Then, as a simulated *strengthening* effect, the external post-tensioning was applied to the selected, reinforced concrete (non-prestressed) beams (states 6p-8p).
The intended cracking was produced and intensified in the *loading phases* of the deterioration states by force-controlled four-point bending as shown in *Fig. 2.4*. One loading phase belonged to each state.

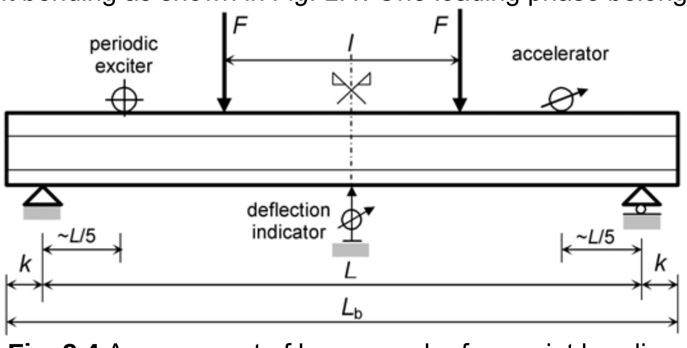
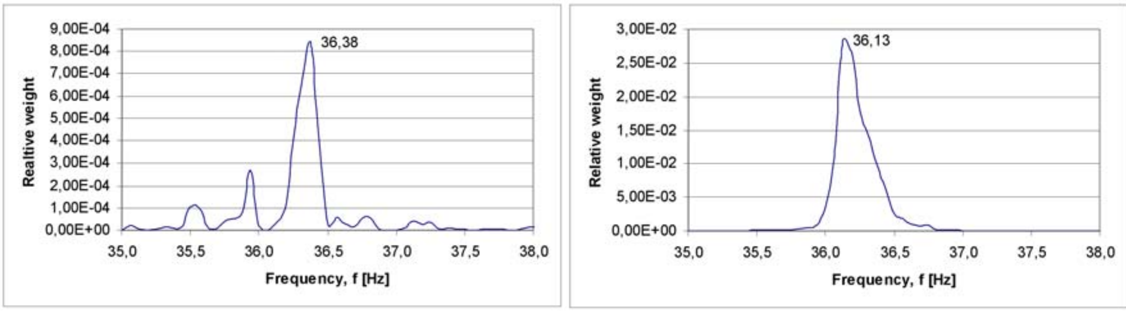


Fig. 2.4 Arrangement of beams under four-point bending

Each state ended in a dynamic *measuring phase* with the aim of determining the first two natural frequencies. The numerical values of natural frequencies were obtained from the statistical analysis of vibration signals. Two types of excitation were investigated. The first was a single mechanical impact made by a rubber covered mallet on the beam. The second was a nearly harmonic effect produced by a periodic exciter (*Fig. 2.4*), by which only the close vicinity of natural frequencies were excited with about constant excitation force. The acceleration-time (*a-t*) signals were recorded in the first case under free vibration for beams after impact and, in the second case, under a nearly harmonically excited vibration for beams being very close to resonance. Resonance effects were produced by coinciding the excitation frequency with the natural frequency to be measured.

2.3 Signal processing

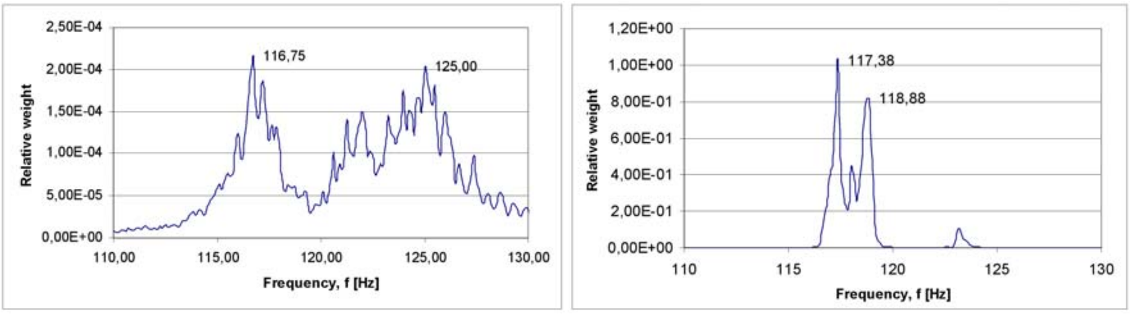
The recorded *a-t* signals first had to be transformed by discrete Fourier transforms into *individual frequency spectrums* then their statistical analysis was carried out.
It was pointed out that both excitation inputs had weaknesses, which might result in unacceptable errors in the individual frequency spectrums. In order to reduce the relative weight of these errors, minimum 30 individual frequency spectrums were averaged manually, which resulted in the average frequency spectrums (*Fig. 2.5*) (*average spectrum method*).



a) From impulse excitation b) From harmonic excitation

Fig. 2.5 Average frequency spectrums for the first natural frequency (f_1) (state 0 of P1)

It was observed that average frequency spectrums generally indicated f_1 with relatively high reliability. Here the reliability was measured by the standard deviation (SD) of frequencies at maximum ordinates in the individual spectrums. However, it was also observed that for beams with considerable cracking the average frequency spectrums compiled from signals due to either impulse or harmonic excitation often contained multiple peaks around f_2 (*Fig. 2.6*). The reason for that was the lost symmetry of bending stiffness distribution caused by the non-symmetric crack pattern. Owing to this, the natural frequency, which was associated with the strictly asymmetric mode shape, shifted slightly and an additional (virtual) frequency appeared in the frequency spectrum and represented itself as a secondary peak close to the shifted natural frequency. This supposition was verified by two facts. One was that the occurrence of such multiple peaks remained marginal for f_1, at which the whole beam vibrates in the same phase. The second fact was that they were experienced around f_2 only for states with intensive cracking (higher possibility for non-symmetric crack patterns).



a) From impulse excitation b) From harmonic excitation

Fig. 2.6 Average frequency spectrums with significant multiple peak around f_2 (state 4 of P1)

When the local sub-peaks fell relatively far from each other in the average frequency spectrum the question "what to consider as natural frequency when multiple peaks exist" remained stressful because non-symmetric crack pattern was relevant and actually uncontrollable. During the evaluation process, two cases were distinguished. In the first (more frequent) case, no dominance between sub-peaks (about the same relative weight) in the related individual frequency spectrums occurred. Here f_2 was set as the simple average of frequencies belonging to local sub-peaks in the individual frequency spectrums (*average frequency method*). However, when the relative weights of sub-peaks were significantly different and one of them was dominant over the others in each individual frequency spectrum (this was presumably the case for significant bending stiffness difference between the two beam halves), the average frequency method reflected only the dominant peaks. In this latter case it was decided to use signals derived from harmonic excitation to more accurately determine the position of both sub-peaks on the frequency scale and then to include the frequencies belonging to both sub-peaks into the averaging process with the same relative weight. In order not to lose information on the presence of multiple peaks when using the average frequency method, the associated SD values were also calculated (and found to be much higher than for frequencies without multiple peaks).

Due to the high coincidence between the determined natural frequencies obtained from signals with different excitation inputs for the P1 beam, the *a-t* signals derived from only the impulse excitation were generally used for further investigation purposes.

2.4 Definition of damage indices

Damage indices are defined to express information on the structural condition in numerical form. Any damage in the structure changes the structural information gained by the index and, consequently, indicated as change in the value of the damage index.
For this test damage indices were defined for identification purposes and classified as follows. Depending on their information content on the structure, the defined indices were classified as either global or local. A damage index was considered as local if its value depended on features of a local zone of the beam. Global indices were generally deduced from structural parameters influenced by features of many (preferably all) zones of the beams.
The non-model-based indices were either defined as static parameters measured under or after loading phases or deduced from those without any additional model. The model-based indices were based on a previous static model and deduced from it by calculations. Here the non-model-based indices were used, on one hand, to describe the structural behaviour of test beams for experimental purposes and, on the other hand, to check the applicability of the calculated model-based indices by conformity checks.
Behind the damage index definition it was supposed that:

a) the amount of energy dissipated in the vicinity of cracks was proportional to the amount of energy transmitted to the beam by the acting forces in the current loading phase;
b) the degree of bending stiffness decrease was proportional to the extent of cracking, which was measured by the number and the width of cracks.

If no new damage developed in an intermediate state of the deterioration process then the applied damage index was defined such that its value corresponding to this state was equal to that corresponding to the previous state. This cumulative form of the indices was intended for damage quantification (Level 3). For detection purposes (Level 1) the increments of the cumulative index between the subsequent states were deduced, which for states without new damage became equal to zero. In this work the applied damage indices were defined according to their latter incrementative forms.
The following non-model-based (measured) damage indices were defined:

1. *Growth of "total length of cracks"* (Δl_r) – global index
   "Total length of cracks" summarized the lengths of individual cracks over one lateral face of beam.
2. *Growth of total* (Δa_{tot}) *and residual* (Δa_{res}) *deflection at midspan* – global indices
3. *Growth of crack width at midspan* (Δw) – local index
   Here the width of the widest breathing crack being within the ~0.5 m long vicinity of midspan was considered.

In calculating the model-based indices nominal geometrical sizes, mean values of material properties and nominal values of controlled loads were assumed. With regard to material law, a linear elastic stress-strain relation was taken into account without limits in stress or strain, therefore plastic deformations were not addressed in the models. Concerning the structural behaviour the applied curvature

$$\rho(x) = \zeta(x)\frac{M(x)}{E_{cm}I_{II}(x)} + [1-\zeta(x)]\frac{M(x)}{E_{cm}I_{I}(x)} \quad (1)$$

and crack-width

$$w_{calc} = s_{r,max}\,(\varepsilon_{sm} - \varepsilon_{cm}) \quad (2)$$

models were adopted by Eurocode 2. For this experimental purpose the following model-based (calculated) damage indices were defined:

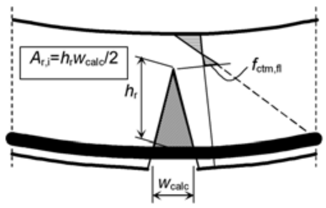


Fig.2.7 Assumed section of an individual crack

1. *Growth of "total of crack sections"* (ΔA_r) – global index (*Fig. 2.7*), where

   $$A_r = \sum_{L_{r,i}} \frac{s_{r,i}}{s_{r,max}} A_{r,i} \quad (3)$$

   where $L_{r,i}$ measured the cracked length of beams in the *i*-th state, $s_{r,i}$ was the distance within $L_{r,i}$, along which w_{calc} and h_r (both included in $A_{r,i}$) were averaged.
2. *Growth of calculated midspan deflection* (Δa_{calc}) – global index
3. *Growth of calculated crack width at midspan* (Δw_{calc}) – local index
4. *Growth of internal strain energy* (ΔW) – global index, where

   $$W = \int_{L_r} M(x)\rho(x)\,dx \quad (4)$$

The idea behind this index was based on the assumption that the damage increment in a concrete beam cross-section, which cracked earlier under flexure and was again subjected to and deformed by flexure ($M(x)$), was proportional to that part of the internal strain energy of the deformation, which was incremental to the internal strain energy, which occurred due to previous bending deformations.

The applicability of model-based indices, whether they appropriately reflect the actual structural behaviour, was confirmed by conformity checks with the corresponding non-model-based (deflection and crack width-related) indices (*Fig. 2.8*).

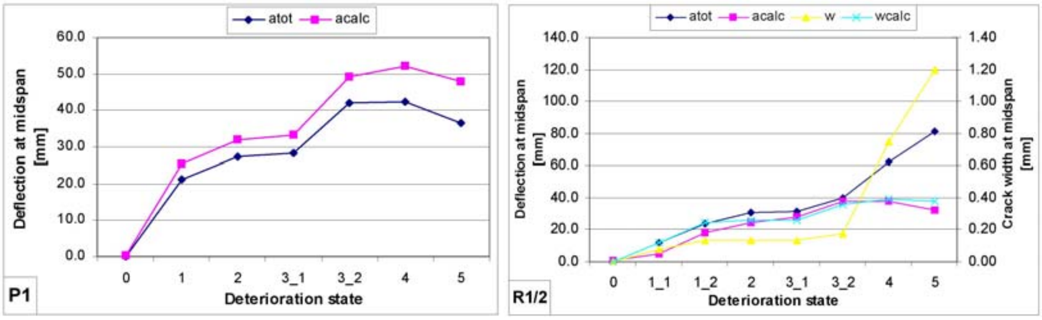


Fig. 2.8 Conformity check of non-model-based and model-based indices

Generally good correlation was found between the corresponding deflection and even crack width-related indices with the exception of states, when plastic deformations occurred (e.g. states 4 & 5 for R1/2). It was in accordance with the applied material law (linear-elastic) for the model-based indices.

2.5 Damage identification and assessment of model beams

For identification purposes, the incrementative form of damage indices as well as of measured frequencies ($\Delta f^i = f^i - f^{i-1}$; i marked the state number) were used.

When focusing on *detection* (Level 1), possible damage was indicated at states with high amplitudes of damage indices. This indication was confirmed (i.e. damage was detected) if the shape of Δf (Δf_1 and/or Δf_2) was similar to that of damage indices. Generally good correlation in shape between the applied non-model-based indices (*Fig. 2.9a*) as well as between the model-based indices (*Fig. 2.9b*) as well as between non-model-based and model-based indices was found for most of the beams. Due to its local character, Δw_{calc} regularly indicated more intensive damage in states when the first cracks appeared, and less intensive damage growth in the subsequent states, in contrast with other indices. Plastic deformations were clearly addressed by the non-model-based indices and completely disregarded by the model-based indices. Damage was definitely confirmed by both Δf_1 and Δf_2 in states when first cracks or when plastic deformations appeared.

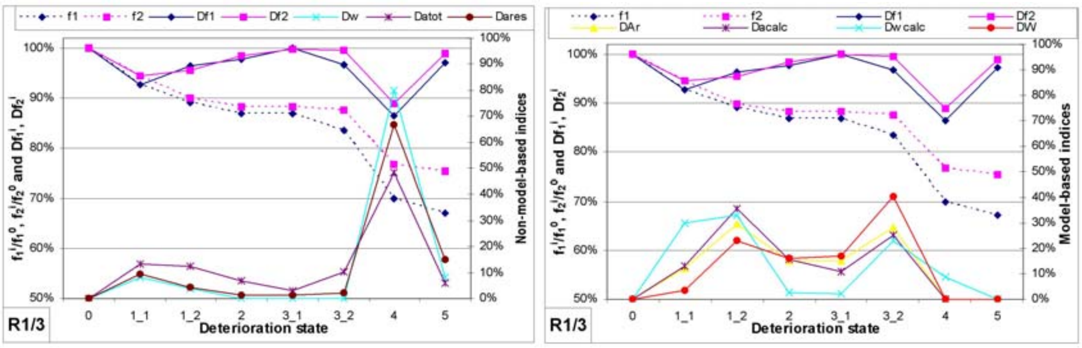


Fig. 2.9 Comparison of non-model-based and model-based indices for the R1/3 beam

Quantification (Level 3) focused on the magnitudes of values of incrementative damage indices. The magnitudes were proportional to the extent of damage occurring in the considered state. High amplitudes were found in states when the first crack appeared for all beams and for states of the related beams when plastic deformation occurred. Intensive cracking outside these regions was also reflected by relatively high magnitudes of the indices (e.g. at state 32 of the P1 beam in *Fig. 2.10*). Similar magnitude ratios for Δf confirmed the indications of damage indices.

For *localization* purposes (Level 2), the amplitudes of Δf_1 and Δf_2 belonging to the same state were compared to each other, state by state (comparison of trends in Δf). A natural frequency shift was mostly influenced by damage located around the highest amplitudes of the associated mode shape. Thus, owing to the applied load configuration, Δf_1 being higher than Δf_2 was expected and confirmed for states in which loads were positioned close to midspan, and the opposite ($\Delta f_2 > \Delta f_1$) for states with loads acting far from midspan (*Fig. 2.10*).

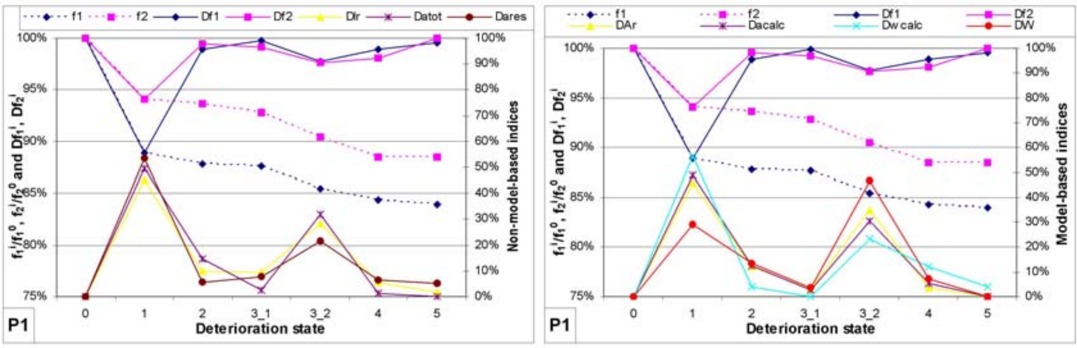


Fig. 2.10 Natural frequency shifts versus damage indices for the P1 beam

The assessment of beams focused on relationships between the measured natural frequencies (and their derivatives) and the structural performance. The main goal was to set basic trends and to define the likely intervals of natural frequency changes in typical deterioration states.

- For the *reinforced concrete (non-prestressed) beams* the following relationships were analysed:

a) Influence of the internal force level on the total decrease in f

The internal force level in beams was measured by proportioning the maximum bending moment at midspan (M_{max}) to capacity-like parameters such as M_r, M_{Rd} or M_{Rm}. The use of the M_{max}/M_{Rd} ratio was recommended for design purposes while M_{max}/M_{Rm} was proposed for experimental purposes. The total decrease in f ($\Sigma\Delta f$) was calculated generally as $\Sigma\Delta f = f^{last} - f^0$, where f^{last} and f^0 were the value of f_1 or f_2 in the deterioration state associated with the highest M_{max} (maximum damage) and in state 0 (no damage), respectively (*Fig. 2.11*). As shown, $\Sigma\Delta f$ had a clear increasing tendency with the increase of M_{max}/M_{Rm}. Starting from $M_{max}/M_{Rm}\approx 0.9$, $\Sigma\Delta f$ increased at a much higher rate than before, which clearly indicated the appearance of plastic deformations. For beams with $M_{max}/M_{Rm}\approx 1$, $\Sigma\Delta f$>15% was observed. For design purposes $M_{max}/M_{Rm}\approx 0.67$ was applicable, where on average a ~8% natural frequency decrease (in both f_1 and f_2) compared to the undamaged beam was found.

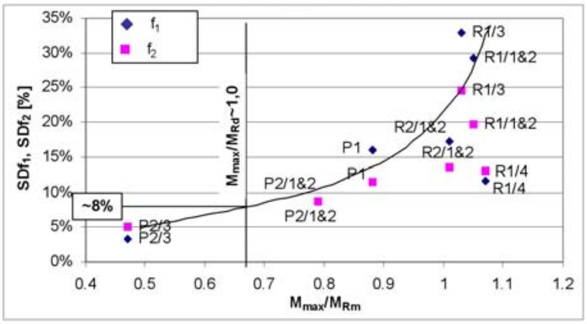


Fig. 2.11 $\Sigma\Delta f$ versus M_{max}/M_{Rm}

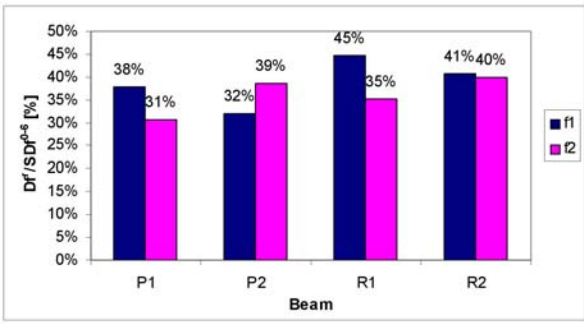


Fig. 2.12 Δf [%] at the appearance of first cracks

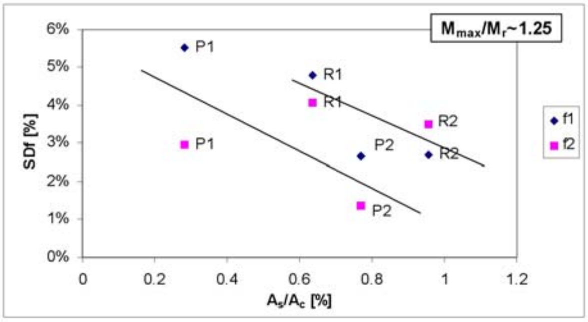
b) Influence of first cracks on the magnitude of Δf

The appearance of first cracks, which is of huge practical relevance for concrete structures, was identified as a significant damage event for all beams. *Fig. 2.12* shows how effectively this event could be indicated by natural frequency shifts proportioned to shift over the full range of practical applicability of beams. Δf^f measured the natural frequency decrease in the state when cracking started to develop and $\Sigma\Delta f^{0\text{-}6}$ measured the total decrease in f over (the first six) states, during which beams gave a dominantly elastic response. As shown, Δf^f obtained at least 31% of $\Sigma\Delta f^{0\text{-}6}$ for all beam types. This demonstrated that natural frequency shifts were good indicators of first cracks.

c) Influence of type and amount of reinforcement on $\Sigma\Delta f$

For reinforced concrete beams under pure flexure, the extent of cracking is tightly influenced by the amount of reinforcement. *Fig. 2.13* shows the influence of the steel ratio ($\rho = A_s/A_c$) on the total decrease in f ($\Sigma\Delta f$) at two typical stages (defined by the M_{max}/M_r ratio) of the introduced deterioration process.

Note that damage induced by first cracks is not affected by steel strength (if steel remains elastic after crack formulation). For $M_{max}/M_r\approx 1.25$ (immediately after first cracks) both $\Sigma\Delta f_1$ and $\Sigma\Delta f_2$ fell between 1% and 6%. Because of the load position in these states, f_1 was affected to a higher degree than f_2. For beams with the same type of steel (identical bond), the magnitude of $\Delta\Sigma f$ unambiguously decreased with the increase of ρ. For identical ρ values, higher $\Sigma\Delta f$ was obtained for beams with high-bond reinforcing steel compared to those with low-bond (unstressed) prestressing steel. Similar tendencies, but with higher $\Sigma\Delta f$, which fell between 4% and 15%, were obtained for $M_{max}/M_r\approx 3.05$. *Fig. 2.14* shows the same relationship but for M_{max}/M_{Rm}≈0.72. Note that M_{Rm} is highly sensitive to steel strength, therefore the strength-modified steel ratio ($\rho_f=\rho f/f_{tk}$) was applied. As shown, $\Sigma\Delta f$ values equal to around 8% were obtained with no clear sensitivity to ρ_f. This supports what was found above in a).



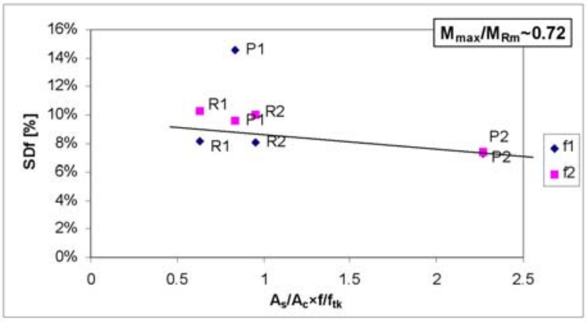


Fig. 2.13 $\Sigma\Delta f$ related to ρ for M_{max}/M_r≈1.25 **Fig. 2.14** $\Sigma\Delta f$ related to ρ_f for M_{max}/M_{Rm}≈0.72

- For the *prestressed beams*, the influence of internal, bonded prestressing and of its degradation as well as of cracking on the natural frequencies was analysed.

It was demonstrated that the application of internal, bonded prestressing had influence on the natural frequencies of beams even in the absence of any tendon- or crack-related damage.

For beams with negligible crack-related damage, it was found that natural frequencies were not sensitive to small degradation of internal, bonded prestress. However, degradation of internal, bonded prestress resulted in greater extent of natural frequency decrease for beams, on which intensive cracking formulated before the degradation in prestress started to develop, in comparison with beams being uncracked before the degradation process.

- By the application of *post-tensioning to reinforced concrete (non-prestressed) beams,* which had already been affected by significant crack-related damage, the influence of post-tensioning on the natural frequencies was analysed.

Since the opening of existing cracks could be considerably decreased by post-tensioning, gradual increase in bending stiffness of the already damaged beams was expected with the increase of the post-tensioning force. The tests demonstrated that for low and medium intensity of post-tensioning force, the measured natural frequencies (f_m) changed at a much higher rate than natural frequencies calculated on the basis of linear-elastic, second-order numerical calculations with the consideration of constant bending stiffness along the full length. This indicated that the existing crack-related damage of beams influenced f_m in much greater extent than the intensifying post-tensioning.

It was also demonstrated that cracking-induced bending stiffness decrease of beams could not be eliminated even by high-intensity post-tensioning if previously-occurred, intensive plastic deformation existed in the bars.

2.6 Practical application of frequency measurements on a highway bridge

The applicability of natural frequency measurements in practical damage identification and/or assessment was investigated on a four-span, reinforced concrete highway bridge. The load carrying structure was an 11.0 m wide voided slab built according to the conventional Gerber system (*Fig. 2.15*).

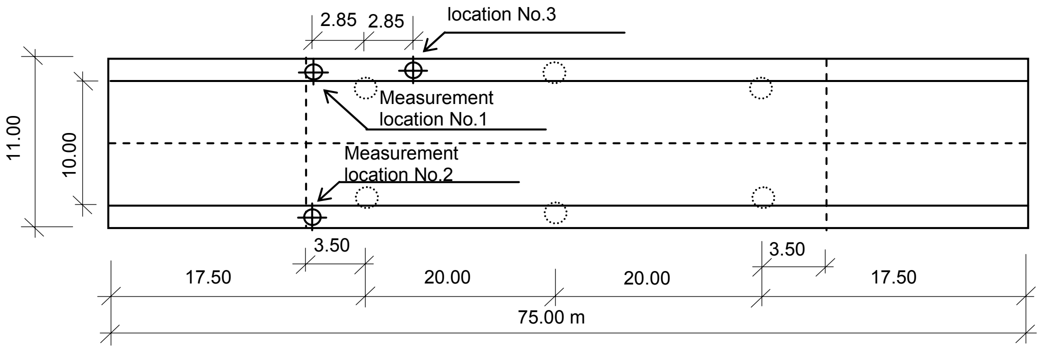


Fig. 2.15 Scheme of the investigated bridge and the measurement locations

The aim of the test was to verify that reliable determination of the first few natural frequencies of the structure was possible under the service load of the bridge. Thus, for excitation purposes the normal road traffic was used. The number of acceleration sensors, their positions on the structure, the length of the in-situ recorded *a-t* signals (~40 min.) and the settings of the applied measurement devices were chosen accordingly. The *a-t* signals were recorded simultaneously in the applied three measurement locations.

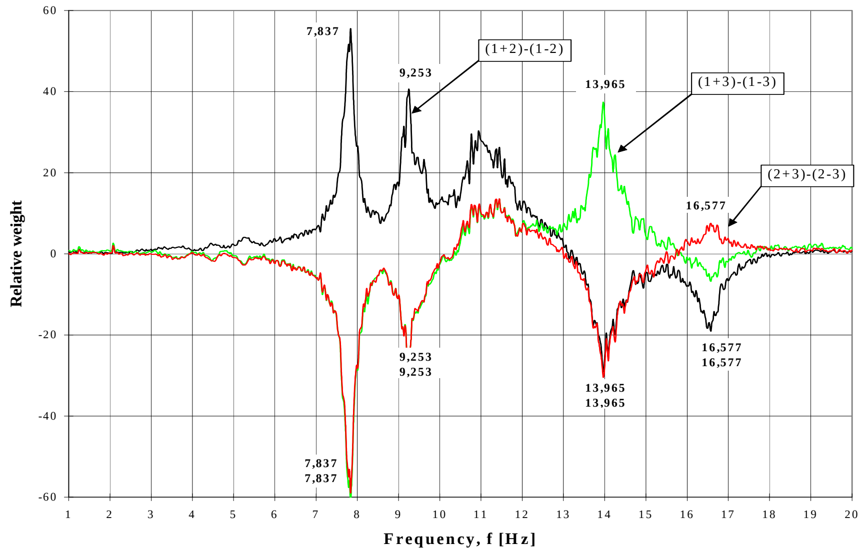


Fig. 2.16 Calculated phase spectrums as linear combination of average frequency spectrums

The excitation force proved to be sufficiently high and smooth over the target frequency range (5-20 Hz) of the first four natural frequencies of the bridge. Average frequency spectrums determined from sums and differences of *a-t* signals belonging to measurement locations of selected measurement location pairs were computed. To assess the measured modes, differences of these average frequency spectrums were calculated according to *Fig. 2.16* and resulted in phase spectrums. Whether the considered measurement locations vibrated in the same or in the opposite phase could be read from the sign of the belonging phase spectrum at the associated natural frequency. The use of phase spectrums made the reproduction of mode shapes associated with the determined natural frequencies possible.

3 PRACTICAL APPLICABILITY AND POSSIBLE FURTHER DEVELOPMENT OF THE RESEARCH

The introduced experimental, vibration-based damage identification and assessment method consisted of a natural frequency measurement technique and a crack-related damage assessment procedure. The measurement technique is available at low cost for intermittent or continuous observation of natural frequencies of reinforced, prestressed and post-tensioned concrete structures. Its application is mainly intended for girder-type bridge structures, for which normal road traffic can be used for excitation purposes. Using the relationships established between the measured natural frequencies and various performance-based structural parameters, any change in the structural performance of the investigated structure during the service life can be detected, localized and quantified on the basis of measured changes in the natural frequencies.

Further development of the introduced method is possible in two main directions. The first is the expansion of the range of measured dynamic characteristics in order to increase the information content of on-site measurements on dynamic behaviour. This is recommended only if the required effort (cost and time) of the on-site measurements can be kept within reasonable limits. From this point of view, the inclusion of damping parameters in this range is recommended. The second direction is either the further improvement of the existing relationships between the measured dynamic characteristics and the structural performance, or the definition of new ones. In this area, the conducting of sensitivity studies, the provision of a statistical basis behind the relationships as well as the extension of the concept of damage beyond flexural cracks is desirable.

4 NEW SCIENTIFIC RESULTS

4.1 New scientific results in English

New result 1

Based on a literature review, It was concluded that the existing dynamic-based damage identification methods focus mainly on detection, localization and sometimes quantification of damage but do not closely address structural performance-related issues, which limits their practical applicability.
To overcome this, a new damage identification level (Level 3a) was defined. Beyond damage identification, assessment methods fulfilling Level 3a are able to provide information on the structural performance. It was pointed out that the effective practical applicability of these methods requires:

a) the definition of performance-related damage indices that are based on previous static models and able to identify damage as well as to measure its extent (identification);
b) the establishment of relationships between the measured dynamic parameters and the performance-related structural properties (assessment).

Related publications: [1],[3],[4],[5],[6],[7],[8],[9],[10],[14],[15],[17]

New result 2

Cracking-induced deterioration processes were modelled in experimental conditions on simply supported, reinforced and prestressed concrete beams under flexure. Meanwhile the first two natural frequencies of beams were determined at gradually increasing damage levels. Damage was modelled by:

a) gradually intensified cracking due to defined, four-point mechanical loading for the reinforced beams;
b) the combination of a) and artificially-made tendon cuts for the prestressed beams;
c) the application of external post-tensioning (simulated strengthening effect) to a few, already deteriorated reinforced concrete beams.

The extent of damage was measured by the extent of cracking and by the number of tendon cuts. Alternatively, single mechanical impact and a periodic signal were used for excitation purposes during the natural frequency measurements.

Related publications: [1],[3],[4],[5],[6],[7],[9],[10],[11],[12],[13],[14],[15],[17],[18]

New result 3

A computational procedure for the evaluation of numerically-recorded acceleration-time signals was developed. By the use of this, the first two natural frequencies of the test beams were determined. The procedure was based on the statistical analysis of discrete Fourier-transformed individual frequency spectrums. Using this technique, the following were pointed out:

a) Both the average frequency spectrum (average spectrum method) and the average value of frequencies associated with the maximum ordinate in the individual spectrums (average frequency method) were solely able to indicate the first two natural frequencies with an acceptably low (<1%) coefficient of variation for the states of the beams in which the bending stiffness distribution remained symmetric along the beam length.
b) For states where the bending stiffness distribution became non-symmetrical due to a non-symmetrical crack pattern, multiple peaks occurred in the relevant frequency spectrums. The value of the standard deviation associated with the average frequency clearly indicated the presence of multiple peaks. If one of them was dominant, then the combined use of the average spectrum method and the average frequency method was required to estimate the true natural frequency as the average of the frequencies associated with sub-peaks.
c) The first two natural frequencies could reliably (with a relative difference less than 1.5%) be determined from vibration signals derived from either impulse or periodic excitation input, if no multiple peaks existed in the related frequency spectrums. If multiple peaks existed, then the relative difference increased significantly (>>1.5%).

Related publications: [2],[6],[8],[16],[19]

New result 4

Static-based damage indices which reflected cracking-related damage and were in accordance with a) of New result 1 were defined as follows:

- Growth of “total length of cracks”
- Growth of total and residual deflection at midspan
- Growth of crack width at midspan
- Growth of “total of crack sections”
- Growth of calculated midspan deflection
- Growth of calculated crack width at midspan
- Growth of internal strain energy

Their values were determined for all deterioration states of the test beams. Their classification was based on:

- their information content on the structure (global or local indices);
- the absence or the presence of any calculation model behind the index itself and their involved parameters (non-model-based (measured) or model-based (calculated)).

Depending on their intended purpose in damage identification, the indices were interpreted in both incrementative (for detection purposes) and cumulative (for quantification purposes) forms. Successful conformity checks were carried out between the corresponding non-model-based and model-based indices.

Related publications: [1],[3],[4],[5],[6],[7],[9],[10],[11],[12],[13],[14],[15],[17],[18]

New result 5

By the use of the defined damage indices, damage identification as well as assessment of test beams were carried out.

The defined indices were able to detect and to quantify the introduced artificial, crack-induced damage in the beams. The identified damage was confirmed by the measured natural frequency shifts.

Within the assessment, numerical relationships were established between the natural frequency shifts and:

- for the reinforced concrete beams
  - the internal force level;
  - the extent of cracking immediately after the occurrence of first cracks;
  - the type and amount of reinforcement;
- for the prestressed beams
  - the internal force level;
  - the extent of degradation in prestress;
- for the post-tensioned beams
  - the intensity of the post-tensioning force.

Related publications: [5],[15],[17]

New result 6

An in-situ frequency measurement and the respective signal evaluation were carried out on an operational reinforced concrete highway bridge. The aim of the test was to verify that reliable (with low standard deviation) determination of the first few natural frequencies of the structure was possible under the service load of the bridge. For excitation purposes the normal road traffic was used. The acceleration-time vibration signals were recorded simultaneously in three measurement locations. It was found that:

a) the excitation force of the normal road traffic was sufficiently high and smooth over the target 5-20 Hz frequency range of the first four natural frequency of the bridge. By the use of 34 min. long in-situ recorded acceleration-time signals at each measurement location, this frequency range could be covered by discrete Fourier transformed frequency spectrums with 0.02 Hz resolution;
b) the phase spectrums, computed as differences of average frequency spectrums determined from sums and differences of acceleration-time signals belonging to measurement locations of selected measurement location pairs, were capable of indicating the phase differences between the measurement locations of the selected pairs in each mode. This made it possible to reproduce the mode shapes associated with the measured natural frequencies.

Related publications: [2],[6],[8],[16],[19]

4.2 New scientific results in Hungarian

1. tézis

Irodalmi elemzés alapján rámutattam, hogy a meglévő, vasbeton szerkezeteknél alkalmazott dinamikai alapú károsodásvizsgálati módszerek elsősorban a károsodás kimutatására, lokalizálására és esetenként a mértékének a meghatározására irányulnak, míg a szerkezet teljesítőképességével kapcsolatban kevés információval szolgálnak, emiatt gyakorlati alkalmazhatóságuk korlátozott.

Ez alapján egy új károsodásvizsgálati szintet (3a szint) határoztam meg. A 3a szintnek megfelelő módszerek a károsodás-kimutatáson túl alkalmasak a szerkezet erőtani állapotának az értékelésére is. Rámutattam, hogy e módszerek hatékony gyakorlati alkalmazáshoz szükség van

a) előzetes statikai modellen alapuló, az erőtani állapottal összefüggő károsodási indexek meghatározására, melyek alkalmasak a károsodás kimutatására és a mértékének a közelítő meghatározására (károsodás-kimutatás), továbbá
b) a szerkezet mért dinamikai és erőtani jellemzői közötti összefüggések meghatározására (károsodás-értékelés).

Kapcsolódó publikációk: [1],[3],[4],[5],[6],[7],[8],[9],[10],[14],[15],[17]

2. tézis

Kísérlettel modelleztem egy repedezettségen alapuló károsodási folyamatot hajlított, kéttámaszú, vasbeton és feszített vasbeton gerendákon, melynek során folyamatosan növekvő károsodási szinteken meghatároztam a gerendák első két önrezgésszámát. A károsodást

a) vasbeton gerendák esetén négypontos elrendezésű mechanikai terheléssel létrehozott, fokozatosan növekvő mértékű repedezettséggel;
b) feszített vasbeton gerendák esetén az a) szerinti terhelés és feszítőbetét-elvágások kombinált alkalmazásával;
c) néhány, előzetesen károsodott vasbeton gerenda esetén külső vezetésű utófeszítéssel (megerősítés szimulációja)

modelleztem. A károsodás mértékét a repedezettség mértékével és a feszítőbetét-elvágások számával jellemeztem. Az önrezgésszám-mérésekhez alternatív módon impulzusszerű és periodikus gerjesztést alkalmaztam.

Kapcsolódó publikációk: [1],[3],[4],[5],[6],[7],[9],[10],[11],[12],[13],[14],[15],[17],[18]

3. tézis

Kidolgoztam egy numerikusan rögzített, gyorsulás-idő jelsorozat feldolgozására alkalmas számítási eljárást, melyet felhasználtam a vizsgált gerendák első két önrezgésszámának meghatározásához. Az eljárás diszkrét Fourier-transzormációval meghatározott frekvenciaspektrumok statisztikai elemzésén alapul. Ez alapján kimutattam, hogy:

a) mind az átlagos frekvenciaspektrum (átlagspektrum-módszer), mind az egyes frekvenciaspektrumok legnagyobb ordinátájához tartozó frekvenciák átlagaként meghatározott frekvenciaérték (átlagfrekvencia-módszer) önmagában alkalmas a gerendák első két önrezgésszámának elfogadhatóan kis (<1%) relatív szórással történő meghatározására mindazon károsodási állapotokban, ahol a hajlítási merevség eloszlása a gerenda hossza mentén szimmetrikus marad.
b) Azon károsodási állapotokban, ahol a hajlítási merevség eloszlása a nem szimmetrikus repedéskép miatt nem marad szimmetrikus, a frekvenciaspektrumban többszörös csúcsok jelennek meg. Az átlagfrekvenciához tartozó szórás mértéke egyértelműen jelzi a többszörös csúcsok jelenlétét. Ha a csúcsok egyike domináns a többihez képest, akkor az átlagspektrum-módszer és az átlagfrekvencia-módszer kombinált alkalmazására van szükség a lokális csúcsokhoz tartozó frekvenciák átlagával közelített önrezgésszám meghatározásához.
c) Ha a frekvenciaspektrumokban nincsenek többszörös csúcsok, akkor az első két önrezgésszám megbízhatóan (<1,5% relatív különbséggel) meghatározható mind impulzusszerű, mind periodikus gerjesztés alkalmazásával. Ha többszörös csúcsok jelentkeznek, akkor a relatív különbség jelentősen megnő (>>1,5%).

Kapcsolódó publikációk: [2],[6],[8],[16],[19]

4. tézis
A repedezettség miatti károsodás vizsgálatára alkalmas, statikai jellemzőkön alapuló, következő károsodási indexeket definiáltam az 1. tézis a) pontjával összhangban, majd osztályoztam azokat és meghatároztam a vizsgált gerendák károsodási állapotaihoz tartozó értékeiket:

- Összegzett repedéshossz-növekmény,
- Mezőközépi teljes és maradó lehajlás-növekmény,
- Mezőközépi repedéstágasság-növekmény,
- Összegzett repedésfelület-növekmény,
- Számított mezőközépi teljes lehajlás-növekmény,
- Számított mezőközépi repedéstágasság-növekmény,
- Alakváltozási energia-növekmény.

Az osztályozás szempontjai a következők voltak:

- a szerkezettel kapcsolatos információtartalom mennyisége (globális vagy lokális index),
- az index vagy az abban foglalt egyéb paraméterek meghatározásához alkalmazott számítási modell hiánya vagy megléte (nem modell alapú (mért) vagy modell alapú (számított) indexek).

A károsodás kimutatásában betöltött szerepüktől függően a károsodási indexeket növekményes (a károsodás felismeréséhez) és halmozott (a károsodás mértékének meghatározásához) formában is meghatároztam. A nem modell alapú és a modell alapú indexeket sikeres megegyezőségi vizsgálatoknak vetettem alá.

Kapcsolódó publikációk: [1],[3],[4],[5],[6],[7],[9],[10],[11],[12],[13],[14],[15],[17],[18]

5. tézis
A károsodási indexek felhasználásával a vizsgált gerendákon károsodás-kimutatást és károsodás-értékelést végeztem.
Az alkalmazott indexek alkalmasnak bizonyultak a mesterségesen létrehozott, repedezettség miatti károsodás felismerésére és a mértékének a becslésére. A kimutatott károsodást a mért önrezgésszám-változások igazolták.
A károsodás-értékelés során számszerű összefüggéseket határoztam meg az önrezgésszám-változások mértéke és

- vasbeton gerendák esetén
  - az erőtani kihasználtság,
  - az első repedések létrejöttekor kialakuló repedezettség mértéke,
  - a vasalás típusa és mennyisége,
- feszített vasbeton gerendák esetén
  - az erőtani kihasználtság,
  - az feszítőerő leépülésének mértéke,
- utófeszített vasbeton gerendák esetén
  - a feszítőerő mértéke

között.

Kapcsolódó publikációk: [5],[15],[17]

6. tézis
Helyszíni frekvenciamérést végeztem és végrehajtottam a hozzá tartozó jelfeldolgozást egy vasbeton közúti híd esetén. A vizsgálat célja annak igazolása volt, hogy a szerkezet első néhány önrezgésszáma megbízhatóan (kis szórással) meghatározható a híd forgalmának fenntartása esetén is. Gerjesztő hatásként a szokásos közúti forgalmat használtam. A gyorsulás-idő jelsorozatot egyidejűleg három mérőhelyen rögzítettem. Kimutattam, hogy

a) a közúti forgalomból származó gerjesztő hatás elegendően nagy intenzitású és egyenletes eloszlású volt a híd első négy önrezgésszámát tartalmazó 5-20 Hz közötti frekvenciatartományban. Mérőhelyenként 34 perc hosszúságú, helyszínen rögzített gyorsulás-idő jelsorozat felhasználásával az előirányzott frekvenciatartomány 0,02 Hz felbontású, diszkrét Fourier-transzformált frekvencia-spektrumokkal lefedhető volt.
b) a mérőhely-páronként előállított gyorsulás-idő jelsorozat-összegekre és –különbségekre megszerkesztett átlagos frekvenciaspektrumok különbségeiként meghatározott fáziskép-spektrumokkal kimutathatók voltak a mérőhely-párokhoz tartozó mérőhelyek fáziskülönbségei az egyes módusokban. Ez lehetővé tette a mért önrezgésszámokhoz tartozó sajátalakok reprodukálását.

Kapcsolódó publikációk: [2],[6],[8],[16],[19]

5 RELATED PUBLICATIONS

Book, sections in book

[1] Kovács, T. – Farkas, Gy.: Condition monitoring of reinforced concrete beams by dynamic measurements, Sec. 3.5.4. in book entitled as *New Materials, systems, methods and concepts for prestressed concrete structures*, final report of COST 534, Eds: R.B. Polder, M.C. Alonso, D.J. Cleland, B. Elsener, E. Proverbio, Ø. Vennesland, A. Raharinaivo, Cost Office, 2009, ISBN 9789059863323, pp. 192-194.

Paper in collected volume

[2] Kovács T. - Farkas Gy. – Kálló M.: Vasbeton hidak dinamikus jellemzőinek vizsgálata, *BME Vasbetonszerkezetek Tanszéke Tudományos Közleményei* (szerk.: Tassi G., Kovács T.), 1998., Műegyetemi Kiadó, Budapest, pp. 111-124.

[3] Kovács T. – Farkas Gy.: Károsodásbecslés dinamikai jellemzők alapján, *BME Hidak és Szerkezetek Tanszéke Tudományos Közleményei* (szerk.: Tassi G., Hegedűs I., Kovács T.), 2001., Műegyetemi Kiadó, Budapest, pp. 125-138.

[4] Kovács, T. – Farkas, Gy.: Damage assessment by frequency measurements on reinforced concrete beams, *Workshop of COST on „NDT Assessment and New Systems in Prestressed Concrete Structures*”, COSt Action 534, Institute for Terotechnology – NRI, Kielce, Poland, September 19-21, 2005. pp. 201-216.

[5] Kovács, T.: Natural frequency shifts for concrete beams due to deterioration-induced cracks, *Scientific publicatons of the Dept. of Structural Engineering,* (ed.: Tassi, G., Hegedűs, I., Kovács, T.), 2009., Műegyetemi Kiadó, Budapest, pp. 115-122.

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[12] Kovács, T.: The effects of cracking on the natural frequencies for concrete beams, *Proc. of the 4th International PhD Symposium in Civil Engineering*, Munich, 19-21 September 2002, Volume 1, pp. 338-344.

[13] Kovács, T. – Farkas, Gy.: *Condition monitoring of concrete beams by dynamic measurements*, *Proc. of the IABSE Symposium on „Responding to tomorrow’s challenges in structural engineering*”, Budapest, Hungary, September 13-15 2006, pp. 112-113, CD-ROM

[14] Kovács, T. – Farkas, Gy.: Damage assessment by dynamic measurements on concrete bridges, *Proc. of the International Conference on Bridges*, Dubrovnik, Croatia, May 21-24 2006, pp. 1067-1074.

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