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Experimentální analýza dynamického chování stavebních konstrukcí

Experimental Analysis of Dynamic Behaviour of Building Structures

Summary

Building structures are usually unique. They differ one from another in dimension, shape, construction systems, used materials or soil properties in bottom of a footing. Therefore the questions of experimental examination of building structures are generally based on individual approach to each of the tested structures. Every structure, especially if the unique supporting system or new building material is used, may cause unexpected static or dynamic problems during its design, construction or its operation, which must finally be solved experimentally.

It is important to set every experiment in such a way so that obtained results will be specific, meaningful and useful and not just purposeless. It is necessary to analyze the expected static or dynamic behaviour of the structure in the experiment preparation on the basis of theoretical analysis and practical experience. According to the analysis results, it is important to identify decisive parameters of investigated structure, which are suitable and useful to monitor experimentally, to select the appropriate set of measurement equipment, to determine procedure of data acquisition and results evaluation. Every experiment is loaded by an error and it is essential to quantify the uncertainty of measurement results to interpret the data correctly.

A great number of experiments can be understood under the term experimental analysis of dynamic behaviour of structures. For practical applications in the questions of structural dynamics the characteristics of natural vibration of building structure (natural frequencies, natural modes and damping) or parameters of forced vibration (when external excitation forces affect the building structure) are usually measured. In this publication several questions of building structures are described in which the dynamic experiments are important or even essential. The presented dynamic experiments were solved during the last ten years by the author and they are especially interesting in terms of results, uniqueness of the tested structures or measurement system.

Prestressing force in suspenders and cables is an important parameter of the load bearing structure. On the current structure, prestressing forces can be determined most reliably by an experiment. The frequency method is one of often used experimental techniques in which the measured natural frequencies are used. This method provides results accurate enough if the suitable setting of experiment and method of evaluation are designed. Characteristics of natural vibration of a structure determined by an experiment describe their immediate dynamic behaviour. Therefore, they can be used for verification and identification of mathematical models, especially if these models are used for dynamic computations. Characteristic of natural vibration can be used also for detection and localization of structural damage. Methods and procedures used for determination of degradation degree of a structure are suitable to verify on simple structural elements where the level of their damage is known. The study describes the experimental analysis in which the influence of the intensive cyclic fatigue loading on characteristics of natural vibration of the fully prestressed concrete slabs was observed.

The task of dynamic experiment is also important in cases when significant dynamic loading acts on structure which causes an expressive transient vibration of the structure and which has random character difficult to describe by analytical relations. In this publication, questions of forced vibration of structures are documented on three examples of important footbridges where dynamic experiment eminently contributed to verification and ensuring their reliability and below it is documented on experiment focused on long-time monitoring of the road bridge response caused by heavy traffic.

Souhrn

Stavební konstrukce jsou obvykle unikátní. Navzájem se liší svými rozměry, tvary, konstrukčními systémy, použitým materiálem nebo uložením v základové spáře. Proto je problematika experimentálního vyšetřování stavebních konstrukcí většinou založena na individuálním přístupu ke každé zkoušené konstrukci. Každá konstrukce, především pokud je použit originální nosný systém nebo nový materiál, může přinést v průběhu návrhu, výstavby nebo jejího provozu neočekávané statické a dynamické problémy, které musí být nakonec řešeny pomocí experimentálního zkoumání.

Je důležité každý experiment uspořádat tak, aby získané výsledky byly konkrétní, smysluplné a prakticky využitelné a ne pouze samoúčelné. Proto je potřebné při přípravě experimentu na základě teoretické analýzy a praktických zkušeností provést rozbor očekávaného statického nebo dynamického chování konstrukce. Podle výsledků rozboru určit rozhodující parametry vyšetřované konstrukce, které je vhodné a účelné experimentálně sledovat, vybrat sestavu vhodných měřicích prostředků, stanovit postup zpracování a hodnocení výsledků. Každý experiment je zatížen chybou, pro správnou interpretaci získaných údajů je podstatné kvantifikovat nejistotu výsledků měření.

Pod pojem experimentální analýza dynamického chování stavebních konstrukcí lze zahrnout celou řadu experimentů. Při praktických aplikacích v úlohách stavební dynamiky se obvykle zjišťují charakteristiky vlastního kmitání stavební konstrukce (vlastní frekvence, vlastní tvary a útlum) nebo parametry vynuceného kmitání, při kterém na stavební konstrukci působí vnější budící síly. V této práci je popsáno několik problematik stavebních konstrukcí, ve kterých je role dynamických experimentů důležitá až nezbytná. Uvedené dynamické experimenty byly autorem řešeny v posledních deseti letech a jsou zajímavé zejména z hlediska získaných výsledků, výjimečnosti zkoušených konstrukcí nebo originality uspořádání měření.

Předpínací síly v závěsech a kabelech jsou důležitým parametrem nosné konstrukce stavby. Na stávající konstrukci lze předpínací síly stanovit nejspolehlivěji pomocí experimentu. Jedním z používaných experimentálních postupů je frekvenční metoda, ve které jsou využívány změřené vlastní frekvence. Tato metoda při vhodném uspořádání experimentu a způsobu vyhodnocení poskytuje dostatečně přesné výsledky. Změřené charakteristiky vlastního kmitání konstrukce popisují její okamžité dynamické vlastnosti, proto je lze s výhodou využít pro verifikaci a identifikaci matematických modelů. Zejména tehdy, pokud tyto modely mají být použity k dynamickým výpočtům. Charakteristiky vlastního kmitání je možné využít také při detekci a lokalizaci poškození konstrukce. Metody a postupy, které jsou používány ke stanovení stupně degradace konstrukce, je vhodné verifikovat na jednoduchých konstrukčních prvcích, u kterých je známá úroveň jejich poškození. V práci je popsána experimentální studie, při které byl sledován vliv intenzivního cyklického únavového namáhání na charakteristiky vlastního kmitání desek z plně předpjatého betonu.

Úloha dynamického experimentu je důležitá také v případech, kdy na stavební konstrukci působí významné dynamické zatížení, které vyvolává výrazné přechodové kmitání konstrukce a které má nahodilý charakter obtížně popsatelný analytickými vztahy. Problematika vynuceného kmitání stavebních konstrukcí je v práci dokumentována na třech příkladech významných lávek pro pěší, u kterých dynamický experiment významně přispěl k ověření a zajištění jejich spolehlivosti, a na experimentu zaměřeném na dlouhodobé monitorování odezvy silničního mostu na účinky těžké nákladní dopravy.

Klíčová slova:
Experimentální analýza, dynamické chování, monitoring, vlastní frekvence, vlastní tvar, tlumení, vynucené kmitání, stavební konstrukce, lávka pro pěší, most, verifikace modelu, identifikace modelu, těžká nákladní doprava, únava.

Keywords:
Experimental analysis, dynamic behaviour, monitoring, natural frequency, natural mode, damping, forced vibration, building structure, footbridge, bridge, model verification, model identification, heavy traffic, fatigue.

CONTENTS

1 INTRODUCTION

Hand in hand with the great development of computer technology in the last three decades, the massive development of computational methods and programs which make possible to solve extensive and complicated engineering problems have occurred. Nowadays, computations enter into domains in which experiment still has been dominating recently. The question is: “Has the experiment meaning at present time?” The answer to above mentioned question is: “Yes, it has!” The experiment investigates existing reality. Its role is important mainly for determination of material properties, for investigation of static and dynamic parameters of current structures, for observing quantity of climatic loading or for assessment of dynamic loading effects with random character.

In this publication several problems of building structures are described in which the role of dynamic experiments is important and necessary. Introduced dynamic experiments were solved by author during the last ten years and they are especially interesting in terms of results, uniqueness of the tested structures or originality of measuring arrangement.

2 THE EXPERIMENTAL INVESTIGATION OF NATURAL VIBRATIONS

The characteristics of natural vibrations (natural frequencies, natural modes and corresponding damping) are the natural properties of an investigated structure.

The experimentally obtained characteristics of natural vibrations of a structural element or a structure describe its immediate dynamic characters affected only its immediate stiffness, mass and structural setting.

2.1 THE IDENTIFICATION OF CABLE FORCES IN THE ROOF STRUCTURE OF THE ADMINISTRATIVE CENTRE AMAZON COURT

There are a number of building structures in service in Czech Republic at present time where the main structural element is a cable or a suspender, for example: cable roofs, cable-stayed bridges and bridges with external prestressing cables. Knowledge of value of cable tension force is important for appreciation of reliability both during their construction and their operation.

The three categories of experimental techniques are applied for monitoring of cable forces in practice. One is the method that directly measures the cable forces by pre-installed load cell, the second is the magnetoelastic method and the third is the vibration frequency method that indirectly evaluates the cable tension force using the measured natural frequencies. The vibration frequency method is often used in practice (e.g. [9]) because it provides an efficient, cheap and relatively easy way to determine the cable forces and a standard measuring line for dynamic experiments can be used.

The typical procedures of the vibration frequency methods are as follows. At first, the excitation of cable vibrations is carried out for example by the help of an impact hammer or ambient sources such as wind and technical seismicity. Secondly, the time waveform of cable free vibration is measured. Next, several lowest natural frequencies are extracted from the measured data by the help of Fast Fourier Transform (FFT) and various modal analysis techniques. Finally, the cable tension forces are determined by using an appropriate relationship between natural frequencies and cable tension forces.

The relationship between frequencies and cable tension forces that will be used during results evaluation of an experiment is dependent on parameters of examined cables and on theoretical presumption of the bending stiffness and the boundary conditions of the cable supporting.

The Administrative Centre Amazon Court is situated in Prague - Karlin. The atrium of this building is covered by a pneumatically controlled roof on a steel structure. V-shaped bracing struts are placed on the system of steel cables stretched across the atrium (Fig. 1). The roof is covered with belts of triple-layer ETFE foil, stretched between purlins and inflated. ETFE foil is transparent, not flammable, leakproof, and heat resistant up to 270°C.

Figure 1: The view on the steel structure of the roof of Administrative Centre Amazon Court

The cable tension force was not observed during installation of the steel structure of the roof. The cable anchoring structure did not make possible to rectify the forces in cables additionally.

After finishing installation of the roof in August 2008, the designer wanted to verify forces in individual cables of the roof and if there was made sufficient tension force reserve for loading the roof by wind uplift. Precision of results was required less than 10%. The vibration frequency method was selected as very suitable and precise enough.

The vibration transducers installed on the cable are shown in the Fig. 2. The Fig.3 shows the example of the measured cable free vibration and the corresponding frequency spectrum. The experimentally evaluated natural frequencies $f_{(j)}$ for selected cables are mentioned in Tab. 1. For evaluation of prestressing forces in investigating cables of the roof structure, these theoretical models of cables were used:

- the string model,
- the simply supported beam,
- the fixed beam.

The string theory was used by experiment in situ only for the first approximation, because it neglects influence both bending stiffness and boundary conditions.

The relationship which utilizes the simply supported beam was used consequently for identification of the cable bending stiffness EI and the cable tension force N.

$$N + \left(\frac{j\,\pi}{L} \right)^2 EI = \mu \left(\frac{2\, f_{(j)}\, L}{j} \right)^2 \tag{1}$$

where μ is the mass per meter of the cable length, L is the length of the cable, and $f_{(j)}$ is the j^{th} natural frequency. The mass μ was retired from producer's catalogue of used cables, length L was measured during the experiment in situ.

The five lowest natural frequencies were analyzed for each cable of the roof structure. Based on the relation (1), it was possible to make five equations for two unknowns.

$$\begin{bmatrix} 1 & \left(\frac{1\,\pi}{L} \right)^2 \\ \vdots & \vdots \\ 1 & \left(\frac{j\,\pi}{L} \right)^2 \\ \vdots & \vdots \\ 1 & \left(\frac{n\,\pi}{L} \right)^2 \end{bmatrix} \begin{Bmatrix} N_{PŘ,ID} \\ EI_{ID} \end{Bmatrix} = \begin{Bmatrix} \mu \left(\frac{2\, f_{(1)}\, L}{1} \right)^2 \\ \vdots \\ \mu \left(\frac{2\, f_{(j)}\, L}{j} \right)^2 \\ \vdots \\ \mu \left(\frac{2\, f_{(n)}\, L}{n} \right)^2 \end{Bmatrix} \tag{2}$$

This equation system can be solved using Gauss Markov theorem.

$$[A]^T [A] \{x\} = [A]^T \{y\} \tag{3}$$

Natural frequencies and corresponding natural modes were measured on two selected cables. It resulted from evaluated data that real boundary conditions of the cable supporting had the character somewhere between fixation and hinged support. Therefore prestressing forces which were determined based on the simply supported beam model and based on the fixed beam model create bounds between which the real value of the prestressing force is.

From the resulting values of the cable tension forces that are shown in Tab. 2, it is evident that cables are very differently stressed even if the parameters of observed cables are practically the same. The maximal evaluated cable tension force (142.5 kN) is almost three times higher than the minimal cable tension force (49.3 kN).

Figure 2:The view on the vibration transducers (accelerometers) installed on the cable.

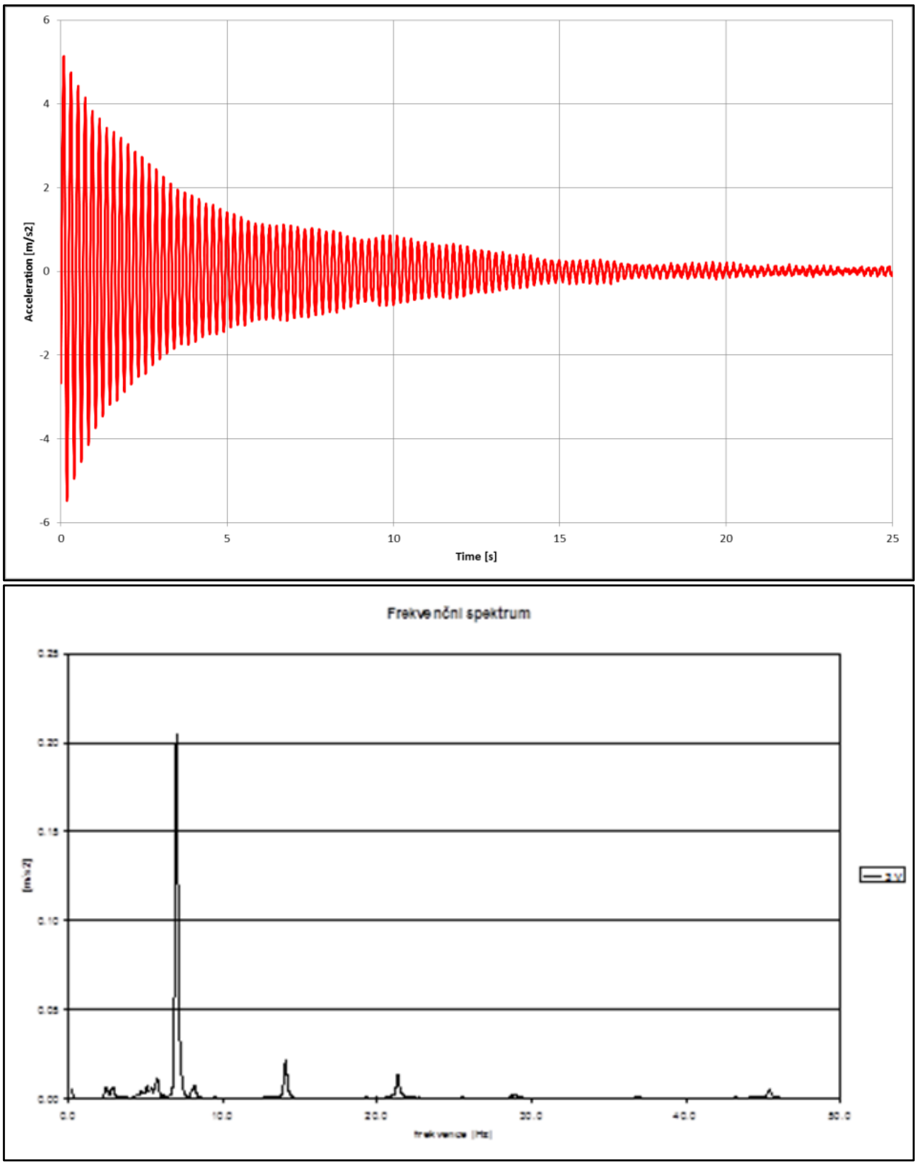


Figure 3: The example of the measured cable free vibration and the corresponding frequency spectrum.

Table 1: The experimentally evaluated natural frequencies $f_{(j)}$ for selected cables

Cable No.	Cable length [m]	j - Number of the natural frequency 1 [Hz]	2 [Hz]	3 [Hz]	4 [Hz]	5 [Hz]	Cable temperature [^{o}C]
4	10.860	7.00	14.19	21.41	29.13	37.19	35.9
5	10.885	8.34	16.81	25.38	34.28	43.56	36.1
6	10.880	7.13	14.47	21.88	29.63	37.84	36.1
7	10.870	5.03	10.22	15.63	21.44	27.88	36.1
10	10.880	7.31	14.69	22.25	30.16	38.56	35.0
11	10.890	6.44	13.03	19.88	26.94	34.53	37.3
12	10.880	7.41	14.97	22.59	30.75	39.13	37.3
13	10.890	8.09	16.41	24.78	33.47	42.50	37.3

Table 2: The determined tension forces N for selected cables

Cable No.	The theoretical model of cable: Simply supported beam [kN]	Fixed beam [kN]	Resulting values of the cable forces N: Value [kN]	Uncertainty [%]
4	103.7	94.3	**99.0**	± 4.8
5	148.0	136.9	**142.5**	± 3.9
6	108.3	98.7	**103.5**	± 4.6
7	52.8	45.8	**49.3**	± 7.1
10	112.7	102.9	**107.8**	± 4.5
11	88.2	79.5	**83.9**	± 5.2
12	116.6	106.7	**111.7**	± 4.5
13	140.5	129.7	**135.1**	± 4.0

2.1.1 Conclusions

The described experiment shows that production tolerances and mounting method can significantly influence the stresses in statically indeterminate load bearing structure with cables. Therefore, it is important to verify forces in cables experimentally in these structures and it is also suitable to monitor forces during their service life.

The vibration frequency method is very suitable for experiments done only one time or occasionally. This method provides results precise enough for suitable setting of an experiment and evaluation method.

2.2 THE DETECTION OF FATIGUE DAMAGE OF FULLY PRESTRESSED CONCRETE SLABS BY USING OF EXPERIMENTAL MODAL ANALYSIS

During last two decades traffic speed has increased, the number of vehicles has rapidly increased and they are much heavier. At the same time, new higher strength building materials are used and new structures are constructed more slender. This is why dynamic forces and fatigue loads caused problems on bridge structures. Determining the rate of degradation for existing building structures especially bridges has become important technical and also economic problem today.

New methods for monitoring of structure conditions and damage detection of a structure at the earliest possible stage are needed. Characteristics of natural vibration determined by an experiment (natural frequencies, natural modes and corresponding damping) describe its dynamic characters during implementation of the experiment. Therefore they can be used for detection and damage localization of a structure. The advantage of methods using results of an experimental modal analysis for estimation of a degradation degree of a structure is that they can be applied to complex structures.

Experimental methods and procedures used for determination of degradation degree of building structure are suitable to verify on simple structural elements where their damage state is known (e.g. [1], [4] and [6]).

Experimental study described in this chapter follows two similar long-term experiments that were focused on fatigue damage detection on reinforced concrete elements. The first test was carried out on the three reinforced concrete beams with rectangular cross section and the second test on four reinforced concrete slabs (e.g. [8]).

The cracks arose on reinforced concrete elements. They clearly influenced characteristics of natural vibration and they were seen with the naked eye already during the static loading, which had been considerably less than used dynamic fatigue load. So for next experiment, elements from fully prestressed concrete in which the cracks would not arise theoretically were used.

The objective of the experimental study described in this chapter was to investigate the influence of high intensity fatigue loading on the change of the modal characteristics of two fully prestressed concrete slabs (e.g. [11]).

2.2.1 Description of the fully prestressed slabs and the fatigue loading

For the purpose of this study, SMP CZ, a.s. made two fully prestressed concrete slabs (Fig. 4). The dimensions of the slabs were 130 x 1155 x 4500 mm with ends expanded to the height 400 mm for anchoring of the prestressing cables. The slabs were made from concrete C45/55 with eleven prestressing cables of diameter 15.7 mm. The slabs were put on two bearings to be a simply supported with the span 3500 mm with cantilevered ends 500 mm on both sides (Fig. 4).

The tested slabs were designed atypically compared to normally produced fully prestressed slabs. The centric prestressing was used to create great pressure reserve in the lower part of the slabs for application of high intensive cyclic loading.

The amplitude of the dynamic load was chosen to not satisfy the safety condition for the fatigue loading of the concrete according to ČSN EN 1992-1-1 and EN 1992-2

Figure 4: The view on the slabs concreting and the position of the slab in the loading stand.

$$\frac{\sigma_{c,max}}{f_{cd,fat}} \le 0.5 + 0.45\frac{\sigma_{c,min}}{f_{cd,fat}} \le 0.9 \qquad (4)$$

where $\sigma_{c,max}$ is the maximal compressive stress, $\sigma_{c,max}$ is the minimal compressive stress and $f_{cd,fat}$ is the design compressive strength of the concrete.

A rate of loading cycles that theoretically causes fatigue failure was determined by the relation mentioned in EN 1992-2 and [12]

$$N_i = 10\exp\left(14\left(\frac{1 - E_{cd,max,i}}{\sqrt{1 - R_i}}\right)\right) \qquad (5)$$

$$R_i = \frac{E_{cd,min,i}}{E_{cd,max,i}} \; ; \; E_{cd,min,i} = \frac{\sigma_{c,min,i}}{f_{cd,fat}} \; ; \; E_{cd,max,i} = \frac{\sigma_{c,max,i}}{f_{cd,fat}} \tag{6}$$

Total number of cycles that cause fatigue failure was theoretically determined to 2 240 000 cycles.

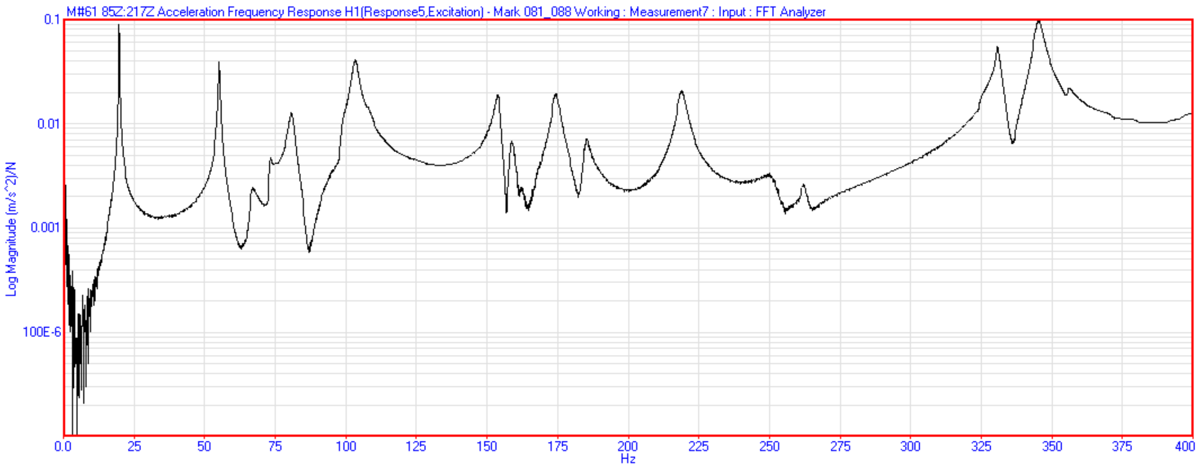


Figure 5: The example of the FRF in the point No.85 in the state of the slab No. 1 after 3 500 000 cycles.

Table 3: The changes of the selected natural frequencies of the slab No. 1

State	State description	$f_{(1)}$ [Hz]	$\Delta f_{(1)}$ [%]	$f_{(2)}$ [Hz]	$\Delta f_{(1)}$ [%]	$f_{(3)}$ [Hz]	$\Delta f_{(1)}$ [%]
A	Virgin state	20.06	-	55.24	-	98.07	-
F	State after 625 000 cycles	20.06	0.0	55.01	-0.4	97.88	-0.2
Q	State after 3 500 000 cycles	19.55	-2.6	55.09	-0.3	98.24	0.2

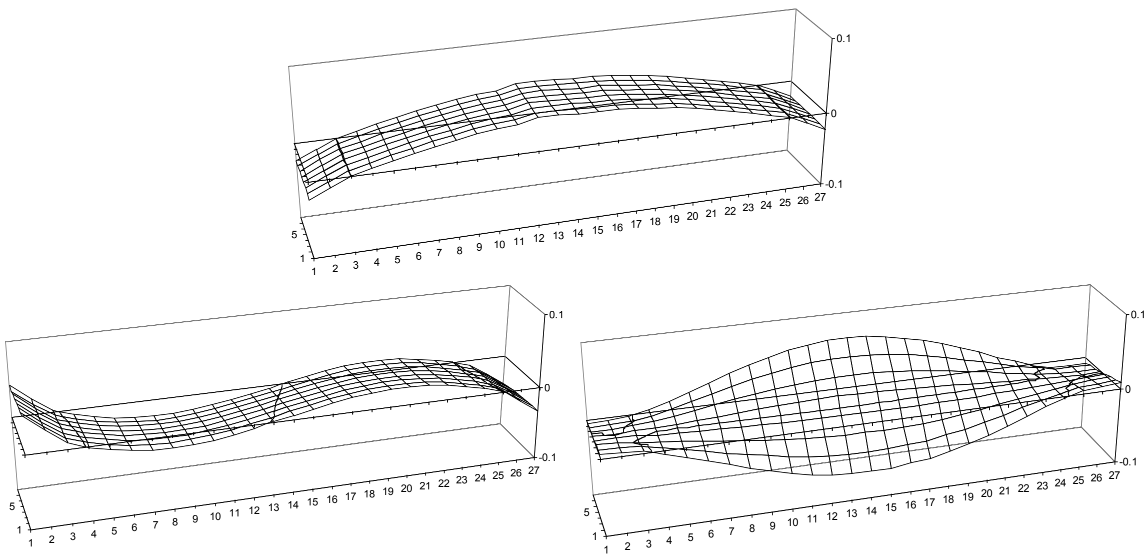


Figure 6: The 1st, the 2nd and the 3rd natural mode shapes of the slab No. 2 ($f_{(1)}$=19.3 Hz, $f_{(2)}$=53.9 Hz, $f_{(3)}$=97.1 Hz) in virgin state.

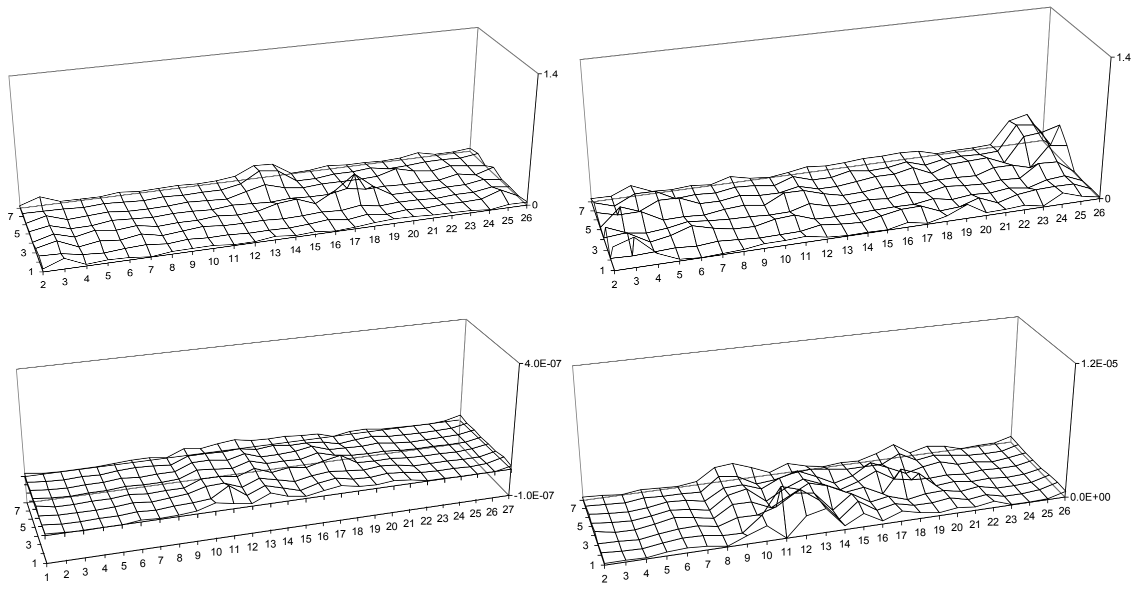


Figure 7: The CAMOSUC$_{(2)x}$ (above left), CAMOSUC$_{(3)x}$ (above right), $\Delta[\delta]$ (below left) and $\Delta[\delta]''$ (below right) between the virgin state A and the state Q after 3 500 000 loading cycles of the slab No. 1.

2.2.2 The natural vibration characteristics evaluation and comparison

The experimental study was carried out in laboratories of Faculty of Civil Engineering, CTU in Prague. The investigated fully prestressed concrete slab was loaded in four point bending test to get a constant bending moment in the mid-section of the slab. The fatigue load was induced by harmonic force with frequency 5 Hz. The dynamic cyclic loading was applied to the slab in several steps. The loading stopped after each 250 000 loading cycles and an experimental modal analysis was carried out subsequently. The experimental study was terminated after 3 500 000 loading cycles (156 % of theoretical cycles to fatigue failure) because of time reasons. The cracks were not indicated.

The Modal Exciter Type 2732 (Brüel & Kjaer) was used for the excitation of the prestressed concrete slab. The exciter was placed under the slab linked to the slab with the flexible drive rod. The exciter produced a random driving force over the frequency range of 5 to 400 Hz. The force transducer Endevco 2311-10 placed between the flexible rod and the slab measured the excitation force. The response of the slab onto forcing by the exciter was measured by eight piezoelectric acceleration transducers Brüel & Kjaer 4507 B 005 in a chosen net of points on the upper face of the slab (27 cross-sections and 8 points in each cross-section). The whole cross-section was measured at the same time. The point of excitation was designed to be able to excite basic bending modes of natural vibration of the slab. Values of the frequency response function (Fig. 5) were obtained as an average from ten measurements**.** The window length of the time signal processing was 32 sec, the frequency range of the window was set to 400 Hz.

The program MEscopeVES (Brüel & Kjaer) was used for evaluation of the natural vibration characteristics from measured frequency response functions. With regard to a frequency range of the dynamic analysis 5-400 Hz nine natural frequencies and mode shapes were evaluated. Examples of the natural mode shapes evaluated in a virgin state of the prestressed slab No. 2 are shown in Fig. 6.

Modal characteristics of the slab, which were measured after each load step, were mutually compared. Changes of the natural frequencies $\Delta f_{(j)}$ of the slab were computed at first. Next three methods (changes of a mode surface curvature $CAMOSUC_{(j),x}$ [4], changes of a modal flexibility matrix $\Delta[\delta]$ and the second derivative of changes of diagonal members of a modal flexibility matrix $\Delta[\delta]''$) were used for the comparison of natural modes.

The changes of a mode surface curvature $CAMOSUC_{(j),x}$ were calculated according to relation

$$CAMOSUC_{(j),x} = \left| \frac{r_{(j)XX,x+1} - 2r_{(j)XX,x} + r_{(j)XX,x-1}}{h^2} - \frac{r_{(j)YY,x+1} - 2r_{(j)YY,x} + r_{(j)YY,x-1}}{h^2} \right| \quad (7)$$

where $r_{(j),XX,x}$ is the value of the j-th natural mode shape in the x-th measured point in one damage (or virgin) state XX of the slab, $r_{(j),YY,x}$ is the value of the j-th natural mode shape in the x-th measured point in another damage state YY of the slab and h is the dimension of the net of measured points.

The modal flexibility matrix $[\delta]$ was evaluated from equation (e.g. [1], [2])

$$[\boldsymbol{\delta}] = [\mathbf{R}_{(j)}] \lfloor \mathbf{1/\omega}^2_{(j)} \rfloor [\mathbf{R}_{(j)}] \quad (8)$$

where $[R_{(j)}]$ is the modal matrix composed of n measured natural modes and $[1/\omega^2_{(j)}]$ is the diagonal matrix composed of natural angular frequencies $\omega_{(j)}$.

The second derivative of changes of diagonal members of a modal flexibility matrix $\Delta[\delta]''$ were calculated according to relation

$$\Delta\delta''_r = \frac{\Delta\delta_{r,x+1} - 2\Delta\delta_{r,x} + \Delta\delta_{r,x-1}}{h^2} \quad (9)$$

where $\Delta\delta_{r,x}$ is the change of the diagonal member r of the modal flexibility matrix and h is the distance of the net of measured points.

The examples of the evaluated changes of the slab No. 1 between virgin state and state after 3 500 000 cycles are shown in Tab. 3 and Fig. 7, some small changes can be seen in the middle part of the slab.

2.2.3 Conclusions

The influence of high intensity fatigue loading on the change of the natural vibration characteristics of two fully prestressed concrete slabs was investigated. The changes of natural frequencies (Tab.3) and natural modes (Fig. 7) evaluated during the experimental study were very small. These changes in dynamic behaviour are not significant even substantially after the end of theoretical fatigue lifetime.

The evaluated levels of changes of the natural vibration characteristics are hardly detectable on a real building structure from fully prestressed concrete in situ.

2.3 THE VERIFICATION AND THE IDENTIFICATION OF THE BRIDGE FEM MODELS BY USING OF EXPERIMENTAL MODAL ANALYSIS

The designer creates a mathematical model of a building structure on specific physical and mathematical conditions. So it is possible to create for one specific structure a number of mathematical models in terms of individual access of the designer. Mathematical models are possible to verify or identify on the basis of experimentally obtained results. Acquired experiences can be used consequently for modelling similar structures.

The term ‘model verification’ means check on appositeness of the mathematical model in terms of comparing between the calculation and experimental results. The model identification is determination of mathematical model parameters by using of results of an experiment so that a maximal consensus between calculated and measured data was achieved (e.g. [3]).

The experimentally obtained characteristics of natural vibration of a structure describe their immediate dynamic behaviour. Therefore, they can be used for verification and identification of mathematical models of new constructed or current structures especially, if these models are used for dynamic computations.

Several studies of dynamic behaviour of road bridges were done in the Czech Republic. This chapter describes results of an experimental and theoretical study of dynamic behaviour of the road bridge (e.g. [10]). An experimental modal analysis was carried out on this bridge as a basis for verification and for identification of the bridge FEM models.

Figure 8: The view on the prestressed concrete slab bridge near the village Heřmanova Huť.

2.3.1 Description of the investigated bridge

The investigated slab bridge (Fig. 8) made of prestressed concrete is situated across the highway D5 near the village Heřmanova Huť in the Czech Republic. It is a four span continuous bridge (14.0m + 18.5m + 18.5m +14.0m). The bearing structure of this bridge is a prestressed concrete slab of the width 9.8 m. The thickness of slab along the cross-section is 0.825 m in the mid-section 5.9 m long and 0.257 m at the ends of the cross-section. The change of the thickness is linear. It is a skew bridge with the angle 60.1963 deg.

2.3.2 The natural vibration characteristics evaluation

Experimental modal analysis (e.g. [2]) was carried out on this bridge. The electrodynamic shaker TIRAVIB 5140 was used for excitation of the bridge. The excitation force was measured by three force transducers S-35 LUKAS, which were interconnected to measure directly the whole driving force. The response of the bridge was measured by ten inductive accelerometers B12/200 HBM. Vibration control system 2550B Spectral Dynamics with control computer Sun was used for data acquisition and data analysis. The bridge was excited by random driving force of white type noise of the frequency range from 0 to 20 Hz. The driving force was controlled by signal generator SG 450 ONO SOKKI.

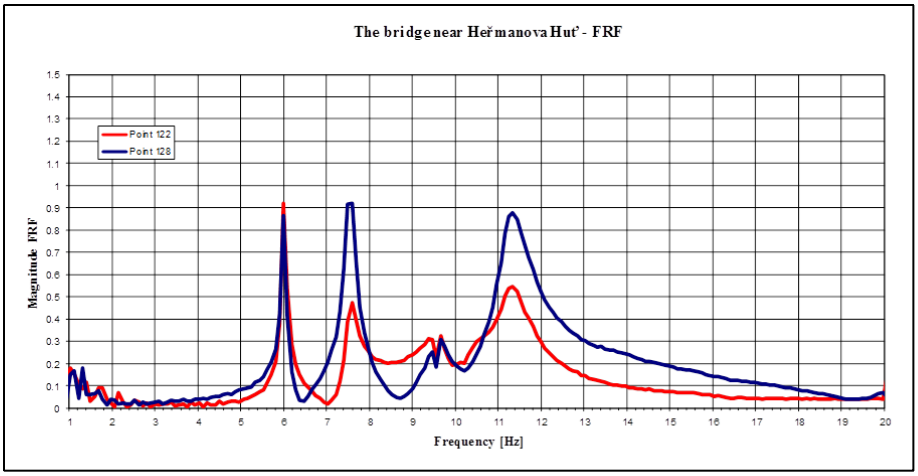


Figure 9: The example of two measured frequency response functions.

The response of the bridge was measured in a vertical direction in a chosen net of points (310 points – 31 cross sections and 10 points in each one) on the upper face of the bridge and in a longitudinal direction in the points on the longitudinal axis of the bridge.

Program STAR Spectral Dynamics was used for off line evaluation of natural frequencies and natural modes of vibration.

Six natural frequencies, mode shapes (Fig. 10) and damping frequencies were evaluated after the experimental modal analysis in the excitation range to 20 Hz.

The independence of the measured mode shapes was verified by MAC. Measured mode shapes are independent, the largest value of MAC is for mode shapes No. 3 and 6 ($MAC_{(3,6)}$=0,149).

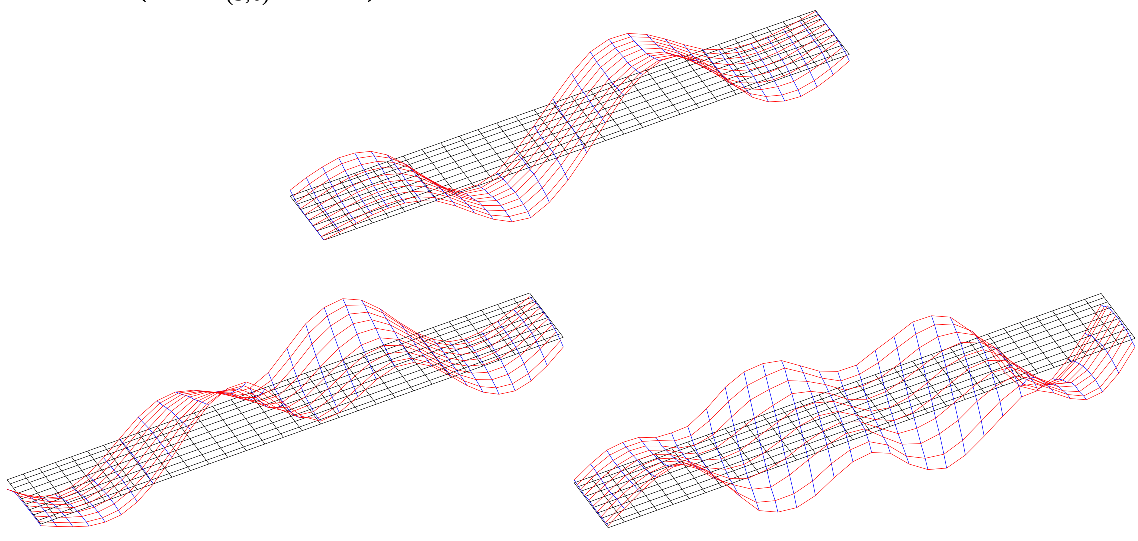


Figure 10: The 1st, the 2nd and the 3rd natural mode shapes of the bridge ($f_{(1)}$=6.02 Hz, $f_{(2)}$=7.56 Hz, $f_{(3)}$=9.69 Hz).

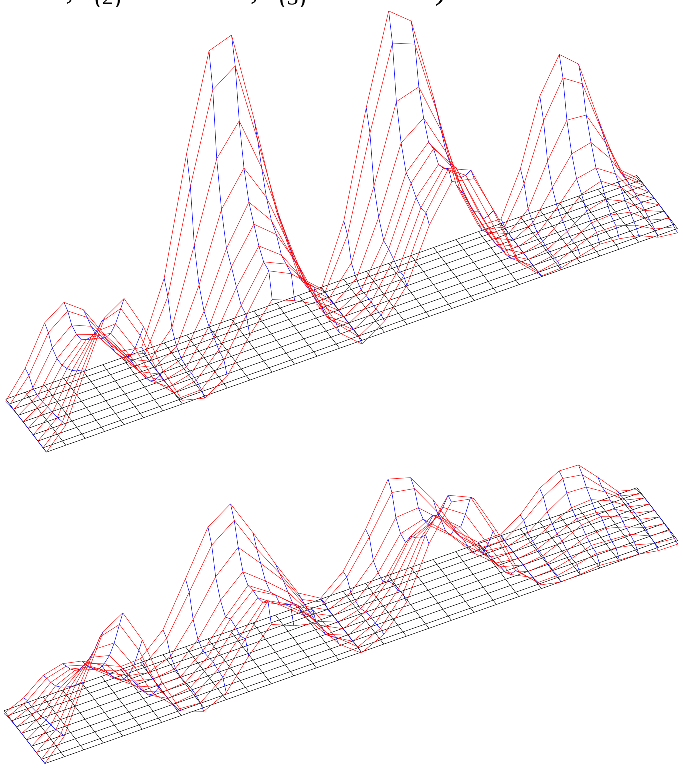


Figure 11: The comparison of experimental and theoretical natural modes using $\Delta[\delta]$, model A (above), model B (below).

2.3.3 The verification and the identification of the bridge FEM models

Two FEM models of the investigated bridge were created in program NEXIS 32 using shell-plate and beam elements. The program NEXIS was chosen because it was usually used for static and dynamic computations of bridges by a number of designers in Czech Republic at the time of the experiment.

The model A was the initial model and it was created before realization of the experiment. The results of this model were used for preparing an experiment.

The model A was verified based on evaluated experimental results. The procedures used for localization of damage (changes of a mode surface curvature $CAMOSUC_{(j),x}$, changes of a modal flexibility matrix Δ[δ] and the second derivative of changes of diagonal members of a modal flexibility matrix Δ[δ]″) were used for comparison of measured and computed modes. The essential differences were found out as between measured and computed natural frequencies (Tab.4) and between measured and computed natural modes (Fig. 11 and 12). Therefore, identification of bridge model was carried out. The bridge model B was the result of the identification.

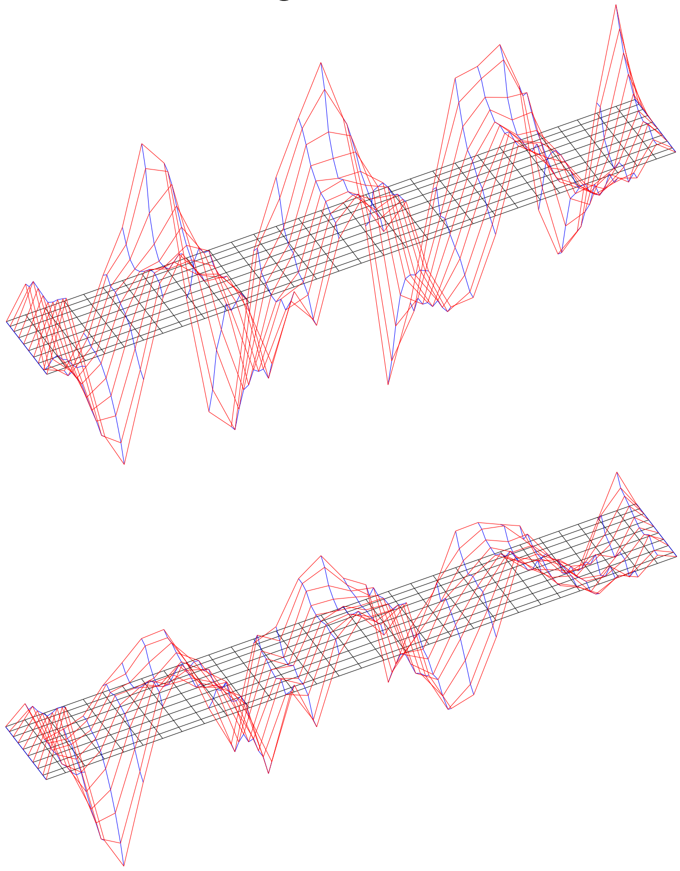


Figure 12: The comparison of experimental and theoretical natural modes using Δ[δ]″ , model A (above), model B (below).

Based on comparison of natural modes using Δ[δ] a Δ[δ]″ (Fig. 11 above and 12 above), the areas with the greatest differences of dynamic behaviour of the real structure and the model A were localized (especially along whole more distant edge of the bridge). The changes of the model B were concentrated to these areas during its identification.

The main differences between models A and B are that stiffness of both concrete cornices, road layers, pavements and concrete levelling topping was added to the stiffness of the bridge slab.

The lower values Δ[δ] and Δ[δ]″ (Fig. 11 below and 12 below) computed for comparison of experimental and theoretical results computed on the model B show that the model B is more apposite than the model A.

Table 4: The comparison of experimental and theoretical natural frequencies using $\Delta f_{(j)}$.

Frequency No. (j)	Experiment f(j)exp [Hz]	Calculation			
		Model A		Model B	
		f(j)theor [Hz]	Δf(j) [%]	f(j)theor [Hz]	Δf(j) [%]
1	6.02	5.17	-16.55	5.70	-5.67
2	7.56	6.63	-14.08	7.35	-2.84
3	9.69	8.02	-20.76	8.83	-9.78
4	10.90	9.91	-10.02	10.31	-5.74
5	11.29	10.44	-8.19	10.55	-7.05
6	13.32	13.06	-1.97	14.08	5.37

FEM models of the bridge were verified based on MAC (Modal Assurace Criterion) too.

2.3.4 Conclusions

The FEM model verification and identification was done for investigated bridges. From these techniques, it results that concrete cornices, road layers, pavements and concrete levelling topping have to be included to the stiffness. The influence of these parts was substantial for the dynamic behaviour of the structures.

The lower values of $\Delta f_{(j)}$ (Tab. 4), $\Delta[\delta]$ and $\Delta[\delta]''$ (Fig. 11 below and 12 below) for the model B of the bridge near Heřmanova Huť show that the model B is more apposite than the model A. Beside this, it is possible to localize places, where the biggest dissimilarities between the real structure and the model have occurred.

3 THE EXPERIMENTAL INVESTIGATION OF THE FORCED VIBRATIONS

The forced vibration occurs when external dynamic forces – forces variable in time - act on a structure.

The most of building structure loads have variable character. If the change of load is slow the dynamic effect of this load could be totally ignored or it is considered only approximately with the dynamic coefficient mostly. If dynamic effects are not possible to ignore, it is necessary to investigate them by the dynamic computation or by an experiment in more detail.

The role of the dynamic experiment is important especially in case when a significant dynamic load is acting on a building structure. Especially, if this dynamic load induces expressive transient vibration of a structure and it has random character hardly describable by analytic relations. And also when dynamic forces can cause resonance vibrations and damping of building structure is low (e.g. [5]).

In this chapter, questions of forced vibration of structures are documented on three examples of important footbridges where a dynamic experiment eminently contributed to verification and ensuring their reliability (e.g. [7]). And below it is documented by an experiment focused on long-time monitoring of the road bridge response caused by heavy traffic (e.g. [13]).

3.1 THE DYNAMIC BEHAVIOUR OF THE STEEL CABLE-STAYED FOOTBRIDGE

The steel cable–stayed footbridge, which overcomes the Czech motorway D5 near Plzeň, was put in operation in 2004.

The structural system of the footbridge consists of a single span steel box girder with two planes of cables and an external steel pylon (Fig. 13). The box girder is lifted by three pairs of front stays. The pylon is anchored by three pairs of back stays to the massive concrete fundament.

Figure 13: The footbridge overview.

The theoretical span is 64.8 m. The width of the deck is 3.5 m. The height of the pylon is 24.15 m. The box girder is supported by swing steel bearings. The horizontal forces are transmitted to the massive abutment by a special elastomer bearing.

According to the tender design a tuned mass damper (TMD) was necessary to reduce vibration of the footbridge due to live load. During the final design, the structural system arrangement and some parts of the footbridge had to be completely remade but the previous problem with vibration remained. According to the theoretical dynamic analysis two natural frequencies ($f_{(1)}$ = 1,33 Hz and $f_{(3)}$ = 2,93 Hz) were within the problematic frequency range <1,3 Hz ; 3,4 Hz> close to natural walking frequency (typically around 2 Hz) for the vertical direction. It was the main reason to analyze more in detail the forced vibration of the footbridge caused by a pedestrian loading. The comparison of the dynamic analysis results with the pedestrian comfort limits showed, that the vertical vibration of the footbridge deck does not satisfy the limits. On the basis of these results the question to improve damping of the footbridge was discussed. The installation of a TMD was chosen as the best solution in this case. The mounting construction of the TMD was prepared on the horizontal load bearing structure of the footbridge.

Likewise, it was decided to realize the dynamic loading test in two stages.

3.1.1 The 1st dynamic load test in situ

After the final construction works had finished, the first dynamic load test in situ was carried out. The main topics of this experiment were to find out the frequencies and modes of natural vibration, damping and the response of the real structure to different sets of pedestrian loads.

The modal analysis with forced vibration technique (which is described in chapter 2.3.2) was used to find out natural frequencies and natural modes.

The response of the footbridge to shaker excitation was measured in vertical and transverse horizontal directions in a chosen net of points (84 points – 21 cross sections and 4 points in each one) on the footbridge deck.

Table 5: The comparison of the selected experimental and theoretical natural frequencies of the footbridge deck.

Measured natural frequencies		Calculated natural frequencies		Deviation	Description of the mode shapes of the deck
No. (j)	$f_{exp(j)}$ [Hz]	No. (j)	$f_{theor(j)}$ [Hz]	$\Delta f_{(j)}$ [%]	
(1)	1.55	(1)	1.329	-16.63	1st vertical bending
(2)	1.85	(2)	1.725	-7.19	1st horizontal bending
(4)	3.24	(4)	2.927	-10.63	2nd vertical bending
(7)	6.22	(7)	5.766	-7.93	3rd vertical bending

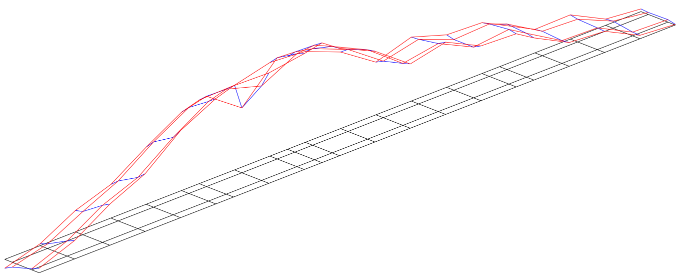


Figure 14: The 1st measured natural mode of vibration, $f_{(1)}$=1.55 Hz, vertical part.

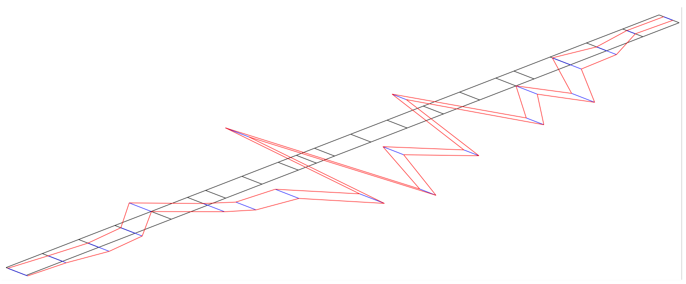


Figure 15: The 2nd measured natural mode of vibration, $f_{(2)}$=1.85 Hz, horizontal part.

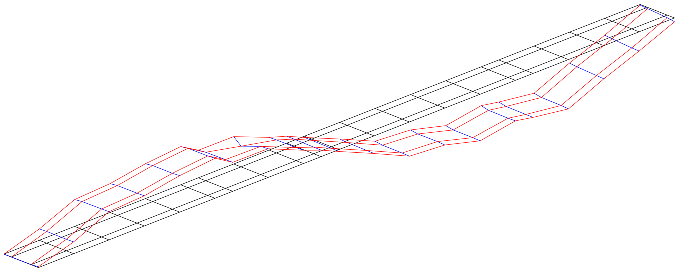


Figure 16: The 4th measured natural mode of vibration, $f_{(4)}$=3.24 Hz, vertical part.

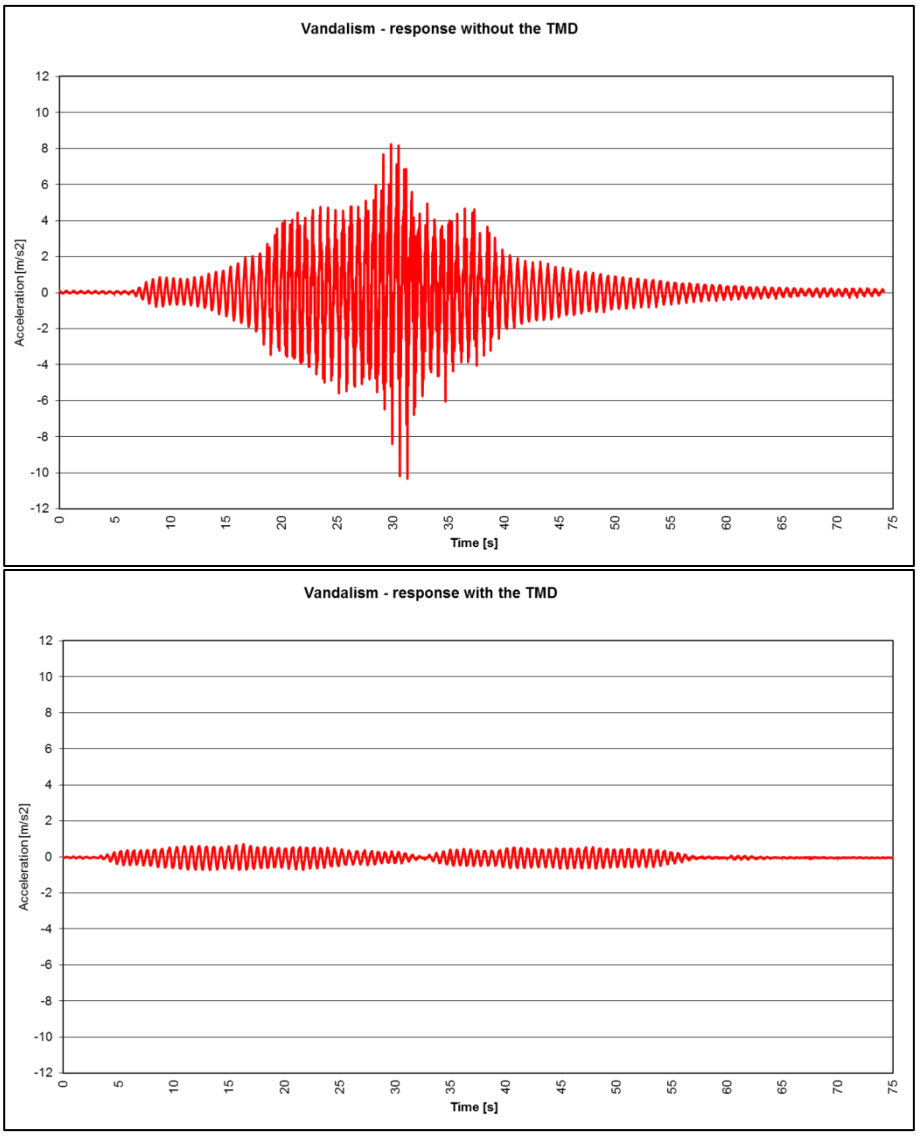


Figure 17: The dynamic response of the footbridge deck to the vandalism without TMD (above) and with the TMD (below).

Six natural frequencies, mode shapes and damping frequencies of the footbridge deck and three natural frequencies of the pylon were evaluated after the experimental modal analysis in the excitation range to 20 Hz.

The chosen measured natural frequencies compared with calculated frequencies are in Tab. 5. The measured significant mode shapes are shown in Fig. 14, 15, 16. The damping of the footbridge was small, the evaluated damping ratio ζ = 0.006 (0.6%)

was identical for all in Tab. 5 mentioned significant natural frequencies. As one can see the measured natural frequency $f_{exp(1)}$ = 1.55 Hz was closer to the most common natural walking frequency 2 Hz than the corresponding calculated natural frequency $f_{theor(1)}$ = 1.33 Hz.

Subsequently, the forced vibration was studied. During this part of the experiment, the vibration of the footbridge was caused by different groups of pedestrians: 2 persons walking in resonance with the natural frequency of the first vertical bending mode, 2 persons running with double natural frequency of the first horizontal bending mode, 2 persons running in resonance with the natural frequency of the second vertical bending mode and 12 persons walking in resonance with the natural frequency of the first vertical bending mode. After walking groups of pedestrians, the footbridge was loaded by vandalism: 12 persons swaying in knees in the place of the highest vertical ordinate of the first vertical bending mode, 10 persons running in resonance with the natural frequency of the second vertical bending mode, 11 persons running in the quarter of the span of the footbridge in resonance with the natural frequency of the second vertical bending mode, 11 persons running in the quarter of the span of the footbridge with double natural frequency of the first horizontal bending mode.

The response of each group of pedestrians was recorded several times. Different couples of pedestrians were created to realize a variability of action of forces depending on individual characters and abilities of pedestrians for investigation of two synchronized pedestrians' effects.

The footbridge forced vibration was observed in vertical and transverse horizontal directions in the cross sections in which the maximal ordinates were evaluated for the first three experimentally determined natural mode shapes.

An example of the footbridge deck response without an absorber caused by vandalism is shown in Fig. 17 above.

Table 6: Examples of maximum accelerations of the footbridge deck caused by groups of pedestrians with and without the TMD.

Description of the pedestrians group	Vertical acceleration			Vertical displacement		
	Without TMD	With TMD	a_{z1}/a_{z2}	Without TMD	With TMD	d_{z1}/d_{z2}
	max RMS	max RMS		max RMS	max RMS	
	a_{z1} [ms^{-2}]	a_{z2} [ms^{-2}]		d_{z1} [mm]	d_{z2} [mm]	
2 pedestrians, synchr. walk at f=1.55 Hz	**0.26**	0.10	2.56	**2.97**	0.77	3.86
10 pedestrians, synchr. walk at f=1.55 Hz	**1.28**	0.19	6.85	**6.39**	1.85	3.45
10 pedestrians, random walking	**0.22**	0.08	2.64	**2.55**	0.69	3.70
10 pedestrians, random runing	**0.24**	0.13	1.44	**0.78**	0.56	1.39
10 pedestrians, vandalizmus at f=1.55 Hz	**3.27**	0.43	7.52	**34**	5.43	6.26
10 OSOB, vandalizmus at f=3.24 Hz	**3.29**	2.45	1.34	**1.16**	1.12	1.04

Notice: The highlighted displacement was only estimated based on the measured acceleration.

Figure 18: The installed absorber (TMD) inside of the box girder.

The pedestrian comfort limits were not exceeded for the groups of pedestrians, which simulated a normal operation on the footbridge. Nevertheless, an idea of using a TMD has been considered to reduce the vibration mainly at the lower natural frequency $f_{(1)}$ = 1.55 Hz due to the very high dynamic sensitivity of the footbridge to vibration at this frequency caused by vandalism (Fig. 17 above), in which the measured acceleration peak value exceeded gravitational acceleration, the evaluated peak value of vertical displacement was 10 cm.

Based on the results of the first dynamic load test in situ, it was decided that a TMD designed by GERB GmbH will be installed.

3.1.2 The 2nd dynamic load test in situ

Before this stage of the experiment, the TMD had been installed and unlocked. The topic of this second stage of the experiment was only to analyse the response of the structure to a pedestrian loads. The same system of loading was used as during the first test.

An example of the deck response is shown in Fig. 17 below. It describes the response of the deck to vandalism. It means 12 persons swaying in knees in the place of the highest vertical ordinate of the 1st vertical bending mode shape at the frequency f = 1.55 Hz. Everyone can see from Fig. 17, that the acceleration of the deck with the TMD is the one eighth of the response of the structure without the TMD.

Using the TMD the dynamic response of the footbridge to pedestrian loading has decreased under the pedestrian comfort limits even for the extreme vandalism.

The damping of the footbridge with the TMD increased about three times. This has a positive influence to all structural members of the footbridge.

3.1.3 Conclusions

The described experiment was focused on forced vibration of the investigated footbridge which was caused by pedestrian loading. The experiment got the significant results that eminently contributed to ensuring its reliability.

Though the pedestrian comfort limits were not exceeded for the experimental simulation of the normal operation on the footbridge, the experiment detected the very high sensitivity of the footbridge to vandalism. The extreme peak to peak amplitude19 cm of vertical displacement of vibration caused by vandalism was evaluated. This detected level of vibration was not acceptable for investigated footbridge.

For potential vandals it was not difficult to find out the resonant frequency from a footbridge vibration excited by a pedestrians' passage. Also the synchronization of the vandals' movement was easy due to strong footbridge vibration.

After the TMD installation, the excessive vibration of the footbridge was not possible to excite.

3.2 THE DYNAMIC BEHAVIOUR OF THE FOOTBRIDGE ACROSS THE „K BARRANDOVU“ STREET IN PRAGUE

The footbridge in Prague – Barrandov (Fig. 19) was put into operation 1.6. 2006. This footbridge connects two urban areas, divided by the frequented highway "K Barrandovu". The footbridge is formed by the tube main girder with the truss bridge deck, which is suspended to two skew pylons. Two tuned mass dampers (TMDs) are used to ensure comfort for pedestrians.

During the design of the footbridge the theoretical dynamic analysis was performed. In this dynamic analysis two natural frequencies ($f_{(1)}$ = 2.29 Hz, $f_{(2)}$ = 2.57 Hz) were found out which corresponded to the natural modes of vertical vibration of the footbridge deck and which were within the problematic frequency range <1.3 Hz ; 3.4 Hz> close to natural walking frequency (typically around 2 Hz) for the vertical direction. Due to the low damping and high slenderness of a footbridge the considerable sensitivity of a footbridge to dynamic excitation by pedestrians was found out. Therefore, it was necessary to ensure the comfort for pedestrians, the footbridge load bearing structure was equipped with two tuned mass dampers (TMDs).

Due to the limited time available, it was not possible to carry out the dynamic load test of the footbridge in two stages. TMDs had been ordered from the company GERB. Both TMDs were made with a mass of 850 kg with adjustment of the masses within the range of ± 50 kg. In production the natural frequency of the first TMD was tuned to 2.18 Hz and of the second TMD to 2.44 Hz, i.e. 95% of the calculated first and second natural frequencies of the footbridge.

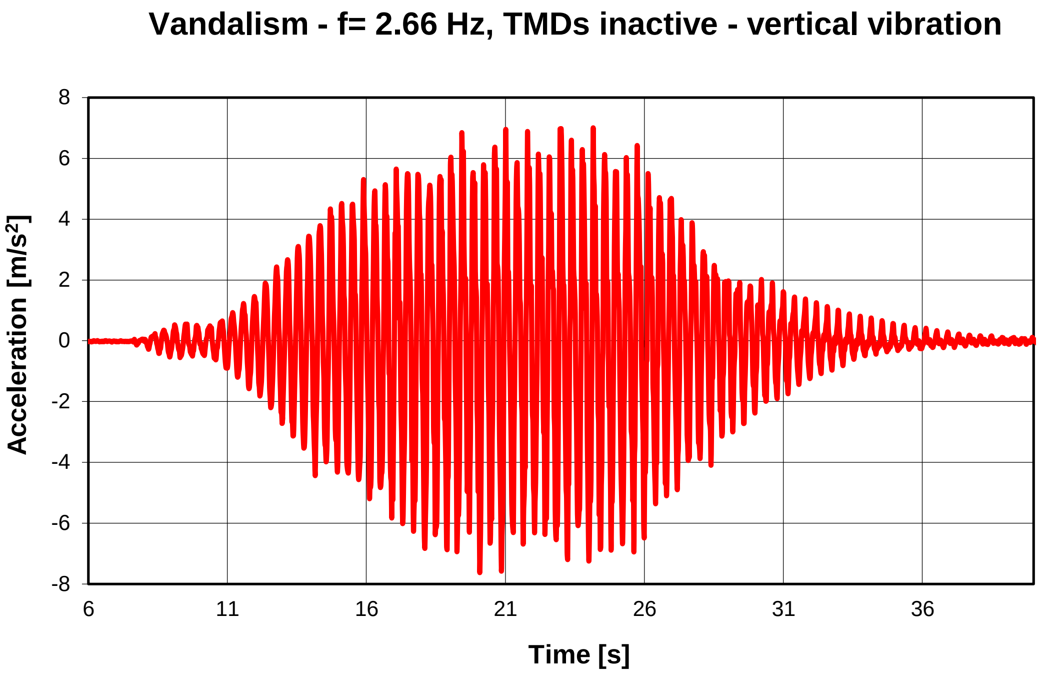
The dynamic load test was carried out on the footbridge after the installation of TMDs just before the footbridge was put in operation. During the first stage of the test the TMDs were not functional. The working activity was divided into three steps:

- The significant natural frequencies, which corresponded to the global natural modes of the footbridge deck, were evaluated from the vibration of the footbridge ($f_{exp(1)}$ = 2.47 Hz, $f_{exp(2)}$ = 2.66 Hz). The modal analysis with the forced vibration technique (which is described in chapter 2.3.2) was used to find out natural frequencies and natural modes. The evaluated natural frequencies were compared with the corresponding calculated natural frequencies.

- The dynamic response of the observed footbridge to the extreme effects of a group of vandals (Fig. 20) was measured.
- The tuning of the installed TMDs was assessed. The modification of the masses was proposed to optimize their ability to reduce vibration of the footbridge on the basis of the differences between the measured and calculated natural frequencies, for which the TMDs were tuned. For the first TMD tuned to a lower natural frequency was proposed the mass modification to 751 kg and for the second TMD to 814 kg.

Figure 19: The footbridge overview.

Figure 20: The group of vandals on the footbridge deck.



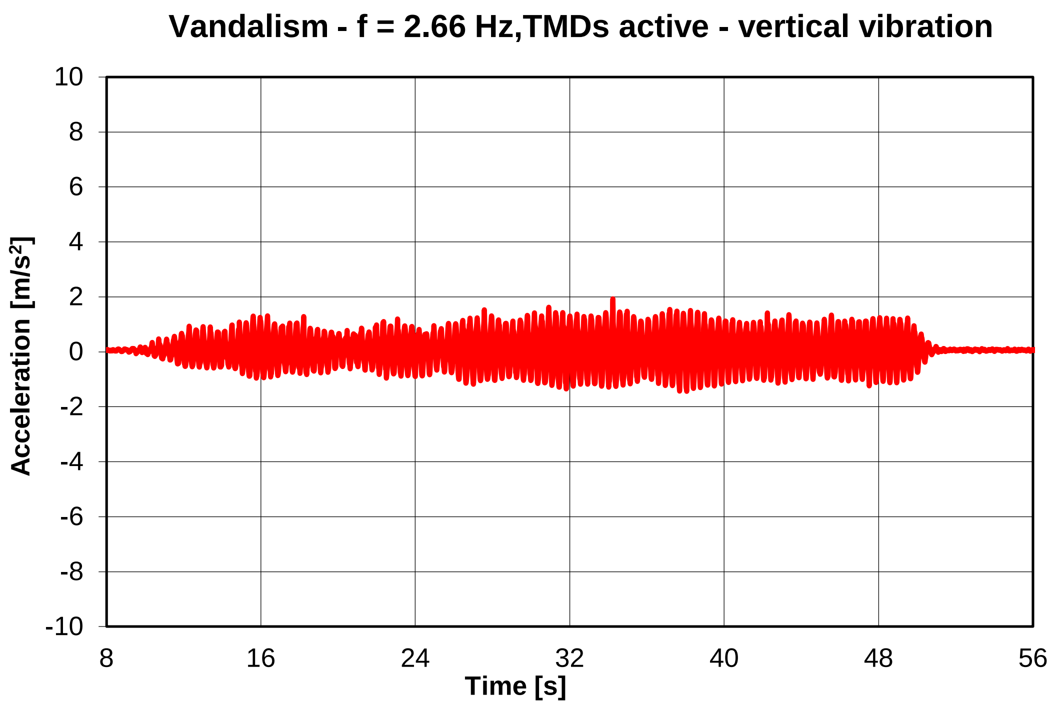


Figure 21: The dynamic vertical response of the footbridge deck to the vandalism with the inactive TMDs (above) and with the active TMDs (below).

Following the completion of described modifications both absorbers were put into operation. Then the second stage of the dynamic load tests was carried out. It was divided into two steps:

- The efficiency of the TMDs caused by the pedestrians was verified based on the dynamic response of the footbridge to the extreme effects caused by the group of vandals (Fig. 21).

- The footbridge response to passages of different groups of pedestrians, which simulated the normal operation of the footbridge, was measured. The evaluated vibration was assessed with the pedestrian comfort limits.

3.2.1 Conclusions

The evaluated differences between the significant measured natural frequencies and the corresponding calculated natural frequencies were small. The mathematical model of the footbridge, which was used in the footbridge design, was accurate sufficiently.

The evaluated damping of the footbridge matched to the type of the footbridge load bearing structure, the estimated damping ratio $\zeta = 0.008$ (0.8%), was identical for two lowest natural frequencies and it is higher than the damping used in computation.

The results of the dynamic load test showed clearly that the footbridge structure without the TMDs would be very sensitive to the dynamic effects of pedestrians and that the installation of two TMDs was necessary to ensure pedestrian comfort on the footbridge.

The ability of the passive dynamic dampers, which are installed on the investigated footbridge, for significant reduction of the level of the footbridge vibration was demonstrated on the response caused by group of vandals. As shown in Fig. 21, the dynamic response of the footbridge significantly decreased after activation of TMDs to one seventh.

The optimal tuning of the TMDs based on the experimentally determined real natural frequencies of the footbridge increased the effectiveness of both dampers in the reduction of a footbridge forced vibration.

3.3 THE DYNAMIC BEHAVIOUR OF THE FOOTBRIDGE ACROSS THE BEROUNKA RIVER IN PRAGUE - RADOTÍN

The footbridge across the Berounka river in Prague - Radotín (Fig. 22) was put in operation in October 2010. The load bearing structure of the footbridge is suspended to cantilevers of two box girder prestressed concrete bridges that are parts of the Prague Ring Road (SOKP 514). The footbridge suspenders are made out of steel rods with a diameter of 26 mm. The distance of cross sections with suspenders is 4.0 m. The horizontal load bearing structure is the steel and concrete composite structure. The cross section consists of two main steel girders HE300B and transverse beams HE300A. The girders are coupled with a reinforced concrete slab with the thickness of 200 mm. The footbridge has three spans of lengths 48 m, 84 m and 48 m. The horizontal alignment of the footbridge is straight line. The vertical alignment of the footbridge is parabolic vertical curve with the rise of 3.75 m in the centre of the footbridge.

A special feature of this footbridge is that the structure supporting is flexible in the longitudinal axis. The bearing, which does not allow movement of the footbridge in the horizontal longitudinal direction, is placed on the transverse beam. This beam is placed between the spans number 1 and 2 and it is fixed on the pillars of road bridges. The expansion joints on the both abutments are covered with elastomeric seals.

Figure 22: The footbridge overview.

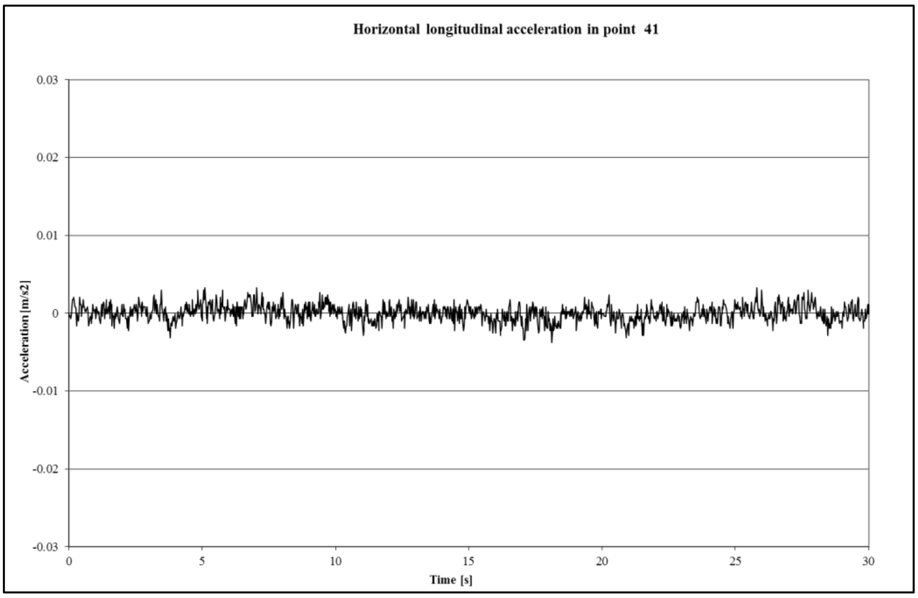


Figure 23: The dynamic horizontal response of the footbridge deck to the full braking of one cyclist.

The theoretical dynamic analysis of the footbridge investigated whether it is necessary to install hydraulic dampers on the footbridge, which would increase damping for horizontal vibration of the footbridge deck in its longitudinal axis.

The dynamic load test was carried out in same way as on the previous two footbridges. In addition, the effects of two special groups of footbridge users were investigated by assessing of the horizontal longitudinal vibration of the footbridge deck:

- The full braking of one cyclist on the footbridge deck.
- The sudden stop of five sprinting runners on the footbridge deck.

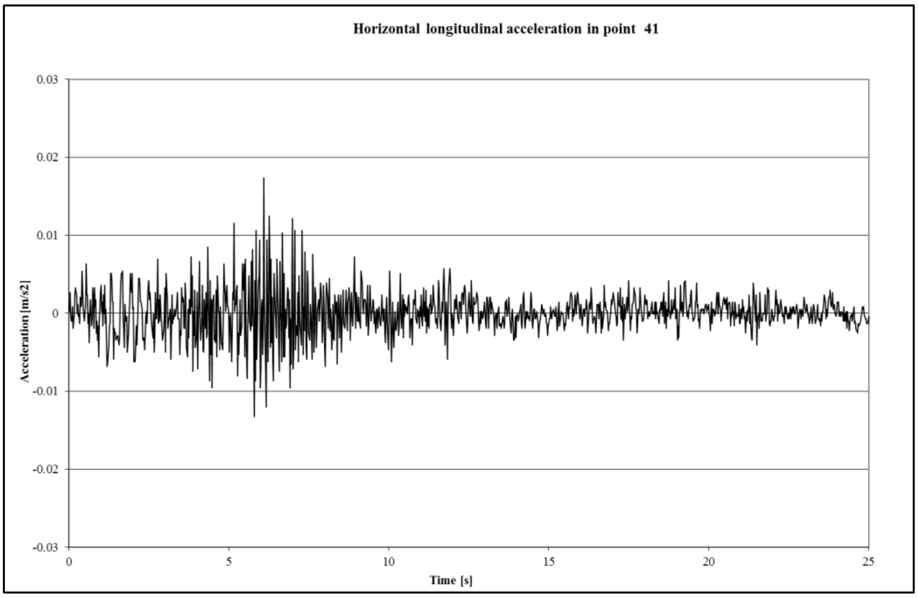


Figure 24: The dynamic horizontal response of the footbridge deck to the sudden stop of five sprinting runners.

The sudden stop of sprinting runners excited especially the vertical vibration of the footbridge deck. As shown in Fig. 23 and 24, the horizontal longitudinal vibration of the footbridge deck caused by the above mentioned two groups of users is small. The pedestrian comfort limit was satisfied. The installation of hydraulic dampers was not necessary.

3.4 THE LONG-TIME MONITORING OF THE SLAB-ON-GIRDER BRIDGE RESPONSE CAUSED BY HEAVY TRAFFIC

In the past twenty years in Czech Republic the traffic density on the roads has increased rapidly, especially on the highways and on the first class roads. The composition of traffic flow has changed too. The number of heavy duty trucks has increased substantially and thus the loading of the roads and the road bridges has increased.

The long-time monitoring focused on the bridge response caused by heavy duty traffic and by temperature changes was carried out on three different bridges at the same time in years 2006 - 2009. The basic aim of the experiments was to obtain real data for assessment of representativeness of models used for modelling of traffic and temperature loads on the road bridges.

Only the results of long-time monitoring of the slab-on-girder bridge response caused by heavy duty traffic are presented below. The investigated bridge (Fig. 25) is the three span continuous slab-on-girder bridge, which is situated across the four lane Prague Ring Road. The slip lane, which is changing into two lane road on the bridge leads to the street „K Barrandovu“ in Prague. Before opening new part of the Prague Ring Road in September 2010, the heavy duty traffic goes in the south lane of the bridge in the direction Plzeň D5 – Prague Ring Road – Brno D1.

The lengths of spans are 17.7 m + 34.5 m + 17.7 m. The bridge is skew with the skewness about 76˚. The load bearing structure of the bridge is composed of four main steel I-shaped girders. The axial distance of the main girders is 3300 mm. The cast-in-place reinforced concrete slab is without transversal inclinations, with constant thickness 240 mm. At the edges, there are short cantilevers of the length 420 mm, resp. 820 mm.

The stress and strain response of two main girders (south outer girder and neighbouring inner girder) and of the reinforced concrete slab was continually measured in the cross section at the middle of the bridge span during the long-time experimental monitoring.

The measurement line for monitoring of the dynamic response of the bridge was composed from the recording station EMS DV 803 of the company EMS Miroslav Pohl from Brno in Czech Republic and fourteen strain gauges. The three resistance strain gauges were used for measuring the relative deformation of the concrete slab (type 100/120 LY41 Hottinger Baldwin Messtechnik). The measurement of the relative deformations of the steel girders were done by using of the eleven resistant strain gauges 10/120 LY11 Hottinger Baldwin Messtechnik. The two accelerometers of type B1 Seika was used for observation of the vertical vibrations of two main girders.

The advantage of the used measurement line is that the control software of the dynamic measurement station EMS DV 803 enables continuous response measurement of the investigated bridge because the station does not need permanent connection to the computer. When the station is connected to the power supply and set properly, it can work with its inner memory (Compact Flash 512 MB) continuously and separately 5 months. The station records the values in two modes. In the basic mode the values of the response is recorded with period 30s. On the background the station measures the dynamic response continuously with the sampling frequency 50Hz. If the investigated structure vibrates significantly or the response reaches the level set in the control program, the station switches to the second mode and starts to record only peaks of the dynamic response, for which is the magnitude higher than the limit value. If the vibration is subsiding, the station switches back to the basic mode. The character of the response record composed of the values recorded in both modes is shown in the Fig. 27 and 28.

Figure 25: The view on the investigated bridge

The heavy truck passages induce mostly the quasistatic response only with the strain cycle up to 100 μm/m (Fig. 28) in the lower flange of the main girders. It corresponds to the dynamic stress cycle 21 MPa. Sometimes the significant dynamic response (Fig. 27) with the dynamic stress cycles about 5 MPa is caused by these passages. Significant dynamic response was detected approximately 50 times a day.

One of the objectives of the described experiment was to monitor the level and intensity of fatigue stress on the investigated bridge. But, against all primary expectations, the observed dynamic and quasistatic stress cycles were usually less than the fatigue threshold (40 MPa) of the bridge load carrying structure. Irregularly the very heavy truck crosses over the bridge and causes approximately the double response than usually (Fig. 28 above).

Figure 26: Two heavy trucks, which caused dynamic response of the investigated bridge (Fig. 27).

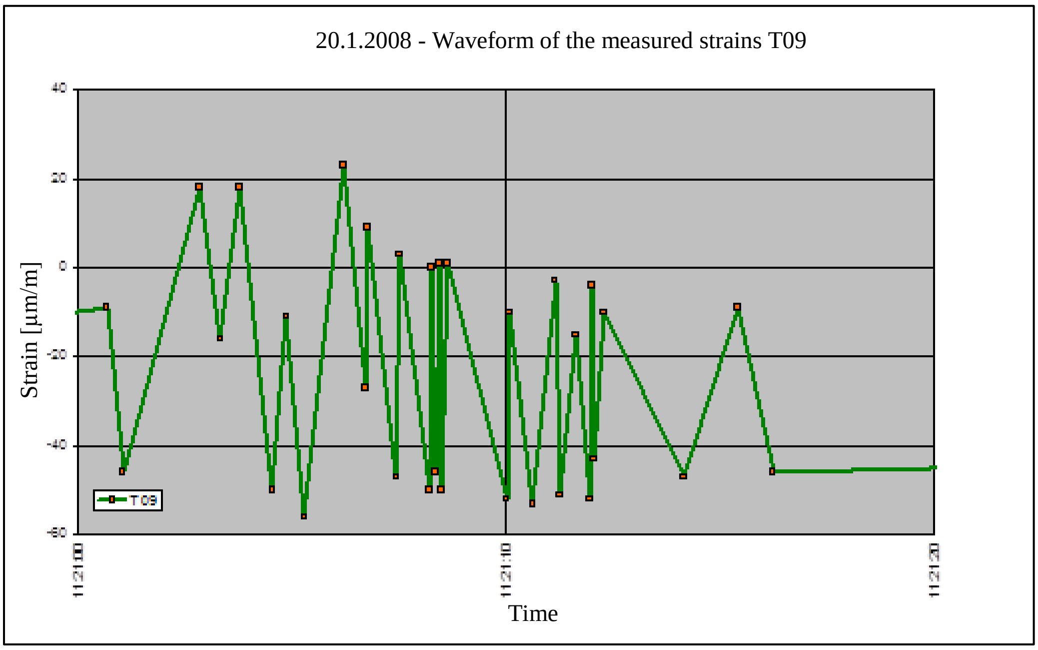


Figure 27: The dynamic response of the investigated bridge caused by two heavy trucks (Fig. 26).

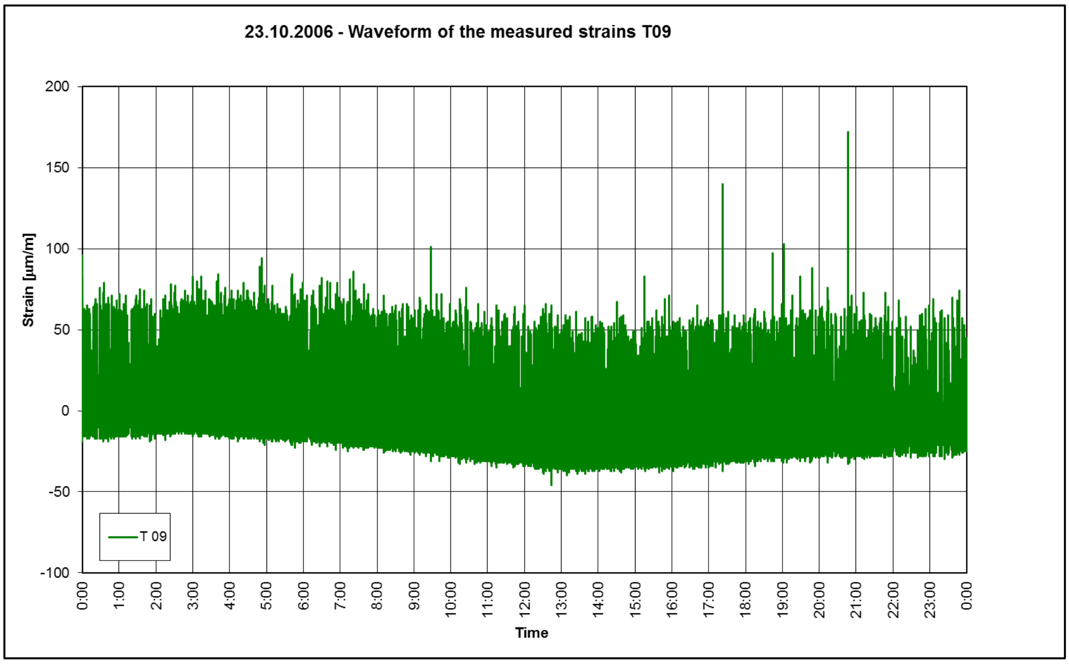


Figure 28: The waveform of he measured strains on the lower flange of the main girders on Monday 23.10.2006

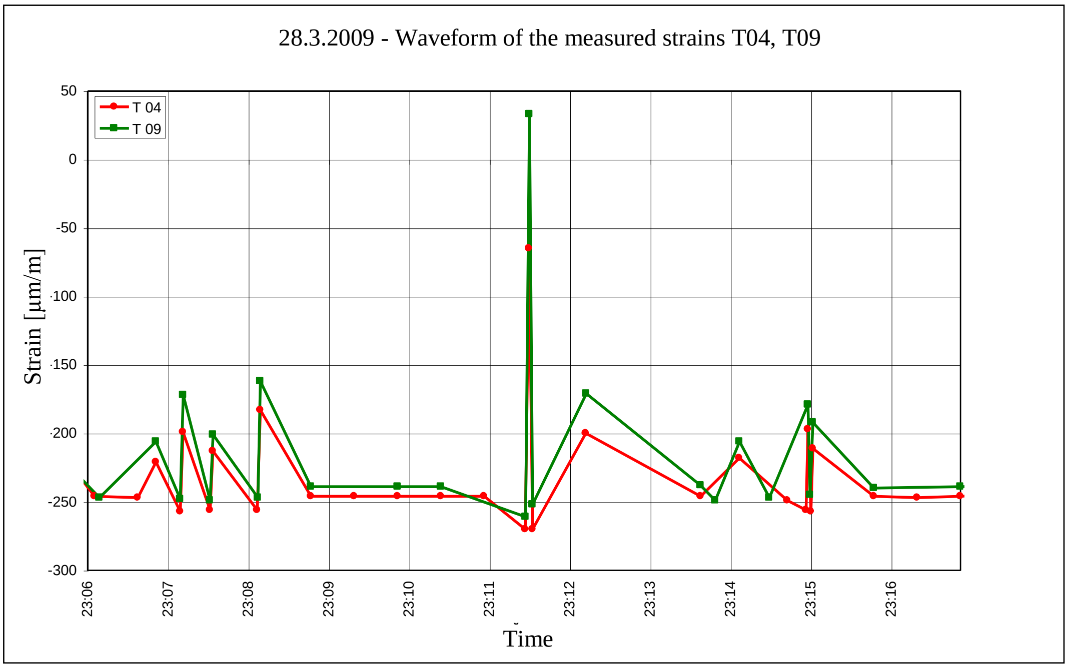


Figure 29: The substantial loading of the bridge structure, the measured extreme strain cycle on the figure is 294 μm/m, which corresponds to the stress 62 MPa – the equivalent weight of heavy truck is 123 t.

Figure 30: The vehicle of well-known weight coming over the bridge.

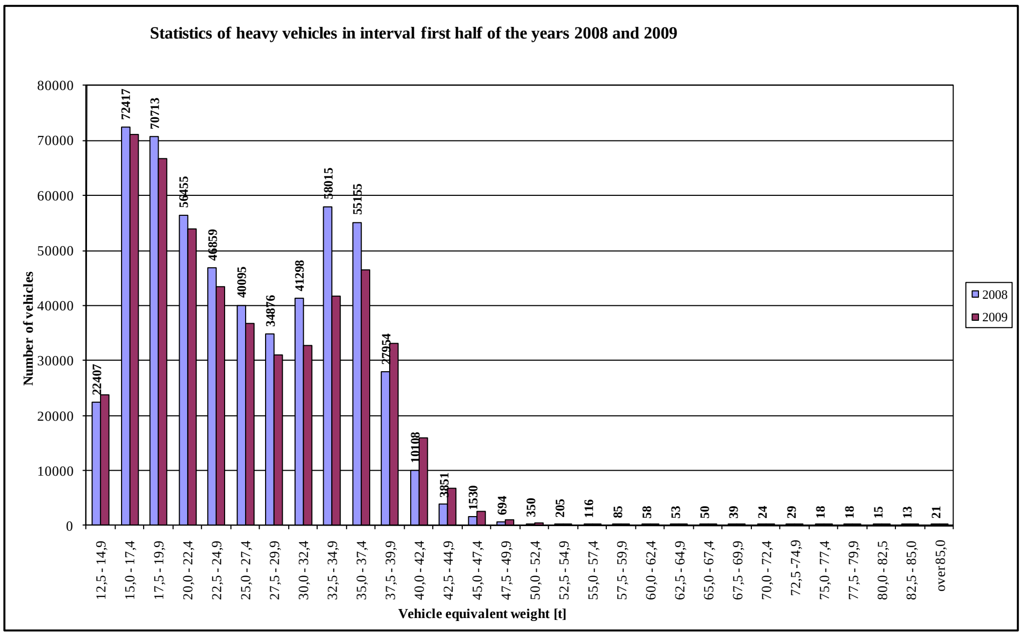


Figure 31: Statistics of the moving load of the bridge – comparison of the number of heavy vehicles for the first half of the years 2008 and 2009 (the total number of the heavy trucks 543 963 (2008) and 509 200 (2009)).

One of the objectives of the experiment was monitoring the real load of bridges under its operation. It is possible to estimate weight of the heavy duty trucks passing over the bridge on the basis of the measured response of the bridge. The calibration of measured relative deformations was done base on several passages with the vehicle of well-known weight. Because the weight of the vehicles is determined indirectly on the basis of the measured response of the bridge it is called “vehicle equivalent weight”. With respect to the different schemes of the axle positions of the “frequent” heavy duty trucks, the equivalent weight of the common heavy truck is determined with the tolerance + / - 15 %. Estimation of the weight for three-axle trucks (e.g. Tatra 815) is lower than reality, for the lorry-trailer combination with two-axle trailer it is higher than reality and for the lorry-trailer combination with three-axle trailer it is the closest to reality. In accordance to the setting of the measuring line, the passages of trucks heavier than 14 t can be evaluated from the results of the quasistatic measuring of bridge response.

The largest bridge response caused by a very heavy truck during the whole experiment was detected 28.3.2009 23:11:20 (Fig. 29). The equivalent weight of the truck was 123 t, it was an extraordinary transportation of heavy load. During the experiment in the years 2008 and 2009 the significant response of the bridge caused by trucks of the equivalent weight 90 t was measured about forty times and five times it was the response caused by trucks of the equivalent weight more than 100 t.

3.4.1 Conclusions

The amplitude of the relative deformation on the lower flanges caused by heavy duty trucks was usually up to 100 μm/m, the corresponding stress is 21MPa. This value is under the fatigue limit value, the fatigue life of the bridge is not influenced by this loading in the monitored cross section

As it is visible from the Fig. 31, the traffic intensity in the year 2009 has decreased about 6% in comparison with the intensity in the year 2008 (as the consequence of the world economic crisis) but the number of the trucks heavier than 37.5 t has increased.

4 FINAL CONCLUSIONS

Building structures are usually unique. They differ one from another in dimension, shape, construction systems, used materials or soil properties in bottom of a footing. Therefore the questions of experimental examination of building structures are generally based on individual approach to each of the tested structures.

It is important to set every experiment on a building structure in such a way to be obtained results specific, meaningful and useful and not just purposeless. It is necessary to analyze the expected static or dynamic behaviour of the structure in the experiment preparation based on theoretical analysis and practical experience. According to the analysis results, it is important to identify decisive parameters of investigated structure, which are suitable and useful to monitor experimentally, to select the appropriate set of measurement equipment, to determine procedure of data acquisition and results evaluation. Every experiment is loaded by an error and it is essential to quantify the uncertainty of measurement results to interpret the data correctly.

In the education of courses, whose content is the experimental analysis of building structures, it is important to complement the theoretical information with the examples of experiments which were carried out on the real structures. At these experiments, it is necessary to explain the reasons of their application, the experiment arrangement and to analyse the results. The practical education is important too. The experimental tasks should be interesting and focused practically. The students should carry out the tasks independently including the interpretation of the evaluated data.

The process of experimental analysis of dynamic behaviour of building structures was documented in this publication on several practical examples in which the role of dynamic experiments was important:

- the identification of cable forces in the roof structure of the Administrative Centre Amazon Court,
- the detection of fatigue damage of the fully prestressed concrete slabs by using of experimental modal analysis,
- the verification and the identification of the bridge FEM models by using of experimental modal analysis,
- the experimental analysis of dynamic behaviour of three important footbridges (the steel cable-stayed footbridge, the footbridge across the „K Barrandovu" street in Prague and the footbridge across the Berounka river in Prague – Radotín),
- the long-time monitoring of the slab-on-girder bridge response caused by heavy traffic.

The described experiments are especially interesting in terms of results, of uniqueness of the tested structures or by original arrangement of the measurement system.

Acknowledgement

Author gratefully acknowledges the support from the Czech Science Foundation (the project No. GAČR 105/11/1529) and the support from the Ministry of Education, Youth and Sports (the project No. MSM 6840770031).

Special thanks must be extended to my colleagues from the Experimental Center, the Department of Steel and Timber Structures and the Department of Mechanics without their help it was not possible to realize the experiments described in this publication.

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Investigator and coinvestigator of 5 contracted research projects of the Czech Science Foundation, 3 contracted research project of the Ministry of Traffic, 2 contracted research project of the Ministry of Education, Youth and Sports, author and co-author of practical applications of experimental and theoretical research work on site (bridges, footbridges, engineering structures).

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