Monitoring and evaluation of an arch bridge affected by the blasting of the adjacent highway bridge

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ABSTRACT: The bridge object W4-Steyrermühl consists of two separate superstructures. As one of the two structures was blown in August 2010, the remaining one had to be monitored in order to evaluate the impact of the blasting on its structural safety and operability. The comprehensive dynamic analysis – by means of BRIMOS® Structural Health Monitoring - focused on the primary load-bearing structure (arch). The on-site assessment was of crucial importance for the decision to re-open the bridge to traffic after the blasting of the adjacent structure. In a subsequent stage the effects of the blasting were analysed in detail – coming up with results which broadened the on-site findings.

1 BRIDGE DESCRIPTION

The bridge object W4 Traun Bridge Steyrermühl- constructed in 1959- is part of the Austrian highway A1 and leads across the Traun River. It is composed of 18-spans (reinforced concrete) and has a total length of 240.45 m. The bridge consists of two separate load bearing structures – one for each driving direction – with a width of 13.22 m each. The 11-span main bridge is designed as an arch bridge and is connected to two approach bridges at the western and the eastern side. The arches' cross section is made of a three-cellular 9.0 m wide box girder with a varying height of 3.0 m at the abutments and 1.50 m at the vertex. Due to the fact that the bridge did not fit into the overall traffic and infrastructure concept anymore (the highway is going to be broadened) it was decided to replace it.

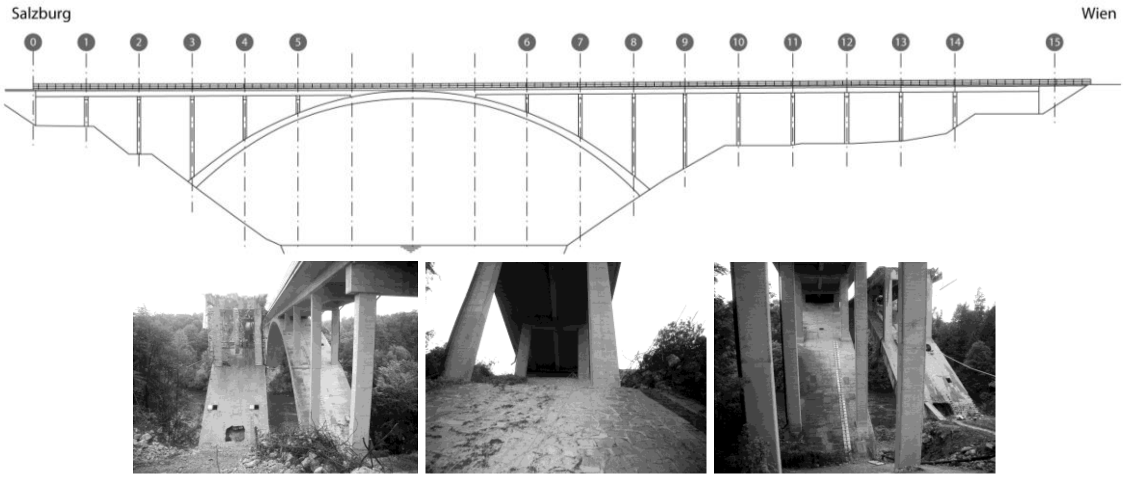

Figure 1: Elevation for the driving direction Vienna (top) and photo documentation (bottom) – W4 Traun Bridge Steyrermühl.

2 OBJECTIVES OF THE CAMPAIGN

While the bridge structure – related with the driving direction Salzburg – was removed (blown) in August 2010, the remaining structure (driving direction Vienna) was monitored in order to evaluate the impact of the blasting with regard to its structural safety and operability. The investigation was focused on the primary load-bearing structure (arch).
The monitoring campaign consisted of two major tasks: The on-site assessment was of crucial importance for the decision to re-open the bridge to traffic after the blasting of the adjacent structure. In a subsequent stage the effects of the blasting were analysed in detail – coming up with results which broadened the on-site findings.

2.1 Applied approach

In 2005 referential dynamic BRIMOS® measurements at both load-bearing structures were performed by VCE. Based on these measurements the structural behaviour was analysed in detail. In addition to the dynamic measurement a finite element model of the bridge was developed enabling an extensive system identification of the dynamic behaviour.
As the measurement from 2005 was used as a reference for the monitoring of the bridge blasting in 2010 the latter one could be performed already more efficiently by means of hot spot measurements. A direct comparison after 5 years of structural service life (2005-2010) shows the progression of structural behaviour and possible changes respectively under varying loading conditions (bridge deck under the influence of traffic load and the bridge blasting).

In the course of the accompanying monitoring and evaluation process the following tasks had to be covered:

- Preliminary survey and basic model
  Considering the reference measurement in 2005
- Installation of the follow-up monitoring system
  A permanent online measurement set-up was installed on the day before the blasting and was in operation until the bridge was opened to traffic again.
- Observation of the current structural behaviour under regular operation
  By means of a measurement lasting several hours the actual dynamic characteristic of the structure under current traffic conditions was captured. The extracted parameters represent reference values for 2010.
- Measurement at the closed bridge and during the blasting
  Capturing of the dynamic characteristics in terms of “unloaded” condition (under ambient - environmentally excited vibrations only), the characteristic during the blasting and afterwards.
- Comparison of the results
  Based on the measurements before and after the bridge blasting the key performance indicators eigenfrequency, damping and mode shapes were determined and compared. The bridge’s load bearing capacity curve was up-dated.
- Re-opening to traffic and confirmation
  In case of unsuspicious behaviour the bridge should be opened to traffic 30 minutes after the blasting at best or within 2 hours at the latest.
- Subsequent works
  Due to time constraints on-site a detailed analysis of the dynamical behaviour in longitudinal, vertical and transverse direction was carried out afterwards (in the office) to detect potentially phenomena such as displacement of the abutments or deflections. The subsequent analysis focused on temporary changes only. Irreversible changes would have attracted attention already at the on-site analysis.
- Determination of the loading level
  Via post-processing of the recorded data a loading factor was calculated reflecting the relation between the level of regular traffic loading and the exceptional loading caused by the blasting.

The assessment was subdivided into several timeframes listed below:

- Regular traffic 2005 (Phase I)
- Regular traffic 2010 (Saturday evening & night - Phase II)

- Closed bridge immediately before and after the blasting (Sunday morning - Phase III & IV)
- Regular traffic 2010 (Sunday morning- Phase V)

3 DYNAMIC SYSTEM IDENTIFICATION – COMPARISION OF MEASUREMENT AND NUMERICAL SIMULATION

In addition to the initial dynamic measurement in 2005 a finite element model of the bridge was developed. Since the structure's initial condition immediately after the completion was not stated in terms of a dynamic measurement the undamaged condition was modeled providing expected values from numerical analysis.

The relevant eigenfrequencies are primarily located in the range of 0 to 6 Hz. All of those eigenfrequencies are global eigenfrequencies representing the global stiffness in vertical and transversal direction. Figure 2 shows the relevant eigenfrequencies, their corresponding mode shapes and the deviation between the calculated eigenfrequencies and the results of the measurements in 2005. The comparison of the results from the model with those of the measurements supports the assessment of the structural condition and indicated a satisfying global condition at the time of the measurement.

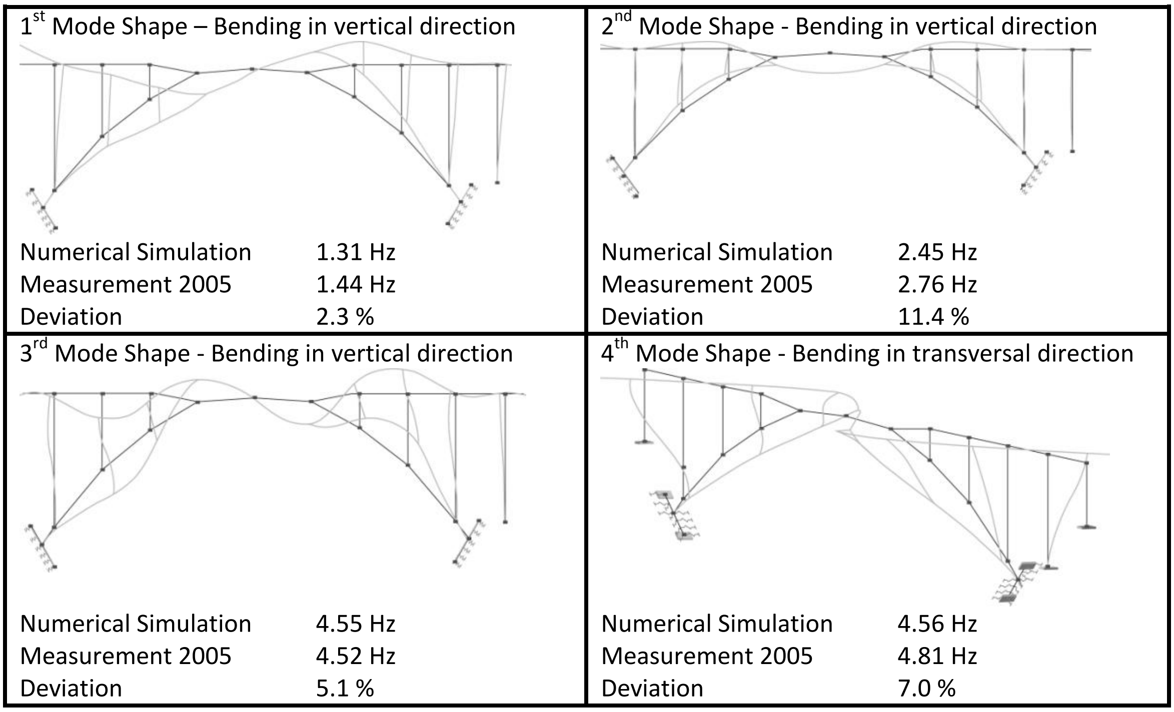

Figure 2: Relevant mode shapes and deviation between the numerical simulation and the measurement 2005.

4 MEASUREMENT DESCRIPTION

The blasting of the adjacent bridge structure was scheduled for the early morning of the 8th of August in 2010. To get a reasonable database for the BRIMOS® assessment the measurement – using a mobile acceleration sensor system - was started in the evening of the previous day. The permanent online measurement system was running from 6:30 pm on Saturday (7th of August) to 12:30 am on Sunday (8th of August).

The objective of the monitoring campaign was to document possible changes of the key performance indicators of the structure and to evaluate the condition of the remaining bridge after the blasting of the adjacent one. With regard to their relevance for civil engineering issues the following key performance indicators were used:

- The bridge structure's relevant eigenfrequencies and corresponding mode-shapes
    - ⇨ Load bearing capacity and operability
    - ⇨ Evaluation of the bearings
    - ⇨ Distribution of the global and local dynamic structural stiffness in the bridge's lengthwise and transversal direction
- Sensitivity analysis to investigate the progression, the character, the stability and probable changes in the energy content of the relevant eigenfreuquencies:
    - ⇨ Load bearing capacity and operability
    - ⇨ Localization of weak points with regard to fatigue threat
- Energy dissipation path in the structure's lengthwise direction:
    - ⇨ Dissipation of the induced vibration energy, localization of problematic sections
- Comparison of measured values with the results of the finite element model
    - ⇨ Reference to the undamaged initial condition

The following sketch (Figure 3) shows the measurement layout applied at the measurement in 2010. In total a sensor grid of 8 different positions was applied – enabling a comprehensive observation of the primary load bearing structure's characteristics regarding bending and torsional behaviour.

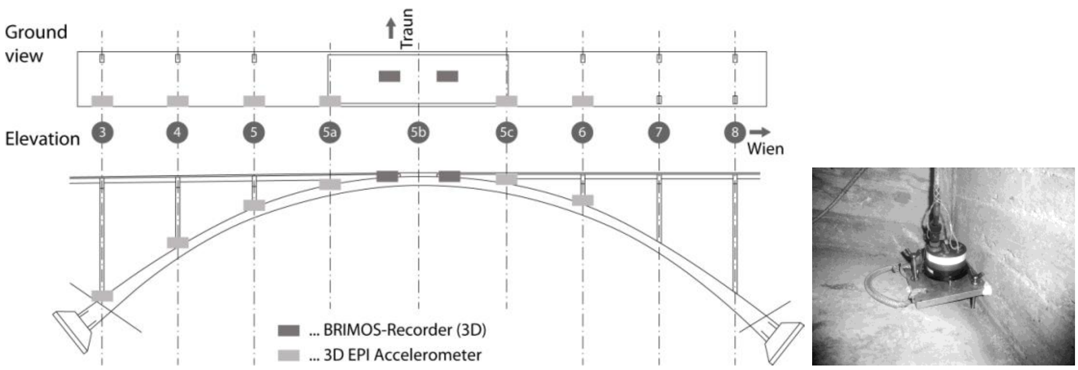

Figure 3: Sensorlayout (elevation and ground view) for the dynamic measurement of the arch – Driving direction Vienna.

Six of the eight sensors were part of the permanent online-measurement system. Those sensors (3D EPI accelerometer) were distributed in an alignment in the southern cell of the 3-cellular arch. Via sensor cables the information was transmitted to the monitoring center, located at the western abutment, where the measurement data were immediately observed and stored afterwards. The recorded files had a length of 5:30 minutes and a sampling rate of 500 Hz (= 2 milliseconds).

To have a back-up in case the permanent online measurement system would have failed two redundant sensor units (BRIMOS® Recorder with internal 3D acceleration sensor) were installed on both sides of the arch's vertex (distance from the western abutment x= 77,03 m and 98,33 m)

5 RESULTS OF THE MEASUREMENT

When comparing different measurements possible environmental influences have to be considered to avoid wrong interpretations of the acquired dynamic response (Veit-Egerer 2007). In particular the influence of the temperature can be relevant in this context. Due to the fact that - in the present case - the measurement in 2010 was done at a temperature of 15°C and the measurement in 2005 at 18-19°C potentially restraints caused by the temperature impact can be excluded.

5.1 Exceptional loading case bridge blasting vs. Regular traffic (freight traffic)

Figure 4 demonstrates the significant difference in terms of loading level between regular condition and the exceptional loading case. To quantify this deviation the relation between the effective measured acceleration amplitudes of the given normal load (traffic) and the exceptional load (blasting) was calculated. The data analysis revealed a vertical amplification up to a factor of 21 and a transversal amplification up to a factor of 15.

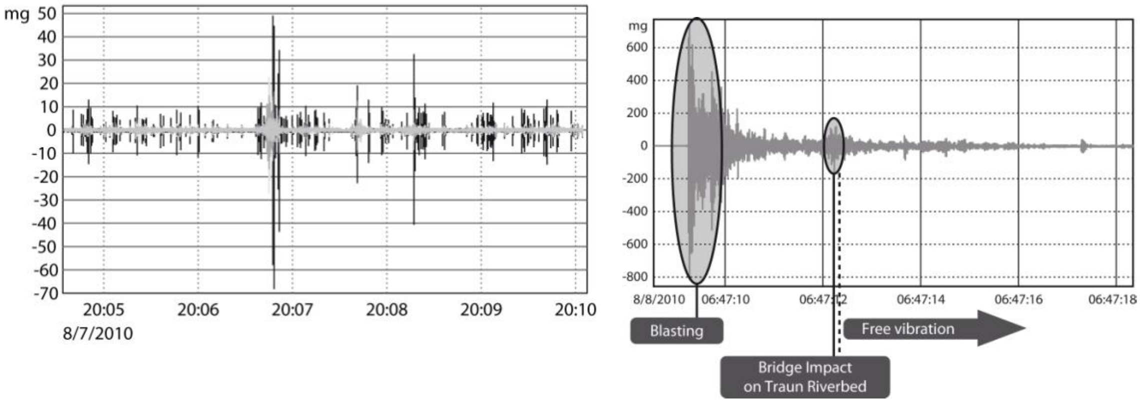

Figure 4: Characteristic acceleration signals under regular loading condition, all measured channels and detailed multi-stage loading history due to the blasting.

5.2 Eigenfrequencies

The eigenfrequencies extracted from the measurements can be understood as main indicators of the effective dynamic stiffness of the structure. If the blasting had any serious effects on the adjacent bridge changes in the frequency spectra would occur. In fact the comparison of the frequency spectra before and after the blasting shows only deviations of the amplitudes caused by the different excitation intensities. The structural stiffness itself remained stable – showing identical frequencies before and after the blasting. No extraordinary characteristics indicating limited operability or damage of the primary load bearing structure have been identified.

5.3 Dissipation of induced vibration

The energy dissipation path is a suitable indicator for the condition of a structure and the main girders respectively. “Problematic zones” mostly dissipate energy caused by friction which is reflected in an increase of the local damping values. Higher damping values in the range of abutments or piers are system-based and thus have no direct influence on the assessment of the structure’s condition. The BRIMOS® software uses the well-known Random Decrement Technique (RDT) and adapts it over time. For more details see (Wenzel 2009).

The pattern of damping values along the applied sensor grid (Figure 5) is typical for such a structural type. This applies to the results before the blasting and after the blasting also. The damping analysis primarily reflects the dominantly occurring system damping due to the mechanical behaviour of the bridge. Increased values at local spots according to material damping, which would point out certain damage of mechanically grave extend, were not determined.

The increased RDT damping values after the blasting are most likely a consequence of the detonation and results in a slightly accelerated consumption of the expected global lifetime. In fact this has not to be classified as critical.

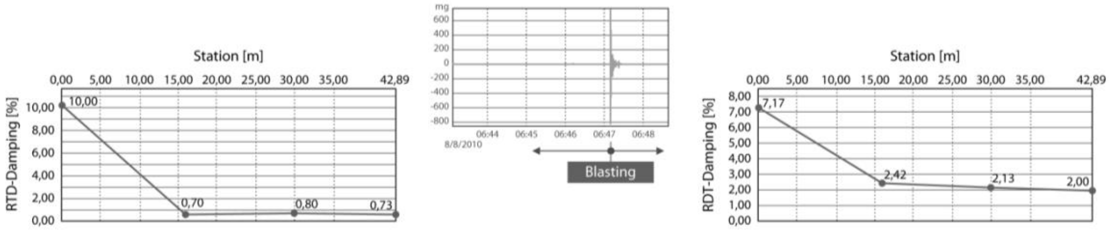

Figure 5: Pattern of damping values in the lengthwise direction: before the blasting (left) vs. after the blasting (right).

5.4 Evaluation of structural integrity – Trend of stiffness over time

So-called trend cards are an essential evaluation instrument in the context of full-scale measurements on bridge structures. Trend cards are obtained by evaluating frequency spectra taken from several measurements, telescoping them together and viewing them from above. For reasons of a better descriptiveness a two-dimensional visualisation is chosen. Showing the behaviour of the structure during certain timeframes of monitoring observations trend cards enable the identification and assessment of extraordinary behaviour.

Based on an initial dynamic measurement further periodic measurements provide the possibility to study the structure's maintenance condition in the course of time in order to identify remarkable changes of the dynamic structural response. A certain frequency of periodically repeated measurements assures the determination, observation and assessment of slowly progressing processes in the structure, which lead to damage or to deterioration of the structure's operational integrity. The derived patterns represent the effective structural dynamic stiffness related with the observation time. As an initial dynamic measurement at the Steyrermühl Bridge was done in 2005 it is possible to extract the trend of structural stiffness over the last six years in terms of a lifecycle curve (lifeline).

By means of the reference sensor (distance to the western abutment = 75.53 m) several trend cards were derived. The first one (Figure 6) shows the arch's relevant stiffness-patterns in the vertical direction in the range from 0.2 to 5.5 Hz over the entire measurement time period. By way of illustration it is referred to the corresponding mode shapes (global bending stiffness in vertical and transversal direction). Figure 7 represents the trend of eigenfrequencies in detail – in the range of 0.2 – 3.0 Hz and in the range of 4.2 – 5.5 Hz.

The whole investigation was subdivided into five major phases with regard to evaluate the effects of the blasting on the remaining bridge structure.

The following table (Table 1) points out those eigenfrequencies which have been considered for further evaluations for every single phase of the multi-level measurement program – starting with the initial measurement from 2005.

Table 1: Relevant eigenfrequencies for structural evaluation.

Eigenfrequencies	Phase I	Phase II	Phase III	Phase IV	Phase V	Δ Phase V to Phase II [%]
1. bending mode - vertical	1,34	1,49	1,57	1,53	1,44	-3,47
2. bending mode - vertical	2,73	2,77	2,78	2,79	2,76	-0,29
3. bending mode - vertical	4,78	4,50	4,60	4,54	4,52	0,35
2. bending mode - transversal	4,88	4,80	4,90	4,90	4,81	0,31

Each of the listed assessment phases is discussed in the following:

- Phase I: Regular traffic 2005
- Phase II (to be compared with Phase I): Regular traffic 2010 – Saturday evening & night

The first vertical bending mode measured in 2010 is significantly higher than the one determined in 2005 – due to a change in structural properties. In the course of the traffic assignment three parallel alignments of interlocked Jersey Profiles (concrete guide rails) were distributed lengthwise along the bridge structure – additionally contributing to the cross-sectional stiffness in comparison to the initial measurement in 2005. All the other relevant eigenfrequencies are excited more intensively due to the increased traffic volume (traffic assignment) but are keeping their frequency values.

- Phase III: closed bridge immediately before the blasting – Sunday morning:

In this phase the bridge was already closed - the excitation consists of ambient vibration only. Without the impact of traffic which is normally the most significant loading source the observed frequencies increase. The absence of freight traffic goes hand in hand with a loss of additional effective mass on the structure (less mass – constant structural load bearing resistance). As the – normally – dominant vertical excitation is missing the vertical modes are less distinctive now, whereas the dynamic mode in transversal direction (influenced by wind and microseismic excitation) is considerably dominant.

- Phase IV (to be compared with Phase III): closed bridge immediately after the blasting – Sunday morning:

In comparison to Phase III no frequency changes in terms of their absolute values occurred.

- Phase V (to be compared with Phase II): Regular traffic 2010 – Sunday forenoon:

In comparison to Phase II no frequency changes in terms of their absolute values occurred. In comparison to Phase IV a decrease of the relevant eigenfrequencies can be stated. This can be traced back to the fact that in Phase V the bridge was re-opened to traffic, so truck traffic - occasionally moving in queues – is acting as additional effective mass again (more mass – constant structural load bearing resistance).

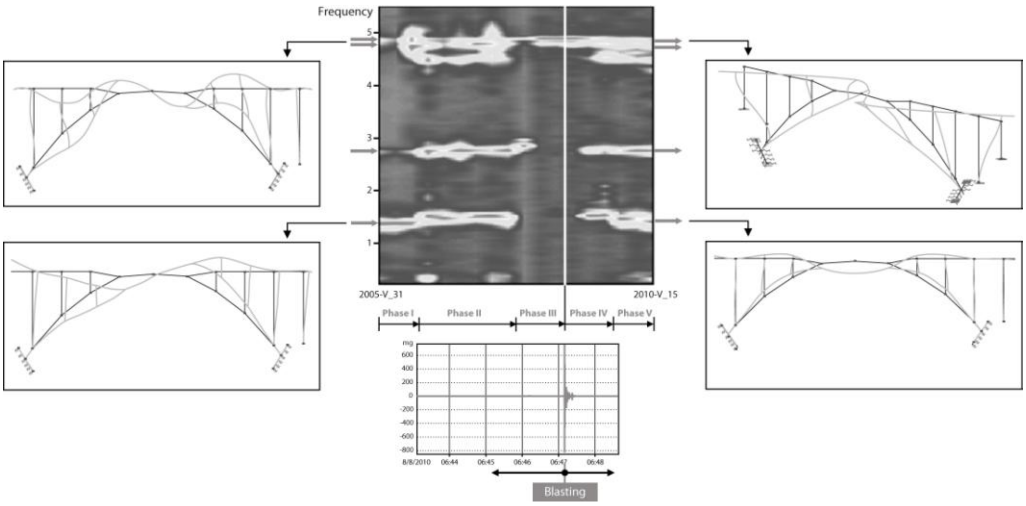

Figure 6: Trend of stiffness over time 2005 – 2010 (0.2 – 5.5 Hz) under the influence of the bridge blasting and after the re-opening to traffic.

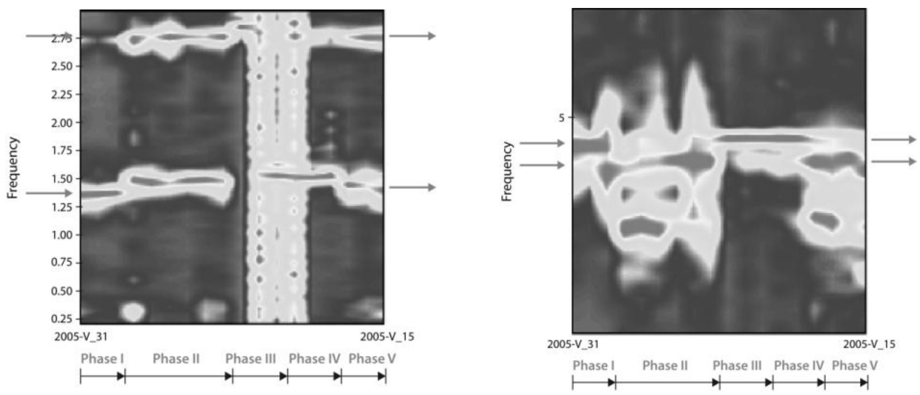

Figure 7: Trend of stiffness over time 2005 – 2010 (0.2 – 3.0 Hz left and 4.2 – 5.5 Hz right) under the influence of the bridge blasting and after the re-opening to traffic.

6 SUMMARY

The detailed assessment at the Traun Bridge Steyrermühl leads to the conclusion that the remaining structure withstood the blasting of the adjacent structure in good maintenance condition. At any time of investigation the structure's load-bearing capacity and its operability were available to a satisfactory extent. An imminent threat of collapse was not recognizable. According to the performed measurements no immediate action concerning traffic loading restrictions or rehabilitation measures was required.

The findings are based on the analysis of the dynamic key performance indicators eigenfrequencies, mode shapes, RDT-damping pattern in the lengthwise direction of the bridge, the effective acceleration and the progression of the measured structural dynamic stiffness.

The trend of eigenfrequencies of the primary load bearing structure – comparable with the structural load bearing resistance – shows a stable, straight-lined progression during the entire measurement. This indicates a proper structural resistance and structural behaviour even if the bridge is exposed to changing loading conditions in particular under the influence of the exceptional loading.

Detailed analysis within the dynamic measurement campaign in 2010 and the comparison of the eigenfrequencies over the entire observation period of 5 years refer to a good global condition of the bridge. Extraordinary changes immediately after the bridge blast affecting the structural integrity were not detected. The only changes observed can be traced back to the varying traffic volume (normal traffic vs. closed bridge vs. queues) and changes in structural properties (Jersey profiles). These phenomena primarily reflect the structural behaviour under changing surrounding conditions but do not affect the evaluation of the structural condition.

The increased structural dissipation behaviour after the blasting is most likely a consequence of the detonation and results in a slightly accelerated consumption of the expected global lifetime.

In summary, it can be stated that the blasting of the adjacent bridge did not affect the analysed structure with regard to its further use.

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