Dynamic Characteristics of an Overpass Bridge during Destructive Test

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Introduction

Vibration measurement during destructive test is a rare an important opportunity for structural health monitoring. In such measurement damage is introduced systematically and structural responses are recorded. Vibration analysis and dynamic characterization of an overpass bridge during destructive test is presented. By applying system identification, evolution of dynamic characteristics caused by damage was observed. The results show the changes of natural frequencies are clearly visible, thus can be used as indicator of global damage presence, while the change in mode shapes can be used as the local damage indicator.

Bridge description and sensing system

The tested bridge is an overpass located in Reibersdorf, Upper Austria west side of Vienna Austria. It is a post-tensioned concrete bridge with the main span of 32 m, two side spans of 12 m, and the width of 6.6 m. The deck is continuous over the piers and is built into abutment (Figure 1). Although there were no known structural problems, the bridge had to be demolished to allow a space for additional lane of highway underneath. Before demolition series of vibration test was carried out by the Vienna Consulting Engineers (VCE).

The main source of ambient vibration was from highway traffic underneath the bridge. In the first day measurement for undamaged condition was conducted and the second day was for damaged condition. Sensing system consists of six triaxial-accelerometers CV-373 produced by Tokyo Sokushin. The sensors measured accelerations at six selected nodes using six sets of sensor arrangement. Two sensors were kept at the same place throughout measurement to provide reference for time-synchronization. The other four were roving sensors that moved from one end to the other end of the bridge to provide complete mode-shape of the bridge for undamaged condition. During damage test, the sensors were mainly placed on one side of the bridge near the location of damage.

Damage Scenarios

Damage was introduced to the structure by slicing the pier above the footing. The cutting was made twice, the first was 2cm layer of column (named hereafter as Damage 1) and the second was also 2cm (marked as Damage 2). When the pier had completely severed from the footing cap, a temporary support was gradually released until the pier was completely suspended. During this time, vertical settlement of the bridge was recorded continuously until the final settlement reached 2cm (Damage 3 and Damage 4). Finally, the pier was completely suspended and no further settlement was observed (Damage 5). At the last stage, a steel plate was inserted to close the gap between pier and the footing. In this condition the pier rests on the plate and the stage is named 'retrofitted' stage. Throughout damage process an ambient response of the bridge was recorded at the sampling rate of 100 Hz. The records were divided into 37 frames of 5 minutes signal.

Methodologies for vibration analysis

The ambient responses were analyzed in both time and frequency domain. In time-domain Eigensystem Realization Algorithm (ERA) is used to identify global modes of the structure before and after damage. In frequency domain Spectrogram analysis provides a close-up look on the frequency components of vibration during destructive test. It is presented as a plot of power-spectra-density (PSD) of acceleration computed at each data frame and normalized to the maximum value.

Results of Vibration Analysis

Figure 2 shows the spectrogram of Reference sensor throughout the measurement. The dominant frequencies at the girder in vertical direction appear in the range of 1-15 Hz. In the undamaged part, one can see four distinct vertical lines representing four natural frequencies within the range of 3 to 14 Hz. Starting from frame number 11 one can observe the leftward shift of natural frequencies especially the forth mode (13 Hz). The other modes show apparent shift starting from frame 20 onward, which correspond to the time when the bridge experienced 2cm of vertical settlement. Leftward frequency shift of the first, second and third mode continues until frame number 34. Largest shifts were observed at the time when the pier was completely suspended indicating the significant reduction of stiffness.

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Table I summarizes the result of ERA. For damage 3 and 4, natural frequencies of mode 2, 3 and 4 experience significant changes as denoted by frequency changes that are larger than the 95% confidence bound. During damage 5, the changes in natural frequencies of all modes become more significant. For damping ratio, there are slight increases in the mean values caused by damage. The averaged values of damping for all four modes were between 1.2 to 1.5 % for undamaged structure with small bound of 95% confidence. These values increase slightly up to 2% for damage 3, 4, and 5; and up to 2.7% for retrofitted condition.

Another important aspect is the effect of damage to the bridge mode shape FEM simulation suggested that the damage in pier support significant changes in mode shapes. This change is obvious because the pier that restraints vertical modal displacement on pier-girder connection does not function anymore in damage condition. Observation on identified mode shapes reveals similar outcome. In both modes, large modal displacements at the pier-girder connection during damage stages are observed, with the largest was in Damage 5.

Conclusions

Based on observation and analysis of vibration data, the following conclusions are drawn:

1. Pier settlement of a bridge –as it is simulated for damage in this study, affects overall stiffness of bridge significantly, as is evident by the significant change in frequency of low order mode.
2. The changes of frequency are followed by local change of mode shape that can be used as indicator in locating the damage.

Table I: Modal parameters of damaged bridge identified by ERA

Mode	Frequency (Hz)			Damping Ratio (%)		
	Damage 3&4	Damage 5	Retrofitted	Damage 3&4	Damage 5	Retrofitted
1st Bending Mode	3.90 (-0.12)	3.65 (-0.37)	3.93 (-0.09)	1.98 (0.61)	1.10 (0.27)	2.76 (1.39)
1st Torsion Mode	5.84 (-0.47)	5.22 (-1.09)	5.76 (-0.55)	2.14 (0.95)	1.72 (0.52)	1.93 (0.74)
2nd Bending Mode	9.21 (-0.44)	8.16 (-1.48)	9.04 (-0.6)	1.12 (0.18)	1.93 (0.63)	1.13 (0.18)
2nd Torsion Mode	11.76 (-1.61)	10.28 (-3.1)	11.06 (-2.31)	1.49 (0.04)	1.23 (-0.22)	1.47 (0.03)

Note: values in bracket denote the change from undamaged stage (- : decrease, +: increase)

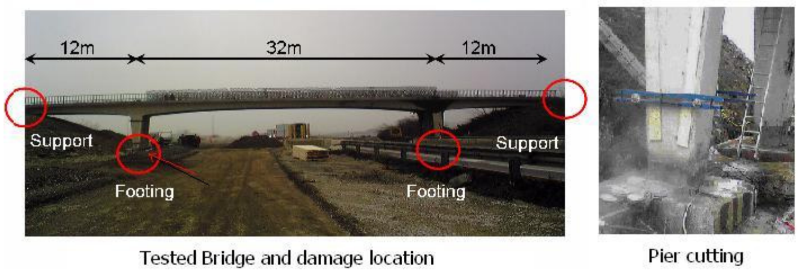

Figure 1. Tested Bridge and damage in progress

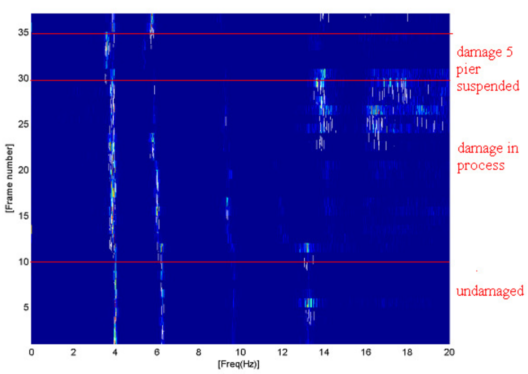

Figure 2 . Spectrogram of vertical acceleration

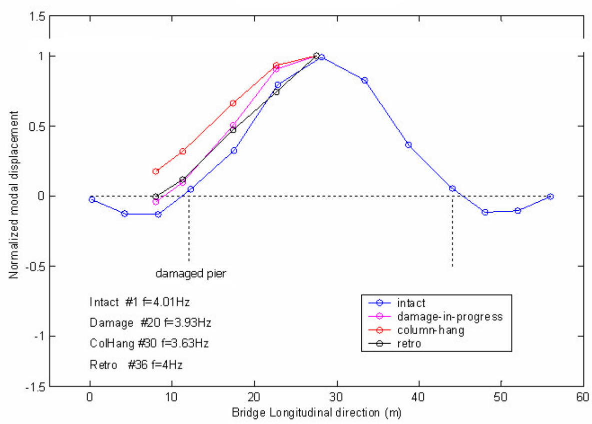

Figure 3. Shape variation of mode 1 caused by damage