Seismic Evaluation of a Large Network of Bridges by Simplified Automated Procedures and Consequent System Identifications by Dynamic Tests.

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ABSTRACT

This work presents the seismic evaluation of a large network of infrastructures located in the Veneto Region in the North-East of Italy. A large bridge database was subject to investigation, and simplified automated seismic valuation methods were developed in order to estimate the behavior of different types of structures located in seismic zones. In particular, analyses of infrastructures insistent in seismic zones, including surveys, investigations, seismic evaluation and seismic hazard assessment of the infrastructure in reference to parametric study of the structural vulnerability have been carried out. After estimating the safety factor of each structure based on the most vulnerable structural element, the key infrastructures on which execute system identification and simulate the response through numerical models were distinguished. In fact, the second step of this study is the structural identification of highly damaged bridges where a straightforward procedure has been applied. Static and modal parameters have been estimated for masonry arch bridges, concrete arch and continuous bridges, reticular and box girder steel bridges. The structural identification was used not only for calibration purposes but also for short and long term structural health monitoring (SHM) and damage detection. The SHM systems revealed good efficiency by maintaining the analyzed bridges open to traffic and constantly controlled.

INTRODUCTION

Many departments of transportation (DOTs) responsible for infrastructural networks have recognized the difficulties of the available bridge stock and their management, in which decisions are traditionally made only on a single project level and by visual inspections. As a result, a significant effort has been undertaken in many countries to develop bridge management systems with the aim of evaluating the condition of a single bridge in the global network level during its life cycle, and to provide, at the same time, efficiency information when allocating resources and establishing management policies in a bridge network. But to contain the inconveniences due to live-cycle damage it is necessary to study in depth the causes of damage of infrastructures and in particular of bridges. Old and historical bridges currently represent almost the entire European road and railway bridge stock. For many bridges, the intrinsic weakness of some structural elements, the deterioration occurrences and the updating of structural codes, evidenced inadequate structural performance and necessity to be upgraded to the standards of the current seismic codes.

The assessment of the actual structural behavior by means of experimental and theoretical investigations helps in choosing the proper intervention in terms of both materials and application techniques. In this framework, structural characterization via dynamic tests is immediate and cheap for not only understanding the structural behavior and damage of the structure but also for calibration and updating of numerical results [1]. Output-only identification technique, which is performed by just measuring the structural response under ambient excitation, is the most commonly used procedure to get information from the structures without exciting them on purpose, action which may cause non negligible inconveniences.

In the context of a large project developed between the University of Padua and the Regional Road Authority of Veneto, in the North-East of Italy, a great number of bridges were subject to structural investigation in order to examine their safety evaluation. The project initiated with the visual inspection and cataloguing of a total of 500 bridges of various structural kinds from masonry to reinforced and steel bridges, while this paper is focused only on 150 of them. The additional step was to extend the investigation on up to 80 bridges by using destructive and non-destructive tests in order to increase the material characterization of all kinds of structures available in the database. After that, a simplified procedure to evaluate the structural behavior of the entire fixed network has been developed by the researchers of University of Padua. During this step, parameterization of the safety levels and indexing of the structures not satisfying the National Code levels [2] was carried out. This phase also stimulated the individuality of the first bridges to execute deeper structural identification and to simulate the response through numerical models. For the structural identification part, the procedure utilized consists in [3]: investigation, damage survey and material characterization of the structure; vibrational tests for dynamic characterization; numerical modeling and calibration (trial-error methods); structural verification; installation of monitoring systems on critical bridges to control the behavior of the structure.

Sismic Evaluation by simpified procedures

Based on an action plan implemented by the DOT and Padua University, an extensive survey, aiming at a preliminary evaluation of the seismic capacity, was carried out on a stock of about 150 masonry, located in zones affected by high seismicity levels (Zone 2 and 3 according to the national zonation map with the maximum PGA on the reference stiff soil of 0.25-0.15g - peak ground accelerations are computed with a 10% probability of excedance in 50 years). Thus, all the bridges were investigated and consequently evaluated into their seismic performance under the national design standards.

It was possible to define not only the level of safety of the structure towards the seismic requirements by current codes, but also the amount of hypothetical costs of intervention necessary for the rehabilitation of the single bridge, and consequently the estimate of the distribution of the costs of intervention on the entire examined network.

The bridges have been classified according to the materials constituting the deck and vertical elements (masonry, concrete, steel, wood, stone), and the main structural types (simple support, arch, continuous girder, etc.). The choice of these structures is mainly based on the importance of the individual artefacts within the network, and therefore the potential discomfort that would cause their damage.

For each group, the structural elements and seismically vulnerable mechanisms were defined. In relation to each vulnerable element investigated, methods of analysis and verification criteria were established. Given the large amount of structures, for simply supported or continuous bridges, generally linear analyses were conducted, limiting the procedures of nonlinear static analysis to the piers and abutments. In particular, then, for masonry and unreinforced concrete arch bridges, the capacity curve has iteratively been obtained through the research of the minimum multiplier of horizontal loads.

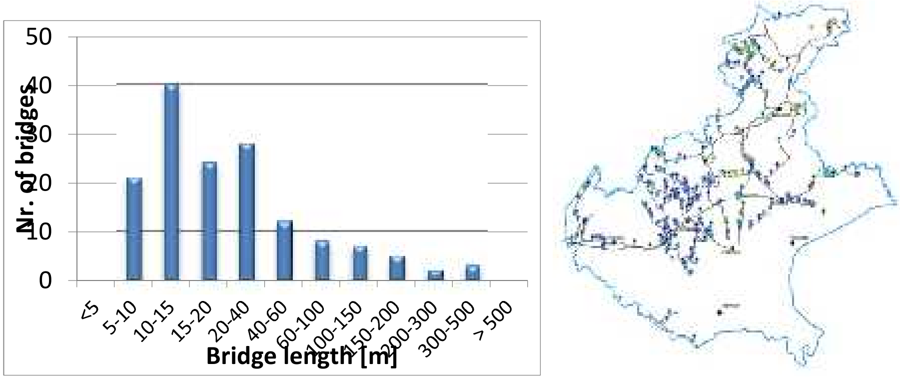


Figure 1 The analyzed network of bridges: for span and for road classification.

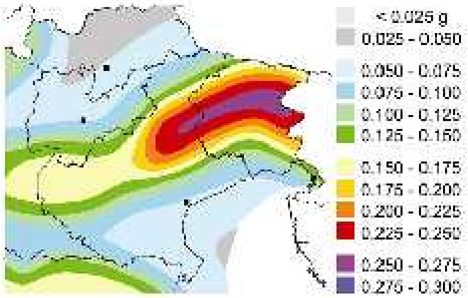


Figure 2 Seismic zonation of the North-East of Italy based on DM 14.01.2008 [2].

Figure 3 150 bridges subdivided by constituting material and deck typology.

From the graphs it can be seen that most of the bridges are in reinforced concrete, with simple support deck and with a single span. A smaller percentage, but nevertheless significant, regards masonry arch bridges. Consequently, for such types, it was decided to prepare and consolidate a semi-automatic procedure that can provide quantitative results on the large number of bridges.

Seismic evaluation

The first level of analysis coincided with the retrieval of information necessary to conduct a satisfactory seismic verification. Mainly the available sources were of two types:

a) Finding the original project.

b) Surveys and investigations in-situ.

In particular have been identified two main groups of bridges, for which a typical report was built:

- Bridges with reinforced concrete simply supported beam;

- Arch masonry, stone and unreinforced concrete Bridges.

To assess the reliability of simplified methods, comparative analysis of finite element models for some reference bridges were conducted. The methodology, described below, shows the different choices made according to the characteristics of the bridge and the type of information available. The primary goal was to determine if the structure was able or not to resist the seismic action. This objective has been translated in the calculation of the ratio between the maximum ground acceleration (PGA) bearable by the actual structure and expected acceleration in accordance with current regulations, with respect to the three limit states provided: Limit State of Light Damage, Limit State of Severe Damage and Limit State of Collapse:

$$g = \frac{PGA_{res}}{PGA_{act}}. \qquad (1)$$

The choice of carrying out the verification by comparing the PGA is explained in the need to determine a homogenous parameter and comparable for all types of bridges or elements. The determination of the PGA_{res} of the structure took place starting from the generic resistant parameter (bending moment, displacement, rotation, etc.) and working backward to the corresponding seismic excitation. In the case of linear analysis, it is possible to determine the PGA_{res} from the formula:

$$PGA_{res} = \gamma \cdot PGA_{act} = \frac{R_d}{E_d} \cdot PGA_{act} \qquad (2)$$

where R_d and E_d are the controlling resistant and acting parameters.

Simplified analysis (Fig. 4) were conducted with linear equivalent static method for these verifications: a) Compression and bending, bending and shear of the abutments; b) Overturning and slipping of the abutments; c) Compression and bending, bending and shear of the piers; d) Overturning and slipping of the piers.

In order to find the multiplier of the loads for the corresponding limit states, an iterative linear equivalent static method has been conducted for a) triggering of the collapse mechanism of the arch; b) triggering the collapse mechanism of the spandrel walls.

Typically, problems were observed on the spandrel walls and masonry abutments, vulnerable to seismic loads, while concrete structures usually had a good performance.

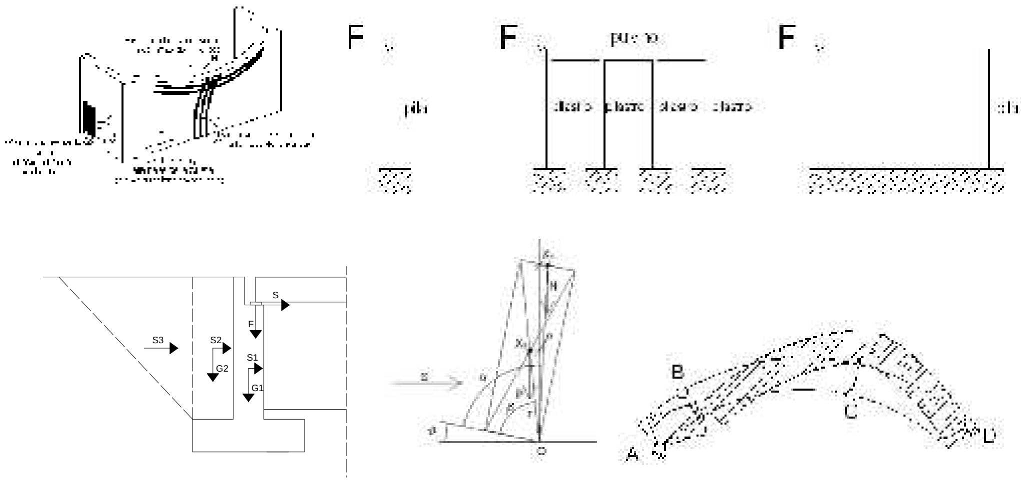


Figure 4 Verification schemes of some of the models utilized for the seismic analysis.

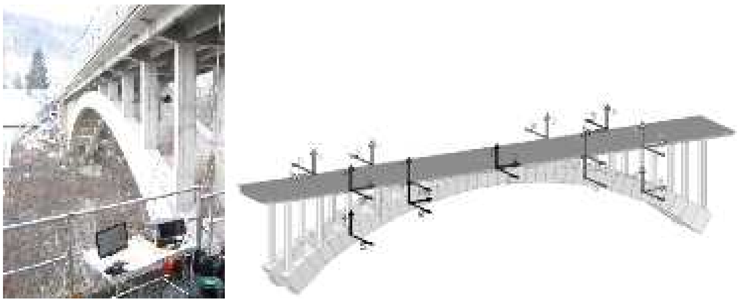
System identification and shm of bridges

The first aim of this section is to present the behavior of different kinds of infrastructures under dynamic identification tests, emphasizing the particular response of different bridges and their model calibration. Therefore, we dealt with the modal analysis through different stochastic system identification for different kinds of structures as slender and squat concrete arch bridges, masonry arch bridges, concrete girder bridges, reticular steel bridges and orthotropic steel box bridges.

After performing acquisition campaigns for each structure, a large number of modal techniques available in the literature, going from the Frequency Domain Decomposition (FDD) [4] and Stochastic Subspace Identification methods (SSI) [5] were used here for modal parameter extraction. Further developments have also been implemented in order to improve the performance of system identification techniques. In particular, advanced Cluster analysis, automatic algorithms for modal parameter identification and regressive models have been applied for accurate data processing [3].

Concrete Arch Bridges

The first bridges presented are two similar concrete arch bridges with single span of 70 and 80m. Both bridges (constructed in 1950-60) are supported by tapered arches bearing the concrete deck through concrete columns. In order to obtain a good spatial resolution, the survey resulted in eight different setups. Piezoelectric accelerometers with vertical axes were used to measure the bridge’s response. Three accelerometers were placed as a reference in the middle of the bridges and eight others were simultaneously moved to cover the entire area of the bridge. Figure 5 and 6 represent the identification results with the FDD technique. Identified clear peaks together with mode shapes were successfully utilized to calibrate the FE models for seismic analysis. From the obtained results (modal model and updated FE model), it can be observed that the theoretical models describe accurately the mode shapes extracted from the FDD with an average error of 4.6%.



Mode	FDD f (Hz)	Model 1 (Straus7)		
		f (Hz)	Δ (%)	MAC
1	3.369	3.402	1.039	0.967
2	4.565	4.952	8.478	0.843
3	6.201	5.636	-9.111	0.645
4	8.472	8.332	-1.653	-
5	8.96	9.187	2.533	-
6	10.91	10.520	-3.575	-
7	12.04	12.540	4.153	0.610
8	13.94	13.630	-2.224	-
9	16.5	14.800	-10.303	0.287
10	17.21	17.050	-0.930	0.347
11	21.68	20.550	-5.212	0.521

Figure 5 Installation, sensor locations and modes identified for one of the bridge tests.

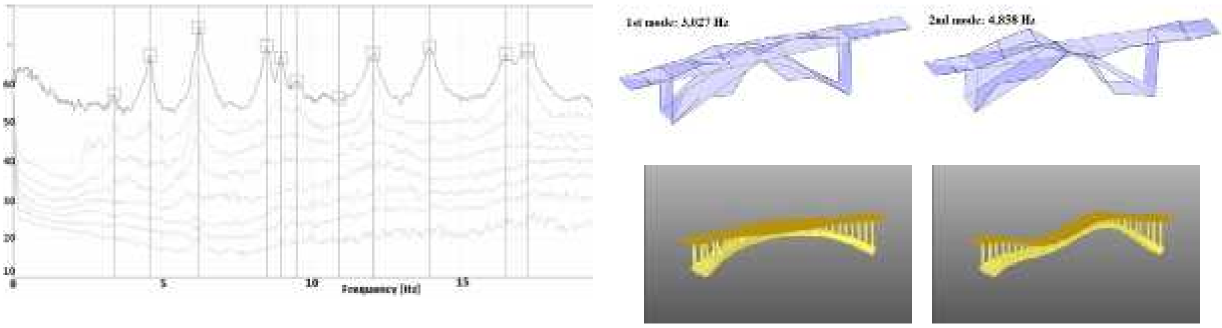


Figure 6 FDD method for the identification of structural modes on arch bridges and comparison with the FE models.

Masonry Arch Bridges

Two masonry arch bridges were subject to dynamic tests under ambient excitation. The first one is a high-rise arch bridge with slender piers, whereas the second one has low rise arch and low piers. Consequently, the first structure has a flexible behavior while the second one is a very squat structure. Thus, modal techniques performed in different ways on the structures. The analyses conducted with FDD and SSI included frequencies corresponding to the first 7 eigenmodes. Despite calibrating its FE models (Fig. 7), the first bridge was subject to seismic retrofitting and the dynamic identification served also to validate such intervention. Fig. 8 shows the identified modes after the retrofit and the increase of frequencies in comparison to the pre-intervention state. By comparing the results of the two dynamic models, it was observed that the intervention improved the seismic response of the bridge.

Figure 7 Sensor installation and model calibration in the first masonry arch bridge.

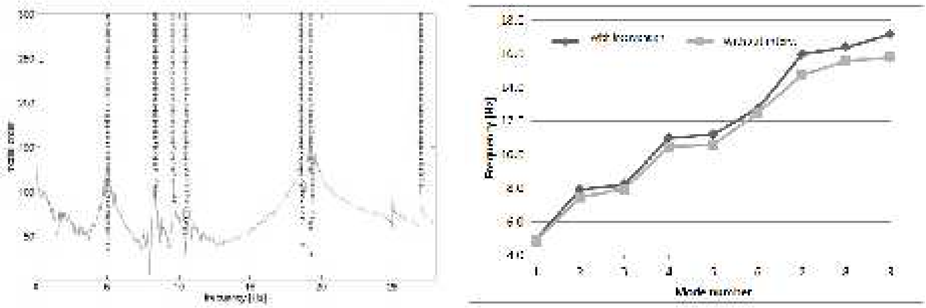


Figure 8 Sensor installation and model calibration in the first masonry arch bridge.

The Liberty Bridge is a bridge of strategic importance, being the only road bridge that connects the city of Venice to the mainland. The height of the piers is 2 meters; the span of 10.63 m; the arch rise is 1.31 m, which demonstrate the squat and stiff structure. As the first step, the FDD was used to analyze the measured data. Difficulties in extracting the exact structural modes were revealed since the graph did not present clear peaks as in the previous cases (Fig. 9). So a valuable solution was developed for squat structures using a cluster analysis (SSI analyses, with different block number and model's order, clustered) able to estimate all structural modes more precisely (Fig. 9).

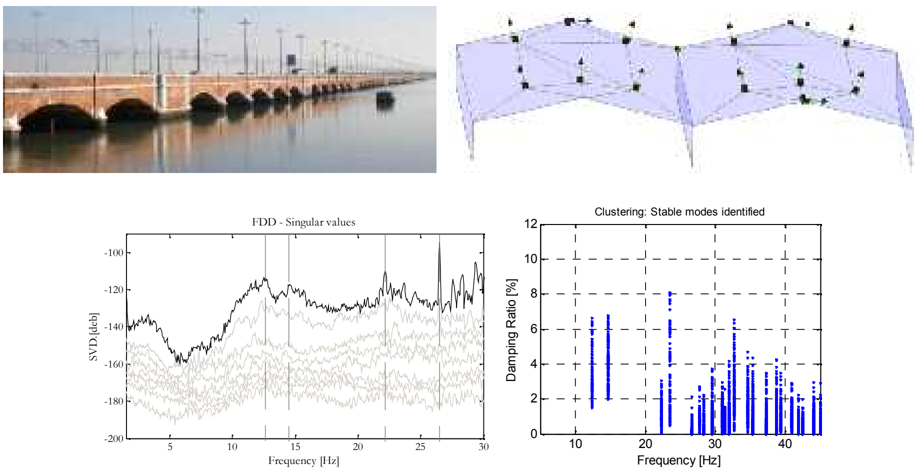


Figure 9 FDD and Stabilization diagram obtained from the cluster analysis.

Steel Bridges

When we talk about steel structures and in particular for steel bridges, we know that they are very flexible and as a consequence easy to identify by ambient vibration methods. In both reticular bridges analyzed, principal modes (first transversal and second vertical) were identified using both time and frequency domain techniques that showed excellent results. In fact, using the stabilization diagram, it appeared to be really simple to estimate the stable structural modes, and at the same time peaks of the FDD appeared to be very sharp. Experimental and theoretical models, in this case, were perfectly correlated (Fig. 11).

Figure 10 Instrumentation installation and setup configurations on reticular bridges.

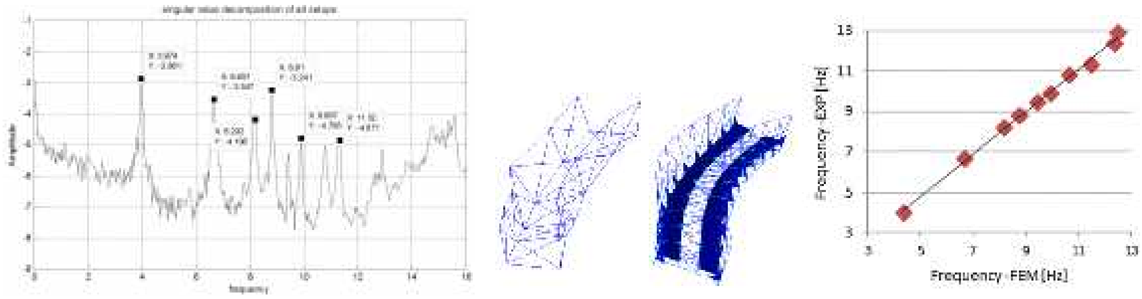


Figure 11 Frequency identification; comparison between Exp. and FE mode shapes.

Remaining in the field of steel bridges, when we talk about box girder structures, it reminds us of their massive orthotropic plates forming the caisson. Generally, all Operational Modal Analyses (OMA) performed well in identifying structural modes. Since the system identification aim, in this case, was to update a FE model that could assess the deterioration under fatigue, together with the accelerometers, a strain monitoring system composed of 56 strain gauges was installed.

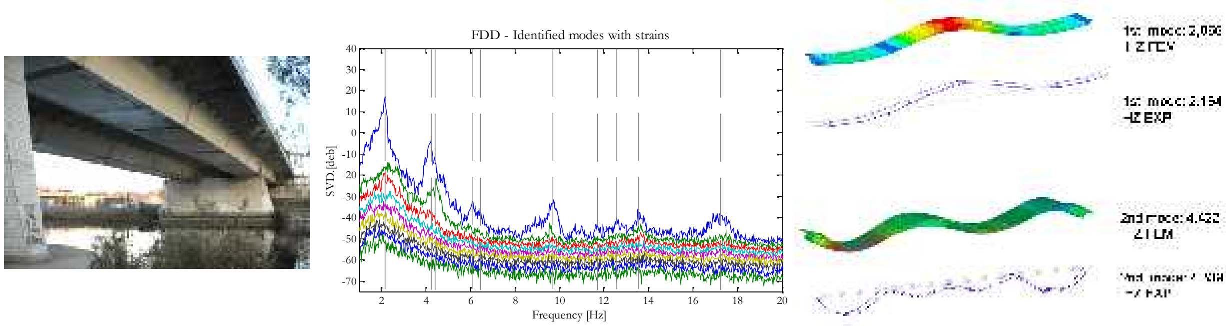


Figure 12 FDD identification through strain measurements; Exp. and FE mode shapes.

Besides the fatigue monitoring, a challenging step was to obtain dynamic results from strain data. Ten principal structural modes were identified and compared with the conventional analysis of acceleration data (Fig. 12). This was confirmed by the automated mode estimation developed in order to check the effectiveness of modal analysis by strain data. Thus, all principal modes were tracked during the months of monitoring, showing a stable behavior.

Concrete Beam Bridges

Simply supported or continuous RC bridges are not as hard to identify as masonry bridges, but harder than steel bridges. After installing a permanent SHM system on a concrete bridge, the local and global behavior of the damaged structure was monitored under traffic load, environment effects and under seismic events as well. The implementation of automatic dynamic identification algorithms made it easy to control the modal parameters on time and to develop regression models that permitted the possible damage detection (Fig. 13).

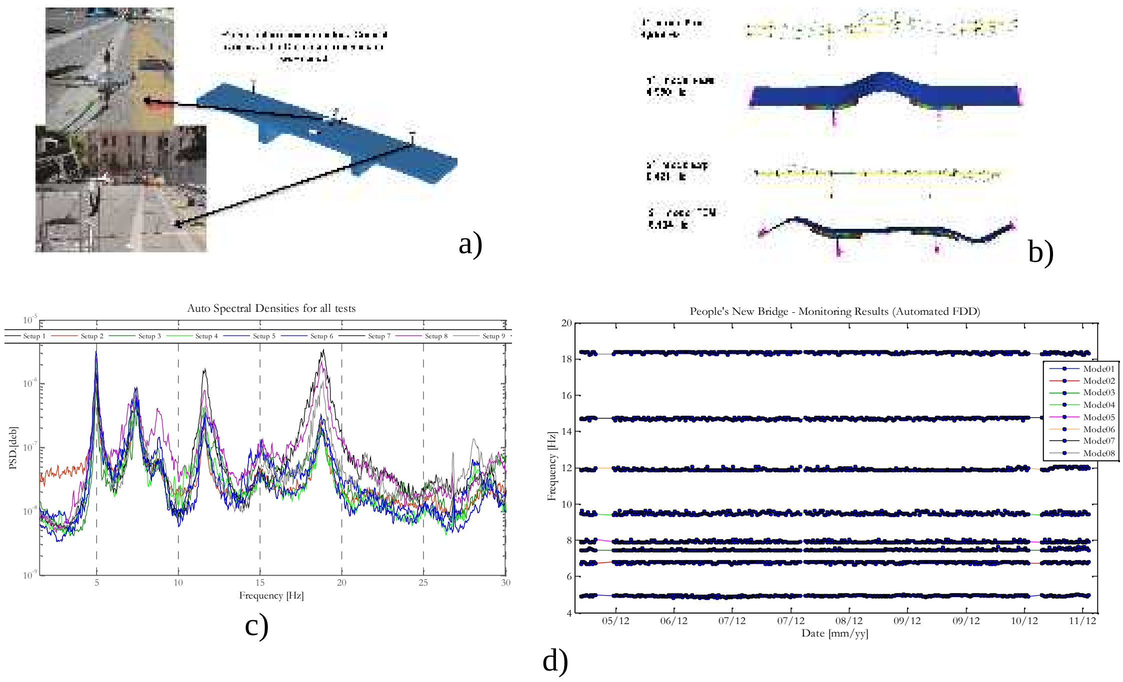


Figure 13 Installation, mode shapes, frequency monitoring through Automated FDD.

Conclusion

A large network of bridges was subject to investigation, and simplified seismic valuation methods were developed in order to estimate the behavior of different types of structures located in seismic zones in North-East of Italy. One by one determination of the state of consistency of the great number of bridges, their retrofitting and cost of intervention were carried out in order to develop the seismic fragility of the network. After these theoretical dynamic analyses, 10 bridges were subject to system identification and monitoring for comparing the dynamic behavior of different kinds of structures, comparing the sensitivity (or performance) of various modal techniques applied, for model calibration and for damage detection. We have examined the response of slender concrete arch bridges, masonry arch bridges, simply supported concrete bridges, reticular steel bridges and steel box girder bridges. In cases were standard OMA techniques did not perform well, new methods have been proposed to better detect the real dynamic behavior of complex bridges and monitoring systems allowed to control the structural behavior.

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Recent Seismic Activity of the Lezha-Ulqini Seismogenic Zone and its Associated Hazard

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ABSTRACT

The results of the analysis, based on the parameters of events and some features of seismicity that have occurred in the Lezha-Ulqini seismogenic zone during period of time 2001-2012, are presented in this paper. This seismogenic zone presents a significant seismic hazard to those living in northwestern Albania, southern Montenegro not only due to the pending earthquake but also due to a lots of earthquakes certain to follow the mainshock. In total, 112 earthquakes are registered during overmentioned period in this zone, and one with M_L=5.0 (Richter) occurred on 21 August 2009. Lezha-Ulqini seismogenic zone present a threat to nearby urban areas in Albania and the Montenegro. The goal of this study is to determine tipology of seismicity, the source parameters of the mainshocks and their aftershocks in order to shed light on the seismotectonics of the area on the stress field and to evaluate the seismic hazard. The region affected by the August 2009 sequence, together with the seismogenic region of the 15 Aprile 1979 Ulqini event (M7.0), forms a roughly NW−SE-trending active seismotectonic zone in western Albania and continues through southern Montenegro.

KEY WORDS: Seismicity, Adriatic Sea earthquake, focal mechanism, faults, aftershocks

I. INTRODUCTION

The Albanian mountain belt is a segment of the Dinarides-Hellenides orogeny that trends NNW–SSE (Fig. 1). It was formed by Alpine orogenic processes related to the Apulia and Eurasia convergence and the closure of the Mesozoic Tethyan Ocean [1],[5]. In 2009 one moderate earthquake of 21 August hit Northwestern Albania with intensity I_0=VI-VII (MSK-64) (Fig. 2). This earthquake highlight the increased seismic activity of the Lezha-Ulqinseismogenic zones in 2009. During the last century, several devastating earthquakes have occurred, causing casualties and substantial damage [15].
The epicenter of the M=5.0, 21 August 2009 earthquake, which caused no damage, was 17 km offshore west Albania in the Adriatic Sea. The most recent previous strong event in this region was the Ulqini earthquake of 15 April 1979 (GMT 06:19:40; Ms7.1; 41.94° N 19.408° E; h=8 km), which caused extensive damage along 100 km of the coastline, killed 129 people, injured 1554 people in the former Yugoslavia and Albania, and left more than 80,000 homeless [13].During this earthquake were observed tsunami wave with an amplitude 0.5-1 m in the Ulqin coastline. Other stronger earthquakes on record in the Lezha-Ulqini-Shkodra zone affecting Albania and Montenegro, such as the M_S6.9 earthquake on January 13, 1563; the M_S7.2 July 25, 1608 earthquake; the M_S7.2 and I_0=X(ten) degrees April 6 1667 earthquake; the