ACOUSTIC EMISSION MONITORING OF REINFORCED CONCRETE FRAME DURING SEISMIC LOADING

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Abstract

Acoustic emission (AE) monitoring was performed during pseudo-dynamic testing of an old, two-storey, one-by-one bay reinforced concrete frame structure. The structure represented a 0.7-scale model of a real-size frame structure designed and detailed according to the standards prevailing in Southern Europe in the 60's without engineered earthquake resistance. Real-time monitoring of AE activity versus the complex applied load resulted in semi-quantitative damage characterization as well as comparative evaluation of the damage evolution of the different size columns. Evolution of the AE energy rate per channel, as revealed from zonal location, and the energy rate of linearly located sources enabled the identification of damage areas and the forecast of crack locations before cracks were visible with naked eye. In addition to that, the results of post-processing evaluation allowed for the verification of the witnessed damaged areas and formed the basis for quantitative assessment of damage criticality.

Keywords: Reinforced concrete structures, earthquake damage assessment, pseudo dynamic loading, real-time monitoring.

Introduction

Old, substandard RC buildings designed in the 60’s on the basis of vertical loads only are often characterized by irregular distribution of strength and stiffness in-plan. These structural characteristics, usually dictated by architectural requirements, are coupled with those owing to the design according to non-seismic design codes of the 60-70’s in Southern Europe (e.g., insufficient detailing of reinforcement, low concrete strength) result in structures with increased vulnerability to earthquakes. An experimental program of a 0.7-scale model of a real-size frame structure designed and detailed as old RC buildings in Greece, without engineered earthquake resistance, was carried out at the Structures Laboratory of the Department of Civil Engineering at the University of Patras, employing the pseudo-dynamic testing method. To represent the actual torsional seismic response of such a structure, pseudo-dynamic testing was performed with four degrees of freedom (DOF): the displacements of the two floors in the direction of the actuators, plus the two floor rotations with respect to the vertical axis. The structure was excited by a 15-sec-long unidirectional input motion that fitted well with the 5%-elastic spectrum of Eurocode 8 modulated after one component of the Herzegnovi record in the 1979 Montenegro earthquake.

Due to their sub-standard design, the columns of the building, both at ground and upper floor, exhibited low strength and ductility capacity and, thus, damage was expected to develop first at their ends, due to the insufficient reinforcement overlapping length there. Acoustic emission was used as an NDT method, to monitor damage development at these vulnerable areas. Due to the complex nature of the applied loading and structural response, techniques previously proposed (Matsuyama et al., 1993, Yuyama et al., 1999) for AE evaluation during controlled stimulus, are not directly applicable. Different source location techniques (zonal, linear and 3-D location) were
applied in order to identify potential high risk areas of increased AE activity during loading. A summary of the experimental procedure and the derived results are reported and discussed herein. Comparison of the differences detected in AE behavior among the different columns as well as between the base and the 1st floor joints is performed and the results are correlated with the applied load. Further processing of the located AE events, enabled the verification of the damage areas observed and the assessment of the building structural integrity.

**Experimental Set Up**

A nearly full-scale model simulating the structural configuration and seismic response of real-size reinforced concrete structures was constructed and tested. The structure comprised of two floors and one bay per direction, while its four columns formed pairs of unequal resistance and stiffness along the longitudinal (X) direction of the structure. The plan dimensions of the specimen structure were 3.15 x 3.85 m and the floor height was 2.30 m and 2.0 m at ground and top floor, respectively as presented in Fig. 1. To create an asymmetric structure that responds with torsion under translational ground motions, columns of one side were one-half size of those of the other side. All columns were 0.175-m wide while their depth varied from 0.175 m for the pair of columns at one side of the structure, to 0.35 m for the opposite pair. Columns were cast on spread footings anchored on the strong floor of the laboratory and were longitudinally reinforced with four 10-mm-diameter smooth (S220) bars for the columns with the smaller section, and six 10-mm-diameter bars for the larger ones. Transverse reinforcement consisted of 6-mm-diameter smooth (S220) bars placed at 200-mm apart. The beams and the slabs were heavily reinforced so that inelastic response would appear first on columns. A mean concrete strength of 22.8 MPa and 18.05 MPa was obtained on the day of testing at the ground and upper floor, respectively.

![Fig. 1. Specimen layout, left, and AE sensor positions, right, (AE channels. 1-8 placed on column 1, ch. 9-16 on column 2, ch. 17-24 on column 3 and ch. 25-32 on column 4.).](image)

Prior to AE monitoring, the structure was subjected to the selected earthquake record scaled at 0.30 g. During this initial testing phase, the structure exhibited strong torsional response,
which - contrary to current beliefs based on simplistic analytical studies - induced larger displacements on the more flexible side of the building. Columns were subjected to biaxial flexure and exhibited damage concentrated at the base of smaller ground-floor columns, owing to the inadequate force transfer along the small bar overlapping length and slippage of vertical bars within the footing. Beams were over-reinforced – as it was common in this category of structures – and displayed no damage. Some hairline diagonal cracks were observed on the sides of the larger columns, owing to the development of torsional moments in these members, which closed after the test. Some cracks formed also at the beam-column interface due to slippage of the (smooth) column reinforcing bars within the joint. To reinstate the torsional balance, smaller columns (denoted here as columns 1 and 2) were reinforced via a 0.05-m-width concrete jacket cast from the footing to the top of the structure. The structure was excited again with the same seismic record at 0.30 g, while being monitored with AE.

Fig. 2. Retrofitted structure: slab displacement (meters) at the ground and the top floor (top); slab rotation (rads) of the ground and the top floor (bottom).
The observation of the structure during testing and the recorded displacement and rotation data (Fig. 2) reveal that the torsional response almost disappeared and the building displaced along the axis of excitation. The observed damage consisted of cracking in the larger columns and spalling of the concrete cover due to and along the lap-spliced bars at the base of the columns. Cracking also developed at the beam-column joints at the ground and the top floor. During the later stages of the excitation though - and due to the damage inflicted on the larger columns - the torsional response re-appeared, albeit at very low amplitude.

Acoustic emission monitoring was performed using a 40-channel PAC-DiSP (PAC, 2001) system. The sensors used were standard PAC resonant type R151, 150-kHz resonance with 40-dB integral preamplifier. The position of the AE sensors and the points of applied deformation are shown in Fig. 1. Sensor position was selected in such a way to permit linear location along the column height and linear x-y location at the 1st floor joints. Prior to the test the AE sensors were calibrated and the velocity of the AE signals was calculated in order to achieve the best possible sensor set-up sufficiently covering the areas with expected high AE activity. In addition to that the attenuation of the AE signals was measured, Fig. 3.

**Acoustic Emission Results**

Prior to the actual test a preliminary, trial loading was performed at low load levels. The maximum load of the preliminary test was 2% of the load that the building was eventually loaded. The AE activity that was recorded during the preliminary test was relatively low with low energy levels, as presented in Fig. 4. This preliminary test resulted in useful information concerning the background noise of the system as well as for setting up acquisition parameters for the AE system. In addition to that, the activity recorded in columns 1 and 2, due to the friction between the jackets and the concrete frame, was useful during the analysis phase in discriminating genuine AE signals from friction and defining damage evaluation levels.

The force exerted by each one of the four actuators is presented in Figs. 5 and 6 (four background plots for actuators 1 to 4, respectively) together with AE energy rate from each column. After jacketing the flexible columns, the building response was mainly uniaxial. The complex nature of loading (cyclic loading with varying amplitude and phase shift between actuators) resulted in complicated structural displacements and render infeasible the direct correlation between the applied loading and the AE activity. The majority of the AE activity, around 98%, is emitted from columns 1 and 2. As can be seen from Figs. 5 and 6, the AE activity starts at a deformation level exceeding that which caused the damage during the past loading.
Fig. 4. Applied load (actuator 1) and total energy for the 2% test.

Fig. 5. Applied load at actuators 1 & 2 (background plot) and AE energy from flexible side column 1 (channels 1-8) and for column 2 (channels 9-16), respectively. (initial test at 0.30 g). Direct comparison between past and present loading, as a means to calculate Felicity effect, is not possible due to the different nature of applied loads as well as the addition of concrete jacket on columns 1 and 2. Moreover, it is worth noting that there is an absence of AE activity during a displacement hold at relative high force level between 5000-5800 sec, implying no creep effects during that period.

Finally, it is worth noting that the energy scale for columns 3 and 4 in Fig. 6 is one order of magnitude lower than that for columns 1 and 2 in Fig. 5.
Fig. 6. Applied load at actuators 3 & 4 (background plot) and AE energy from stiff columns 3 (channels 17-24) and 4 (channels 25-32), respectively.

Source Location and Correlation with Visual Inspection

During the entire test high AE data rates were recorded on most channels covering the structure, especially on the channels attached on flexible columns 1 & 2 after they had been externally reinforced by means of concrete jacketing. The use of various AE signal source-location algorithms provided the necessary focus of the analysis to locate the main areas where damage developed and to find areas with high likelihood of failure. A complex source-location strategy was employed with each column being treated as a separate entity to avoid signal time overlap and event mirroring. The basic source location algorithms used to locate damage was the Linear X-Y and Linear 3D modes in Noesis AE Data Analysis software (Envirocoustics SA, 2005). Typical location results for column 1 are presented in Fig. 7. The height of column 1 is represented on the horizontal axis of the plot, starting from sensor #8 (0 m column bottom) up to sensor #1 (3 m, top area of column) and sensors #3 and #4 projected also on the x-axis.

The linear location strategies showed the areas sustaining damage during the test. This damage is attributed to cracks and hairline cracks or crack face friction in the columns and the reinforcing jacket. The two un-strengthened columns (columns 3 and 4) showed little located AE activity compared to the other two columns. Any activity was mainly from the joint at the first floor and more specifically in the area of beam-column-slab joint at the floor slab. Some activity was also recorded from the lower section of the columns. Columns 1 and 2 that had sustained damage during the initial earthquake record of 0.3 g and had been reinforced with concrete jackets produced significantly more AE activity, which corresponded to more clearly visible hairline cracking on the jacket itself. This is attributed to the eventual slippage between the initial
concrete and the jacket. Visual inspection did not yield any information about damage to the inner column. In this case also, the located activity was concentrated to the lower section of the joint and immediately below the joint of the 1st floor. This is clearly shown in Fig. 8 of Column 1, where the concentration of activity is immediately below the node is the most significant of any column in this test.

![AE energy for column 1 with respect to sensor location along column height.](image)

Fig. 7. AE energy for column 1 with respect to sensor location along column height.

This concentration of activity shows the areas that are actually subjected to damage during the test. It is generally estimated that any failure during the re-positioning of the structure to a higher deformation state will occur in these areas first. Visual inspection and correlation of visible cracks with AE located activity was not always possible during the test, since it was found by closer inspection later and the present data analysis that cracking, which was invisible to naked eye produced significant AE. Although no cracks of significant width were observed, either on the retrofitted or the un-retrofitted columns, the amount of AE recorded distinguishes these areas as subject to further damage. The success of AE to individualize, at an early stage, the specific areas, at which medium damage is manifested and most likely enhanced at higher drifts, was demonstrated in subsequent tests to 0.45 g and 0.55 g. These tests (not reported in this work) produced a level of damage sufficient to visually confirm the existence of concrete cracking and reinforcement slippage. Significant damage at the joint at the top of Column 1 mentioned above was observed and in other areas where AE had given early warning. More specifically, increased AE activity recorded from both the node and the base of column 2. Compared with the activity and events identified on column 1, the activity at the node of column 2 was lower while that at the base (channel 16) was higher.
Concerning the activity from the joints of columns 3 and 4, AE events of similar level, indicating the onset of emission, were recorded from both columns. For example, the events located on column 4 versus time are presented in Fig. 8. More specifically the AE activity is displayed for the full height of the column (y-axis, with sensor number) versus time (horizontal axis). For example, at approximately 800 sec, there is some activity located at joint (sensors #27, #28, #29). On the other hand, the base of column 4 (channels #31 and #32) emitted more signals resulting in well-defined sources. Figure 8, in conjunction with Fig. 6, provides important information about the behavior of column 4 and its correspondence to the level of structural deformation and time parameters. The successful correlation of located events, both in time and position, with the hairline cracks observed, is worth noting.

**Discussion and Conclusions**

A series of full-scale tests on retrofitted or un-reinforced concrete structures were performed, during which acoustic emission was monitored. The retrofitted columns of the building tested showed minor cracking on the reinforced concrete jacket with some damage developing at the base of the ground floor and the joints of the structure. Throughout the test, AE monitoring identified activity at the bottom of ground floor columns and the joints of the ground-floor beam-column intersection, thus, indicating the most damage prone areas. This was evident by the increased overall AE activity in these areas and significant activity concentration.

Results during subsequent loading tests under 0.40 g and 0.45 g seismic events (not reported here) proved that the AE method identified correctly the areas of damage as these were the exact locations where failure occurred (large cracks, concrete spalling and slippage of reinforcement).
Based on conventional analysis, AE data signatures from crack and reinforcement slippage cannot be easily separated. Future work utilizing displacement measurements at the various positions around the structure for correlation with AE activity versus time and position as well as unsupervised pattern recognition for the treatment of AE data, can provide further insight to the damage mechanisms (crack propagation, reinforcement slippage etc) and possibly yield quantitative damage growth assessment in similar structures and repair techniques.

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References


