Health monitoring of reinforced concrete slabs during seismic tests using acoustic emission

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Abstract

Under low-to-moderate intensity earthquakes, which are expected to occur several times during the lifetime of a building located in an earthquake-prone region, the reinforced concrete (RC) structures are designed to behave basically within the elastic range. Yet concrete cracking is certain to occur, since the tensile strength of this material is limited, and the cyclic reversals induced by the seismic action cause deterioration of the bond between concrete and reinforcing steel, leading to the slip of the reinforcement. Techniques that can alert as to the state of damage of a structure (concrete cracking and reinforcement slippage) without requiring regular and costly uncovering work are highly advisable. This paper describes the applicability of the AE (acoustic emission) technique for damage assessment of RC flat slabs subjected to seismic loads. It presents and discusses the AE recorded during a series of dynamic tests conducted with a 3×3 MTS shaking table installed at the University of Granada. The specimen represents, at the 1/3 scale, a flat slab supported on four box-type steel columns, and it was subjected to a simulation of the Campano-Lucano earthquake recorded at Calitri (Italy). The correlation between structural damage, expressed in terms of hysteretic strain energy, and the AE energy is discussed; and finally, some bases for developing formulae to evaluate the level of damage from AE measurements are suggested.

Introduction

One considerable source of damage in reinforced concrete (RC) structures when they are located in earthquake-prone areas is cyclic loading induced by ground acceleration during seismic events. These structures are commonly designed to sustain, essentially, two levels of seismic action: low-to-moderate intensity earthquakes (level I), and strong earthquakes (level II). Under level I earthquakes, the RC structure must remain basically elastic. In moderate or high seismicity regions, a RC structure can experience several tens of low-to-moderate intensity earthquakes during its lifetime. Under this level (I) of seismic action, concrete cracking is certain to occur due to the low tensile strength. One of the important consequences of concrete degradation under cyclic loading is the slip between reinforcing steel and concrete, which is considered serious damage to RC elements [1].

The simple visual inspection of the structure is complicated by the fact that it is usually covered up by non-structural elements. A simple visual inspection cannot provide quantitative information of the damage accumulated on the structure during the cyclic reversals, and it is here where non-destructive techniques can play an important role. The methodology based on Acoustic Emission (AE) proves very effective for this purpose [2].

The AE technique applied to RC elements has been investigated by several authors; most look into the problem insofar as the material (concrete) or individual elements are concerned (beams, columns) [1-4]. Scarcity is the application of the AE technique in monitoring the damage of RC
structures subjected to earthquake-type dynamic loadings; one of the examples of such research is that carried out by Carpinteri et al. [5] on masonry towers affected by regional seismicity.

This paper presents an investigation likewise based on monitoring the AE energy, but applied to assess the damage of RC slabs supported on columns and subjected to seismic-type cyclic loading demands up to the state of damage admissible for level I earthquakes. The cyclic loads were applied dynamically using a shake table that reproduced the ground acceleration recorded at Calitri (Italy) during the Campano-Lucano earthquake. In contrast to experiments conducted with static loads, in the case of dynamic shake table tests, many parts of the auxiliary set-up are in movement during the experiment, thus generating spurious AE from many diverse sources not related to concrete cracking and friction. Once this spurious AE is removed by filtering, the concrete damage is quantified in terms of AE energy and dissipated plastic strain energy. The limit of damage admissible for level I earthquakes is determined by the inception of the slippage between concrete and reinforcing steel, and by the onset of yielding on the reinforcing bars. As a result, a damage index is tentatively proposed that predicts the proximity to the above limit of damage based on AE monitoring.

**Experiment**

A prototype structure consisting of a RC slab supported on four box-type steel columns was designed according to Spanish codes. The prototype structure has one story 2.8m in height and 4.8×4.8m² in plan. It is assumed to be located in the moderate-seismicity Mediterranean area. Accordingly, from the prototype structure, the corresponding test model was derived by applying the following scaling factors for geometry, the acceleration and the stress, respectively: \( \lambda_l=1/2 \), \( \lambda_a=1 \) and \( \lambda_\sigma=1 \). Figure 1 shows the geometry and reinforcing details of the test model. The slab measures 125mm in depth and it is reinforced with two steel meshes, one on the top made with 6mm diameter bars spaced 100mm, and another on the bottom consisting of 10mm diameter bars spaced 75mm. The slab was reinforced at the corners by shearheads consisting of steel U-shapes 60mm in depth in order to prevent punching shear failure. The average yield stress \( f_y \) of the reinforcing steel was 467 MPa, and the average concrete strength, \( f_c=23.5\text{MPa} \).

The test model was placed on the uniaxial MTS 3×3m² shake table as indicated in Fig. 2. To satisfy the similitude requirements between prototype and test model, additional steel blocks were attached on the top of the RC slab (total mass \( m=7.39\text{Ns}^2/\text{mm} \)). The shake table motions were patterned after the Calitri 1980 NS earthquake (Campano-Lucano, Italy) with the time scale compressed by a factor of \( \lambda_t=(1/2)^{0.5}=0.707 \). Two series of seismic simulations were conducted using the Calitri 1980 NS accelerogram scaled to different amplitudes. The values of the peak acceleration (PA) applied in each simulation are summarized in the first column of Table 1, expressed in terms of the acceleration of gravity \( g \).

**Instrumentation**

The Vallen System ASMY-5 was used to measure AE during the tests. Eight AE low-frequency sensors (type VS30 set in the range 20-100 kHz) were placed on the test model as shown in Figure 1b. The threshold detection of the AE sensors was set at 45 dB. To prevent undesired noise generated by the contact between the base plate of the columns and the shake table, four guard sensors were placed near the bottom end of the columns as indicated in Figure 1a (one sensor in each column). Moreover, in order to remove or reduce the sources of spurious friction noise, rubber layers and teflon films were placed between the added steel blocks and the slab. Teflon films were also inserted between any metallic surfaces whose contact could generate spurious friction noise, such as screws, steel plates for fixing the accelerometers to the slab, cables, etc.
Fig. 1. Test model: (a) elevation; (b) plan (bottom view).

Fig. 2: Experimental set-up
Table 1: Seismic simulations

<table>
<thead>
<tr>
<th>Test series</th>
<th>PA (g)</th>
<th>$T$ (s)</th>
<th>$\xi$ (%)</th>
<th>$\varepsilon_{\text{max, reinf}} \times 10^6$</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simulation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A1</td>
<td>0.08</td>
<td>0.26</td>
<td>1.10</td>
<td>497</td>
<td>Columns remained elastic</td>
</tr>
<tr>
<td>B1</td>
<td>0.10</td>
<td>0.29</td>
<td>1.14</td>
<td>568</td>
<td>Columns remained elastic</td>
</tr>
<tr>
<td>C1</td>
<td>0.12</td>
<td>0.30</td>
<td>1.20</td>
<td>638</td>
<td>Columns remained elastic</td>
</tr>
<tr>
<td>D1</td>
<td>0.19</td>
<td>0.31</td>
<td>1.26</td>
<td>909</td>
<td>Columns remained elastic</td>
</tr>
<tr>
<td>E1</td>
<td>0.29</td>
<td>0.31</td>
<td>1.30</td>
<td>1150</td>
<td>Columns remained elastic</td>
</tr>
<tr>
<td>F1</td>
<td>0.38</td>
<td>0.31</td>
<td>1.42</td>
<td>1361</td>
<td>Columns remained elastic</td>
</tr>
<tr>
<td>G1</td>
<td>0.44</td>
<td>0.31</td>
<td>1.48</td>
<td>1537</td>
<td>Plastification of columns</td>
</tr>
<tr>
<td>H1</td>
<td>0.58</td>
<td>0.31</td>
<td>1.60</td>
<td>1670</td>
<td>Plastification of columns</td>
</tr>
<tr>
<td>A2</td>
<td>0.19</td>
<td>0.32</td>
<td>1.66</td>
<td>836</td>
<td>Columns remained elastic</td>
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<tr>
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<td>0.38</td>
<td>0.32</td>
<td>2.11</td>
<td>1295</td>
<td>Columns remained elastic</td>
</tr>
<tr>
<td>C2</td>
<td>0.58</td>
<td>0.32</td>
<td>2.55</td>
<td>1350</td>
<td>Plastification of columns</td>
</tr>
<tr>
<td>D2</td>
<td>0.66</td>
<td>0.32</td>
<td>3.16</td>
<td>1460</td>
<td>Plastification of columns</td>
</tr>
<tr>
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<td>0.74</td>
<td>0.32</td>
<td>3.24</td>
<td>1540</td>
<td>Plastification of columns</td>
</tr>
<tr>
<td>F2</td>
<td>0.95</td>
<td>0.32</td>
<td>3.50</td>
<td>1800</td>
<td>Plastification of columns</td>
</tr>
<tr>
<td>G2</td>
<td>1.10</td>
<td>0.55</td>
<td>4.90</td>
<td>1995</td>
<td>Plastification of columns</td>
</tr>
</tbody>
</table>

Owing to the great complexity of this type of dynamic test, and despite the preventive measures adopted, it was observed that some signals coming from friction or electromagnetic noise were registered by the AE sensors. For this reason, it became mandatory to carry out off-line AE data filtering after the test. A more detailed description of the filtering can be found in reference [6]. After filtering the signals received in the sensors, they were grouped in AE events using the events builder of VisualAE\textsuperscript{TM} software. The event builder groups hits into event data sets, each one consisting of the first and subsequent hits. Over these events, the MARSE energy of the first-hit recorded during each seismic simulation (energy units: $1\text{ue}=1\text{nV}\cdot\text{s}$), and the accumulated AE energy over the whole test.

During each seismic simulation, displacements, strains and accelerations were acquired simultaneously. The relative horizontal displacement $\Delta x$ between the shake table and the slab was measured by displacement transducers indicated by LVDT in Figure 2. Electrical resistance strain gages were attached to top and bottom longitudinal reinforcing bars near the corner of the slab prior to casting the concrete, as indicated in Figure 1. Strain gages were also attached at the upper and lower ends of the columns, as shown in Figure 2. Accelerometers were fixed to the shake table and to the slab as shown in Figure 2, which measured the absolute table acceleration, $\ddot{x}_g$, and the absolute response acceleration of the slab, $\ddot{\chi}'$, in the direction of shaking, respectively.

**Results**

By means a free vibration test, the vibration period $T$ and the viscous damping fraction of the structure, $\xi$, was calculated (see Table 1). While the period $T$ remains almost constant until the last simulation, the damping fraction $\xi$ increases with increasing levels of the seismic loading due to the degradation of the concrete. The maximum strains measured in the longitudinal bars of the slab, $\varepsilon_{\text{max, rebar}}$, are shown in Table 1; they remained below the yield strain $\varepsilon_{\text{y, rebar}} (=2225\times10^3)$ but got very close, as discussed below. The strain gages attached to the top and bottom sections of the steel columns indicated that they remained elastic for $PA$ less than or equal to 0.38g, but underwent plastic deformations for $PA>0.38g$.

As indicated in Table 1, the maximum strain in the longitudinal reinforcement $\varepsilon_{\text{max, rebar}} (=1995\times10^3)$ got very close to the yield strain $\varepsilon_{\text{y, rebar}} (=2225\times10^3)$. In conventional earthquake
resistant design, the structures are prepared so that \( \epsilon_{\text{max, rebar}} \leq \epsilon_{\text{y, rebar}} \) under an earthquake intensity of level I. Since in our experiments \( \epsilon_{\text{max, rebar}} \) reached 0.90 \( \epsilon_{\text{y, rebar}} \), it can be asserted that the test model was driven very close to the limit state admissible in a RC structure under level I intensity earthquakes. The fact that \( \epsilon_{\text{max, rebar}} \) remained below \( \epsilon_{\text{y, rebar}} \) does not mean that the RC slab remained undamaged.

A detailed analysis of the strains recorded by the gages attached to the longitudinal reinforcing bars of the slab revealed that the degradation of concrete associated with the cyclic loading reversals produced serious damage in the form of slippage between reinforcing steel bars and concrete at and beyond the seismic simulation D2.

From basic principles of dynamics, and assuming that the inherent damping of the structure is of the viscous type, the equilibrium equation of the system shown in Figure 2 can be written as follows:

\[
x'' + c x + F_{\text{spr}} = 0
\]

where \( c \) is the viscous damping coefficient \((=4 \pi \xi m/T)\) and \( F_{\text{spr}} \) is the restoring force opposed by the tested structure against the relative displacement \( x \). Solving for \( F_{\text{spr}} \) in equation (1) gives:

\[
F_{\text{spr}} = -(mx'' + cx)
\]

Using (2) the restoring force \( F_{\text{spr}} \) was calculated for each seismic simulation and the corresponding \( F_{\text{spr}}-x \) curves were obtained, and thus the cumulative energy \( W_p \) dissipated by the test model. The cumulative plastic strain energy dissipated by the test model up to the end of seismic simulation F1 will be referred to as \( W_{p0} \), hereafter. Up to the end of seismic simulation F1, the plastic strain energy is dissipated entirely by the concrete, since the reinforcement of the slab and the steel columns remained elastic. In seismic simulations G1 to H1 and C2 to G2, the columns underwent plastic deformations; and thus \( W_p \) is partially consumed by the concrete, \( W_{pc} \), and partially by the steel columns, \( W_{ps} \), (i.e. \( W_p = W_{pc} + W_{ps} \))—the reinforcement of the slab remained elastic in all seismic simulations, as indicated before. Obviously, up to the end of seismic simulation F1 \( W_{ps}=0 \), and at the last instant of this simulation, \( W_{pc}=W_{p0} \). Figure 3 shows in solid lines the history of \( W_p \) cumulated over the successive runs and normalized by \( W_{p0} \) (=1.41×10^7 N×mm).

![Fig. 3: History of cumulative plastic strain energy, \( W_p \), and AE energy, \( E^{AE} \), normalized by their respective values at the end of simulation F1.](image)

From the AE measurement and analyses previous described, the AE energy released in each seismic simulation was calculated. As an example, Figure 4 shows the history of AE energy rate for the seismic simulation C2, together with the history of acceleration of the shake table.
The AE energy accumulated over the successive seismic simulations until a given instant \( t \), \( E_{AE} \), was calculated and is drawn with dash lines in Figure 3, superimposed with solid lines onto the normalized plastic strain energy \( W_p/W_{po} \). Similarly to \( W_p \), in Figure 3, \( E_{AE} \) is normalized by its value at the end of the seismic simulation F1, \( E_{o}^{AE} (=14500 \text{ ue}) \) up to which the columns remained elastic. The values of \( E_{AE} \) and \( W_p \) at the end of the tests (i.e. end of seismic simulation G2) \( E_{I}^{AE} \) and \( W_{p,I} \) are \( E_{I}^{AE} =31100 \text{ ue} \) and \( W_{p,I} = 9.63 \times 10^7 \text{ N}\times\text{mm} \). Again, during the seismic simulations A1 to F1 the steel columns remained elastic and therefore \( W_{ps} = 0 \) and \( W_{pc} = W_p \). The cumulative energy dissipated by the concrete though plastic deformations at the end of stage I, \( W_{pc,o} \), is \( W_{pc,o} = W_{po} \) and all the \( E_{AE} \) is attributable to concrete cracking and friction. During the seismic simulations G1, H1 and A2 to G2, \( W_{pc} < W_p \) because a portion of \( W_p \) is consumed by the plastic deformations of the columns, but \( E_{AE} \) is related only to concrete —i.e. only to \( W_{pc} \). Figure 6 shows that during stage I there is a good correlation between \( E_{AE}/E_{o}^{AE} \) and \( W_p/W_{po} \). Accordingly, recalling that \( W_p = W_{pc} \) and \( W_{po} = W_{pc,o} \), the following approximate relationship can be established for stage I:

\[
\frac{W_{pc}}{W_{pc,o}} = \frac{E_{AE}}{E_{o}^{AE}} \tag{3}
\]

This limit to damage can be associated with a threshold value of plastic strain energy dissipated by concrete, referred to as \( W_{pc,I} \) herein. Here, it is assumed that the relation of Eq. (3) observed in stage I is likewise valid in stage II. Thus, replacing \( E_{AE} \) by \( E_{I}^{AE} \) and \( W_{pc} \) by \( W_{pc,I} \) in Eq. (1) the following relationship can be established:

\[
\frac{W_{pc,I}}{W_{pc,o}} = \frac{E_{I}^{AE}}{E_{o}^{AE}} \tag{4}
\]

Plastic strain energy is considered an appropriate parameter for characterizing low-cycle fatigue damage in RC components, and it is used in well-established RC damage indexes [7]. Here, we postulate that the level of low-cycle fatigue damage in RC slabs subjected to seismic loading at a given instant can be characterized by the index

\[
ID = \frac{W_{pc}}{W_{pc,t}} \tag{5}
\]

Taking into account, Eqs. (3) and (4), \( ID \) can also be expressed in terms of AE energy by:
In a real structure monitored with AE sensors, an estimation of $E_{AE}^I$ is needed to calculate ID at any instant within the loading process. Here we put forth that $E_{AE}^I$ is governed by the volume of concrete, $V$, involved in the cracking process, and that other factors such as the amount of reinforcing steel can be neglected. Past research ascertained that a change in specimen size does not have appreciable effects on damage patterns [8], but the volume $V$ affects the number of AE events as well as the $E_{AE}^I$ [4]. It has also been shown [9] that $E_{AE}^I$ during microcrack propagation occurs in a fractal domain comprised between a surface and the specimen volume $V$. Following this approach, the accumulated AE energy in the concrete when the limit state associated with earthquakes of level I is reached, $E_{AE}^I$, can be related to $V$ as follows [7]:

$$E_{AE}^I = \Gamma_I V^{D/3} \quad (7)$$

The parameter $\Gamma_I$ is called the critical value of fractal energy density, while the parameter $D$ is referred to as the fractal exponent, and it is comprised between $2 \leq D \leq 3$. Pending the accumulation of further experimental data, the value $D=2.3$ obtained by Carpinteri et al. [4] from compression concrete tests is adopted tentatively here. Meanwhile, $\Gamma_I$ is estimated from Eq. (7) by using the $E_{AE}^I$ accumulated at the end of the tests (i.e. seismic simulation G2), and estimating the volume of cracked concrete by:

$$V=4 \times b_e \times h \times (h_c+1.5h) \quad (8)$$

Here, $h$ and $h_c$ are the slab and column depths, respectively. The value $b_e$ is the so-called "effective width" calculated with the equation proposed by Luo and Durrani [10], which gives $b_e=617\text{mm}$. The volume of cracked concrete predicted using Eq. (8) corresponds approximately to the region where most cracks were observed by the naked eye after the seismic simulations. Substituting $V=4 \times 617 \times 125 \times (80+(1.5 \times 125))=82.52 \times 10^6 \text{ mm}^3$ and $E_{AE}^I=31100 \text{ ue}$ in Eq. (7) and solving for $\Gamma_I$ gives $\Gamma_I=0.0265\text{ue} \times \text{mm}^{-2.3}$. Using this value for $\Gamma_I$, the following expression is proposed tentatively for predicting the level of low-cycle damage in slabs subjected to seismic-type cyclic loads:

$$ID = \frac{E_{AE}^I}{0.0265V^{0.77}} \quad (9)$$

where $V$ is in $\text{mm}^3$ and $E_{AE}^I$ in $\text{ue}$.

Conclusions

1. In stage I, the plastic strain energy dissipated by concrete $W_{pc}$ and the AE energy $E_{AE}^I$ due to concrete cracking and friction, both normalized by their corresponding values at the onset of yielding of the steel columns $W_{po}$ and $E_{AE}^0$ respectively, were found to be strongly correlated.
2. Based on this correlation, a tentative formula is proposed for predicting the level of damage and the closeness to the limit state admissible in RC slabs under low-to-moderate earthquakes. This formula is based on the following assumptions: (a) the correlation between $W_{pc}$ and $E_{AE}^I$ is maintained in stage II, that is, beyond the onset of yielding of the steel columns; (b) the low-cycle fatigue damage in the slab can be characterized by $W_{pc}$; and (c) the $E_{AE}^I$ occurs in a fractal domain with exponent 2.3. With the proposed formula, the level of damage can be estimated from the AE energy recorded by sensors located in a radius of 75
cm centred in the column-slab connection and the volume of concrete in the region where damage is expected.

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