PERFORMANCE OF A BUILDING WITH DISSIPATIVE BRACING SYSTEM UNDER STRONG EARTHQUAKES

Fabrizio COMODINI¹, Alessandro FULCO², Marco MEZZI³

ABSTRACT

The paper deals with the damage assessment and the response analysis of a r/c school building built in the seventies and located in Norcia (Italy). A seismic improvement was carried out in the last decade by the insertion of buckling restrained braces acting as dissipative hysteretic devices. The construction is also equipped with a strong motion monitoring system.

The building underwent the last seismic sequence of Central Italy characterized by the main shocks of August 24, 2016 (M6.0), October 26 (M5.4 and M5.9), October 30 (M6.5), and by severe minor quakes. The post-earthquake assessment shows damage to nonstructural elements, particularly to the partitions and infill walls and partly to the external claddings. Modal identifications are done at various conditions, before and after the experienced quakes. To verify the performance of the seismic response of the structural, nonstructural and dissipative systems nonlinear dynamic analyses with the accelerograms recorded on site are performed. The results of the numerical analyses are compared with the actual response shown by the building to verify the correspondence with the actual performances of structural, nonstructural and dissipative systems resulting by the post-earthquake assessment.

Keywords: Energy dissipation; Buckling restrained braces; Strong earthquake; Actual performance

1. INTRODUCTION

In recent years the seismic retrofit of existing buildings with dissipative braces has been largely applied in Italy as well as in other countries. However, there are few cases of buildings equipped with dissipative braces which underwent earthquakes corresponding to the design intensity at the life safety limit state, that is earthquakes comparable to or higher than the maximum expected event. Moreover there is a lack of attention, both by the designers and codes, to the interaction of the nonstructural elements behavior with the response of structures including energy dissipating systems. The aspect is especially important if the consequences of the earthquake is fully assessed, including damage to nonstructural elements.

The last earthquake sequence that stroked Central Italy since August 24, 2016 to the first months of 2017 involved also the area of Norcia county, an earthquake prone area of Umbria region that underwent a number of earthquakes in the history and in the recent past: 1979 Norcia Earthquake (M5.9), 1996-97 Umbria-Marche Earthquake (M6.0). In particularly in 2016 Norcia was stroked by the two strong quakes of October 26 (M5.4 and M5.9) and, most of all, by the quake of October 30 (M6.5), in 2016. These quakes are characterized by an intensity comparable to that of the Maximum Considered Earthquake (MCE) provided by the Italian code for the site. A school building in Norcia, having a framed r/c structure, had been retrofitted few years ago with a dissipative bracing system and it was also equipped with a strong motion monitoring system by DPC (Dipartimento della Protezione Civile), that is the Italian emergency management agency, within the national network of the Seismic Observatory of Structures (OSS) For the first time in the world a monitored building equipped with an energy dissipating

¹Assist. Professor, Faculty of Engineering, University eCampus, Novedrate, Italy, fcomodini@gmail.com
²Granted Researcher, Dept. Civil and Environmental Engineering, University of Perugia, Perugia, Italy, aefulco15@gmail.com
³Professor, Dept. Civil and Environmental Engineering, University of Perugia, Perugia, Italy, marco.mezzi@unipg.it
system underwent very strong earthquakes. The study of the building response, presented in the paper, can give useful information on the performance of the earthquake resistant systems based on the energy dissipation.

2. BUILDING DESCRIPTION

2.1 Original r/c structure

The building was designed in the 60s. The seismic-resistant design was ignored but horizontal forces equal to 7% of the vertical ones were considered in design. It has a rectangular plan 59.8 m long and 12.8 m wide. It has four stories in elevation, including the basement floor. The interaction of the basement with the soil significantly influences the lateral building response. The elevation of the roof top from the ground level is equal 16.0 m. The r/c structures consists of main frames in the transverse direction and by three secondary longitudinal alignments. The main staircase includes two internal and four external columns located on different alignments. Originally the building was separated in three sections by transverse thermal joints.

The floors are made of cast in situ concrete including lightening masonry blocks and are aligned along the longitudinal direction. The column sections vary from 320×600 mm at the basement to 320×450 mm at the top level. All the beams are higher than the floor slab, but the longitudinal external beams have the height of the floor, all the other beams are higher with different heights.

The mechanical characteristics of the structural materials are derived from both the original drawings and in situ tests: the concrete has a mean compressive strength of 37.8 MPa, the steel is FeB38k type with a characteristic tensile yielding strength of 380 MPa.

![Figure 1. External views of the building](image1.png)

![Figure 2. Typical plan (second floor) of the building](image2.png)
2.2 First retrofitting works (2003)

After the Molise Earthquake of 2003, when the school building suffered minor damage, and the issuing of OPCM3274 (2003) a vulnerability assessment of the building was carried out resulting in an inadequate Capacity/Demand (C/D) ratio, especially with reference to the longitudinal behaviour. A design of seismic enhancement was performed providing for the insertion of dissipative braces within some frame grids, in both longitudinal and transverse direction, at all the story levels with the goal of improving the dissipating capacity of the structure and reducing the seismic forces on the structural elements. Moreover the design provided for the strengthening of the stairs structures through the jacketing of elements and for the removal of the thermal joints connecting the floor sections in a single diaphragm.

The project provided for K-type braces made of HEA200 beams in the transverse direction and for diagonal braces made of HEB220 beams in the longitudinal direction. The construction works were subdivided in two steps. The first step included the installation of only some non-dissipating braces at the basement level, the installation of dissipating devices should have been done in the second step.

![Figure 3. Bracing provided by the initial project](image)

![Figure 4. Works provided in the first step (only basement story)](image)

![Figure 5. Initial design: typology of steel braces](image)

2.3 Final retrofitting with dissipating bracing system

The works of the second step have been done only on 2011 after a new design, performed according to the updated code prescriptions. The works consisted on the installation of all the dissipative braces at the elevation levels, in some cases in positions different from those provided by the original design. The device location was adopted to give a symmetry of the disposition and avoid torsion effects on the structure. 56 devices have been installed at the elevation of the building: 24 at the first story and 16 at both the second and third story. At the basement level there are non-dissipative conventional braces made of steel tube 7.1 mm thick with external diameter of 168.3 mm. Buckling restrained brace (BRB) have been used as dissipating devices located along the braces. This kind of device consists of an external steel tube filled of concrete and by an internal steel core that can slide with respect to the
concrete thanks to the interposition of a low-friction material. The internal core can yield both in tension and in compression, since its buckling is prevented by the contrast with the steel-concrete external tube, and dissipates energy through the hysteretic behaviour of the steel. Ordinary market device type BRAD® 14/40-b produced by Italian FIP-Industriale have been installed. Table 1 reports the mechanical characteristics of the adopted device type. The plastic threshold of the devices has been defined so that they can activate, with their yielding, before the activation of the fragile mechanisms of the r/c structure, as it results from the calculation performed at the design stage.

Table 1. Mechanical characteristics of the installed dissipating devices.

<table>
<thead>
<tr>
<th>Yielding force (kN)</th>
<th>Elastic stiffness (kN/m)</th>
<th>Yielding displacement (mm)</th>
<th>Maximum displacement (mm)</th>
<th>Maximum force (kN)</th>
<th>Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>119</td>
<td>60000</td>
<td>1.99</td>
<td>20</td>
<td>129</td>
<td>1560</td>
</tr>
</tbody>
</table>

Dissipating brace (elevation)  
Conventional brace (basement)

Figure 6. Types of braces installed

(a)  
(b)  
(c)

Figure 7. Location of the longitudinal braces in elevation: (a) front, (b) rear, (c) central alignment

Figure 8. In plan location of the dissipating braces at the second story (piano rialzato)
3. MONITORING SYSTEM

The building is equipped by a strong motion monitoring system installed by DPC within a national program providing for the monitoring of a large amount of buildings. The monitoring system consist of strong motion accelerometers strategically located to record the building response under the input of significant earthquakes. The location and direction of the accelerometers installed in the present building are shown in the next figure. The numbers identifying the channels of the recording data-base corresponding to each of the monitored acceleration components are also reported.

![Figure 9. Location and direction of the accelerometers of the seismic monitoring system](image)

4. MAIN SHOCKS AND BUILDING DAMAGE

4.1 Shocks of 2016 Central Italy earthquake involving the school

The 2016-17 Central Italy Earthquake consisted of a seismic swarm that, from the M6.0 quake of August 24, 2016, till the M5.4 quake of January 18, 2017 and after, included hundreds of shakes of similar and even higher intensity. Some of the main shocks had an epicenter near or very near the site of the building in Norcia. With the aim of investigating the behaviour of the present building, the three seismic sequence associated to the main events of August 24, October 26 and October 30, in 2016 have been accounted for. Figure 10 shows the magnitude of the various shakes of the three main sequences.

![Figure 10. Magnitude of the shocks of the main sequence](image)

In Norcia there are two stations of the Italian national strong motion recording network (RAN - Rete Accelerometrica Nazionale), one of these, called NRC, is located not far from the school. The following Figure 11 reports the PGA values of the two horizontal (HGE and HGN) and of the vertical (HGZ) components of the shakes of the three sequences. Figure 12 reports the acceleration response spectra of the NRC station records for the main quakes of 08-24 and 10-30 together with the conventional spectrum at the site corresponding to the Collapse Prevention ultimate limit state). It can be seen that the seismic input, in terms of acceleration response spectra, is comparable, and even greater, than the Maximum Considered Earthquake (MCE) provided by the Italian code for the site.
4.2 Damage survey after the main shocks

After the first main shock of 24/08/2016 the damage was limited to some horizontal capillary cracks of the internal partitions associated to an initial loss of contact of the masonry panels from the structural elements. The contents (bookcases, furniture, desks) at the top levels did not suffer overturning or significant displacement. The building was practically operational, even if it was prudentially declared temporarily unusable for the minor damage of the partitions.

After the maximum shock of 30/10/2016 the building showed significant damage of contents and non structural elements: in particular the internal partitions in the longitudinal direction were seriously damaged and an external masonry sheathing was pushed out due to the absence of effective connections. The structural elements resulted not damaged, but some cracking in stairs connections. The post-earthquake inspection declared the building lightly damaged and temporarily unusable.

5. MODAL IDENTIFICATION OF THE BUILDING

5.1 Modal identification of the original r/c structure

After the installation of the monitoring system a modal identification was carried out in 1999. The data characterizing the building in its original framed configuration, without bracing, is reported in Table 2, where the frequencies of a numerical model simulating the building dynamic behaviour are also reported. The model took into account the non structural elements basement interaction with walls and soil through a suitable adjustment of the stiffnesses.

Table 2. Frequencies identified in the original building (1999).

<table>
<thead>
<tr>
<th>Mode</th>
<th>Shape</th>
<th>Frequency</th>
<th>Period</th>
<th>Model Freq.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Trasl. Long.</td>
<td>3.86 Hz</td>
<td>0.259 s</td>
<td>4.15 Hz</td>
</tr>
<tr>
<td>2</td>
<td>Torsion</td>
<td>4.88 Hz</td>
<td>0.205 s</td>
<td>5.46 Hz</td>
</tr>
<tr>
<td>3</td>
<td>Trasl. Transv.</td>
<td>6.06 Hz</td>
<td>0.165 s</td>
<td>5.17 Hz</td>
</tr>
</tbody>
</table>
5.2 Modal identification of the structure before the main shocks

The modal identification of the building, in its actual configuration, before the strong earthquake sequence has been performed analyzing its response to the M3.4 event of 08-04-2015 having a duration of 36 s. Only the tail of 18 s of the acceleration time histories has been considered to have significant and comparable spectral values on a wide range of periods (Figure 14).

Figure 13. Ground acceleration time-histories of the two input components (channel 1 and 2) used for identification (tail of the record 08/04/2015)

![Figure 13](image1)

Figure 14. Acceleration response spectra of the two input components (channel 1 and 2) used for identification (tail of the record 08/04/2015)

![Figure 14](image2)

Figure 15. Frequency spectra of the monitoring system records used for identification (tail of the 08/04/2015 record)

![Figure 15](image3)

It has been identified a first mode having a frequency of 4.59 Hz, that is a period of 0.21 s, and a modal shape dominated by the translation in transverse direction, as shown in the next figure. The adopted procedure was tot able to identify successive modes. The building appears to be very stiff, as expected. The modal data are compatible with those coming from the original identification, carried out before the stiffening introduced by the bracing system.

![Figure 16](image4)
5.3 Modal identification of the structure after the first main shocks (August 24, 2016)

The modal identification of the building after the first strong seismic sequence of August 24, 2016 has been performed analyzing its response to the M3.1 event of 01-09-2016 having a duration of 36 s. Only the tail of 18 s has been considered to have significant spectral values on a wide range of periods (Figure 18).

![Ground acceleration time-histories of the two input components (channel 1 and 2) used for identification (tail of the record 01/09/2016)](image1)

![Acceleration response spectra of the two input components (channel 1 and 2) used for identification (tail of the record 01/09/2016)](image2)

![Frequency spectra of the monitoring system records used for identification (tail of the record 01/09/2016)](image3)

It has been identified a first mode having a frequency of 3.9 Hz, that is a period of 0.26 s, and a modal shape dominated by the translation in longitudinal direction, as shown in the next figure. Also in this case the adopted procedure was able to identify successive modes.

The building appears to have lost some stiffness, as expected, due to the damage undergone, especially by the non structural elements, in longitudinal direction.
5.4 Modal identification of the structure after the second main shocks (October 30, 2016)

The modal identification of the building after the seismic sequence of October 30, 2016 including the M6.1 maximum exciting event has been performed analyzing its response to the M3.2 event of 02-11-2016 having a duration of 31 s. Only the tail of 12 s has been considered to have significant and comparable spectral values on a wide range of periods (Figure 22).

It has been identified a first mode having a frequency of 2.83 Hz, that is a period of 0.35 s, and a modal shape dominated by the translation in longitudinal direction, as shown in the next figure. Also in this...
case the adopted procedure was not able to identify successive modes. The building appears to have increased, as expected, the loss in stiffness in longitudinal direction due to the damage undergone, especially by the non structural elements.

6. DYNAMIC RESPONSE OF THE BUILDING UNDER THE MAIN SHOCKS

The building response to the main shocks of 24/08/2016 and 30/10/2016 have been computed analyzing the acceleration data at the floor level 2, 3 and 4 of the monitoring system. Due to the absence of instruments at the first floor (top of basement story) there is not any information on the response at this level.

The building response to the 24/08/2016 shock is shown in Figure 25. The Table 3 reports the interstory drifts. It can be observed that the structure presents a prevalent displacement in the transverse direction: the displacement at the top floor is 30 mm in transverse direction and 12 mm in the longitudinal direction. The interstory drifts are extremely low so confirming the low damage level actually observed in non structural components. The displacements at the various locations on the same floor are congruent with a diaphragm behaviour, so demonstrating the effectiveness of the floor connections.

The building response to the 30/10/2016 shock is shown in Figure 26. The Table 4 reports the interstory drifts. In this case the building presents prevalent displacements in the transverse and longitudinal direction: the displacement at the top floor is 60 mm in transverse direction and 60 mm in the longitudinal direction. The interstory drifts are now relevant with values of 15 mm (drift ratio of 4.25‰) in the transverse directions and of 14 mm (drift ratio of 4.23‰) in the longitudinal directions. these values confirm the relevant damage actually observed in non structural components. The floor displacements show some distortions between the end points and the central body of the building and are not fully compliant with the diaphragm behaviour.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Drift Lev4-Lev3 (mm)</th>
<th>Drift Lev3-Lev2 (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Y</td>
<td>-6.50</td>
<td>-4.50</td>
</tr>
<tr>
<td>X</td>
<td>-5.00</td>
<td>-4.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Drift Lev4-Lev3 (mm)</th>
<th>Drift Lev3-Lev2 (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Y</td>
<td>15.00</td>
</tr>
<tr>
<td>X</td>
<td>-10.00</td>
</tr>
</tbody>
</table>
7. RESPONSE SIMULATION ON NUMERICAL MODEL

A numerical model of the building has been built using the SAP2000NL code. Model data and results are shortly resumed for the sake of brevity. Model accounts for the nonlinearity of both the elements of the original r/c framed structure and the dissipative braces of the retrofitting system. The model also reproduces the masonry non structural elements (internal partitions and claddings) through nonlinear equivalent diagonal elements. The perimeter wall and the soil interaction at the basement level have been modeled by springs suitably calibrated in stiffness and damping. Figure 27 shows the model geometry and the deformed shape of the first two modes which periods satisfactorily approximate the identified ones. The nonlinear dynamic analyses performed using as input the main shocks of August 24 and October 30 gives displacements well approximating those resulting by the records of the monitoring system.
8. CONCLUSIONS

The analyses of the monitoring data show that the behaviour of the building is strongly influenced by the so-called non-structural elements and by the soil-structure interaction associated with the basement story and its walls. The non-structural elements, due to their stiffness, react previously than the bracing system and undergo damage due to their fragility. This did not happen in the first main shock of August 24 and the building did not suffer, practically, any damage because it behave below the capacity of partitions. In this case it can be concluded that the dissipative devices did not work or worked at low deformations. On the contrary in the main shock of October 30 the interstory drifts are significantly larger than the deformation capacity of the non-structural elements that resulted heavily damaged. The drift values are enough high to assume that the dissipative devices worked, dissipating energy and avoiding the damage of structural elements that appear actually undamaged. The devices underwent a significant number of cycles of large amplitude, even if below their maximum design displacement, and should be probably substituted. The presence of the energy dissipating system allowed the building to overcome an earthquake comparable with the MCE level at the site without damage to the structural elements. Damage of stiff non-structural element was relevant since their displacement capacity is incompatible with the displacements required by the dissipating devices. Due to the initial stiff behaviour of the building, preceding the damage of partitions, high accelerations can cause damage to contents.

9. ACKNOWLEDGMENTS

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10. REFERENCES


