SHAKE-TABLE TESTS OF A REINFORCED CONCRETE FRAME RETROFITTED WITH HYSTERETIC DAMPERS CONNECTED USING AN IMPROVED JOINT STRUCTURE

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ABSTRACT

In the last decades, the use of hysteretic dampers for the seismic upgrading of existing frame structures increased exponentially. Several types of hysteretic dampers have been used, one of them being the brace dampers that are installed in the structure like conventional diagonal members. In the case of reinforced concrete (RC) structures, the connection between the old RC beam-column joints and the damper devices has been pointed out as the key challenge of this retrofitting technique (Fardis, 2009). In fact, when the damper has the form of a conventional diagonal bar, it will be subjected to high axial loads and its influence on an existing, and probably damaged, beam-column joint is of major concern. Several solutions have been proposed for the joint structure in the past. One of them consists on using constraining members (referred also as “restraining plates” hereafter), independent from the damper brace element, that are fixed onto the concrete slab through steel anchor bolts. Steel anchors on concrete resist high shear loads, while longer anchoring height, larger number of anchors and heavier anchoring plates, are needed when high axial loads appear, making the retrofitting solution costly and non-competitive. This paper investigates an improved solution for the joint structure that is aimed at minimizing the axial forces acting in the anchor bolts. The proposed solution consists on a particular arrangement of interface materials with low friction properties and restraining plates.

To investigate experimentally the efficiency of the proposed solution, dynamic tests were conducted with the 3x3m² shaking table of the University of Granada. First, a three storey, three bay prototype RC frame structure was designed considering only gravity loads and conventional reinforcement detailing. The prototype structure represented pre-70 existing RC structures located in moderate seismic zones, such as the Mediterranean region. A high proportion of the building stock in these regions were under-designed before the appearance of seismic codes or by using rudimentary anti-seismic design criteria (Masi, 2004). Recent earthquakes have shown the poor performance of this type of structures (L’Aquila 2009, Lorca 2011), underlining the need of seismic assessment and retrofitting (Benavent-Climent et al., 2013). Second, from this prototype frame, a test structure was defined by applying scaling factors of 2/5, 1 and 1, for dimensions, stress and acceleration, respectively. Third, the test structure was subjected to several seismic simulations with the shaking table in order to induce moderate damage. Fourth, the damaged test structure was retrofitted with brace-type hysteretic dampers that were connected to the main frame using the improved joint structure. The specimen was subjected to a series of dynamic tests under increasing seismic simulations, which corresponded to very frequent, frequent, rare and very rare earthquakes.

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This paper describes the experiments and presents the overall response of the retrofitted test structure. It is shown that the proposed solution for the joint structure is effective, and proves that an existing RC frame designed only for gravity loads can withstand high seismic loads when retrofitted with brace dampers installed with the improved solution for the joint structure.

INTRODUCTION

Recent earthquakes in the Mediterranean area (L’Aquila 2009, Lorca 2011), have questioned the performance of existing structures. Most of the buildings in moderate earthquake-prone areas were constructed with reinforced-concrete framed structures, which were under-designed before the advent of modern seismic standards and using elementary seismic design principles. Thus, a high proportion of the building stock needs retrofitting with a solution that must be not only reliable and effective, but also must minimize social and economic costs. In that sense, hysteretic dampers have been proved as an economical and feasible anti-seismic device, by which this technique is now widely used. This type of dampers is usually installed as a conventional brace, in a concentric arrangement. They increase the lateral load resistance of the existing frame, as well as its stiffness and energy dissipation capacity. However, the connection between the existing frame and the brace-type damper is of major concern, due to the high concentrated forces transferred by the damper to the existing beam-column joint.

In the last few years, some solutions for connecting steel braces to reinforced concrete frames have been developed. Some of them were designed for steel structures with concrete slabs, and can be adapted for reinforced concrete structures. Ichikawa et al., (2004) proposed a solution to connect the end-plate of the brace damper to the concrete slab of steel structures, by using epoxy resin and chemical anchors. However, large axial forces were being transferred to the anchors, and large tension stresses were developing in the concrete surface. Later, the same authors proposed a solution to reduce these tension stresses acting on the concrete surface, which consisted in using shear-key plates to restrain the movement of the end-plate of the brace damper (Ichikawa et al., 2005). These restraining plates were attached to the concrete slab through anchor bolts and epoxy resin. On the other hand, the end-plate of the brace damper was only in contact with the main frame and the shear-key plates, in such a way that its movement was restrained. As a result, tension axial forces developed in the brace damper were transferred to the shear-key plates as horizontal forces (thus producing shear stresses in the anchors), as well as vertical forces, due to the friction developed in the metal-to-metal contact region between both plates. These vertical forces were non-negligible, since they forced the shear-key plate to lift-up and detach from the concrete surface. Furthermore, they induced axial tension forces in the anchor bolts and bending moments in the restraining plates, so longeranchoring height, larger number of anchors, as well as the use of thicker plates or the addition of stiffeners, were required. All these requirements can make this solution unfeasible for retrofitting purposes (since the quality of concrete may be low), and costly. Finally, it is worth mentioning that the failure of this solution is brittle (concrete failure, anchor pull out, failure of epoxy resin).

This paper presents an experimental investigation aimed at assessing the dynamic behaviour of a RC frame seismic upgraded with brace-type dampers that were connected to the beam-column joints with a simple solution that tried to minimize the effects on the existing stress condition of the joint. The experiments consist in shaking table tests conducted on a frame previously damaged and later retrofitted with brace dampers.

DESCRIPTION OF THE CONNECTION OF THE BRACE-DAMPERS WITH THE RC FRAME

The brace-dampers were connected to the main RC frame by using shear-key plates attached to the concrete frame with anchors. These shear-key plates restrained the movement of the end-plate welded at the end of the brace damper, which was not fixed to the concrete frame. The appearance of vertical-frictional forces in the contact region of the two plates was minimized, by adding a low friction
material (Teflon) on the contact surface between the end-plate of the damper and the restraining plate. In this way, tension axial forces in the anchors and bending moments in the plate were drastically reduced, and only shear forces were being transferred from the end-plate of the brace-damper to the concrete through the anchor bolts. It is worth mentioning that anchor bolts have much larger strength under shear forces than under tension axial forces, so the number and length of the anchor bolts was minimized. In addition, thinner plates without stiffeners were placed, since they needed to withstand reduced bending moments. Therefore, brittle failures of the concrete were prevented, since the brace-damper force was sustained almost entirely in shear by the steel bolts.

**EXPERIMENTAL STUDY**

*Design of the prototype building and test structure before retrofitting*

The studied prototype consisted of a three-story RC frame structure, 3x3m bay, designed only for gravity loads (as shown in Fig. 1). The specimen was representative of an existing structure without specific seismic reinforcement details and without the strength hierarchy defined in modern seismic codes for beams and columns. A 6cm concrete slab reinforced with a steel mesh and one-way joists spaced 80 cm, were placed supported by the main beams (joist-band floor system). The dead loads considered were 3.22kN/m$^2$ and 2.95kN/m$^2$ for the floors and roof, respectively. The corresponding live loads were 2kN/m$^2$ and 1kN/m$^2$. The yield strength for reinforced steel was 500MPa and the concrete compressive strength used for calculations was 25MPa.

![Prototype structure: a) plan; b) elevation](image)

A partial structural model, representative of the explained prototype structure, was separated by cutting through points of nominal zero bending moment under lateral loads. The partial model was one-story-and-a-half height and, one-bay-and-a-half width, in the direction of the main beams. It consists of two frames connected by secondary beams and the described floor system. In addition, the partial structural model was scaled by applying scale factors of $\lambda_L = 2/5$ for length, $\lambda_a = 1$ for acceleration and $\lambda_\sigma = 1$ for stress. Figure 2 shows the geometry and reinforcement details of one of the two identical main frames of the test structure. The yield strength of the reinforced steel used in the tests structure was 551MPa for the longitudinal bars and 636MPa for the stirrups. The concrete compressive strength of the tested specimen was 35MPa at 28th day. The test structure was built at the Laboratory of the University of Granada.
The tested structure presented in this paper was previously damaged through several seismic simulations. The details of these prior tests are beyond the scope of this paper and can be found in (Benavent-Climent et al., 2014). The level of damage attained by the specimen after the cited tests and, before the ones described here, can be summarized as follows: (1) plastic hinges developed at the base of the columns of the first story and the index of Park and Ang (Park and Ang, 1985) ranged between 0.19 and 0.43, corresponding to moderate (reparable) damage; (2) the average strains at member end sections $\varepsilon$ of the rest of the RC frame ranged from $0.4\varepsilon_y$ to $0.9\varepsilon_y$ in columns, and from $0.16\varepsilon_y$ to $0.23\varepsilon_y$ in beams; and (3) the width of the concrete cracks remained between 0.08 and 0.2 mm.

**Retrofitted test structure**

Brace-type dampers of the type shown in Fig. 3 were installed to retrofit the damaged test structure. Two hysteretic dampers with the form of a conventional brace were placed in the main frames of the first story, as shown in Fig. 4. The upper half-height story was stiffened with diagonal bars, to simplify the experimental set-up, so lateral displacements concentrated in the lower story. Fig. 3 describes the geometry and details of the hysteretic dampers. They were constructed by assembling several segments of wide-flange or I-shaped sections, which constituted the energy-dissipating device. These segments were attached to two U-shaped steel bars that are intended to remain elastic and function as auxiliary elements. When the brace damper is subjected to deformations in the axial direction, the web of the wide-flange or I-shape section undergoes out-of-plane flexural plastic deformations and, therefore, dissipates a considerable amount of energy.

The improved joint structure was used to connect the brace damper to the RC frame. As described before, the authors investigated the effect of adding low-friction material (Teflon) to the contact surface between the end-plate of the damper, and the restraining plates attached to the concrete. Thus, Teflon sheets were inserted in two of the four joints, while the plates of the other two joints remained with direct metal-to-metal contact. This condition is referred in Fig. 4 as “w/ Teflon”, for joint structures with Teflon sheets and, “w/o Teflon”, for those without Teflon sheets.
Instrumentation

The specimen was placed on the uniaxial MTS 3x3m2 shake-table of the University of Granada. The total mass of the specimen was 12450 kg, including additional weight placed as steel blocks on top of the reinforced-concrete slab and on top of the half-height columns of the upper floor (as shown in Fig. 4), to reproduce gravity loads. As explained before, the partial structural model representative of the complete prototype was constructed by cutting through points of nominal zero bending moment under lateral loads. To account for these boundary conditions, pin-joint connections were attached to the end of the half-height columns of the upper floor, as well as to the end of the half-length beams of the first floor. In addition, we connected the pin-joint connections located at the end of the half-length beams, to the steel blocks added on top of the half-height columns, by using a rigid steel brace. In this way, vertical displacement of pin-jointed connections was restrained, since the steel blocks were considered to be very rigid in comparison with the main frame.

Several parts of the specimen were instrumented with strain gages: (1) the continuous end-sections of RC beams and columns (as shown in Fig. 2), (2) the sides of the shear-key plates of the brace-frame connection (see Fig. 4), to obtain necessary data to calculate the shear stress acting on the anchors, and (3) the ends of the brace-damper, to measure the strain in the part of the brace which remains elastic, so the axial force acting on the brace damper was well-known. Displacement transducers (linear variable differential transformers, LVDT) were installed to measure the relative displacement in each direction of the first floor, as well as any relative displacements between the main frame and the proposed brace-frame connection (as described in Fig. 4). Furthermore, uniaxial accelerometers recorded the absolute response acceleration in each direction at the basement, the first floor and the upper steel blocks placed on the top of the half-height columns.

All the data were acquired discretely using a scan frequency of 200 Hz. Finally, the overall performance of the specimen in each simulation was recorded, as well as a detailed view of each brace-frame connection, by using five video cameras.

Seismic simulations

Dynamic tests were conducted by applying six uniaxial seismic excitations to the specimen, referred here as c50, c100, c200, c300, c350 and c400. Each signal corresponds to different scaled amplitudes of the Campano Lucano earthquake acceleration record at Calitri (50%, 100%, 200%, 300%, 350% and 400%, respectively). The record was also scaled in time by the factor $\lambda_T = \lambda_L / \lambda_a = 0.63$ to satisfy similitude requirements. The resulting peak accelerations were 0.08g, 0.16g, 0.31g, 0.47g, 0.54g and 0.62g. These PGAs were representative of different seismic hazard level (SHL) at Granada (Spain), referred hereafter as SHL-1 (corresponding to a “very frequent” earthquake), SHL-2 (“frequent” earthquake), SHL-3 (“rare” earthquake), SHL-4, SHL-5 and SHL-6 (“very rare” earthquakes). The SHL defined can be related to different mean return period: 17 years, 97 years, 500 years, 1435 years, 2032 years and 2870 years. In addition, free vibration tests were conducted before and after each simulation, to assess the evolution of the fundamental period, $T_f$, and the damping fraction, $\xi$, of the first vibration mode.
Figure 4. Retrofitted test structure. Set-up and instrumentation; a) elevation; b) plan
PRELIMINAR TEST RESULTS AND INTERPRETATION

Overall response

The overall response of the specimen is summarized in Table 1, for each seismic simulation. Table 1 shows in columns three to seven, the fundamental period $T_f$, the damping ratio $\xi$, the maximum response acceleration $\ddot{u}_{\text{max}}$, the maximum inter-story drift ID and the residual inter-story drift ID. The fundamental period, $T_f$, was determined from the free vibration tests performed after each seismic simulation, averaging the time between response peaks of several cycles. The damping ratio, $\xi$, was obtained from the frequency response curve of each seismic simulation, applying the half-power (band-width) method, as described in Clough and Penzien (1995), in the peak corresponding to the fundamental period. Results show that $T_f$ and $\xi$ remained almost constant, which indicates that the initial lateral stiffness remained unchanged and no significant damage in the main frame can be deduced from the obtained data.

<table>
<thead>
<tr>
<th>Seismic simulation</th>
<th>SHL</th>
<th>$T_f$</th>
<th>$\xi$</th>
<th>$\ddot{u}_{\text{max}}$</th>
<th>ID</th>
<th>IDr</th>
</tr>
</thead>
<tbody>
<tr>
<td>c50</td>
<td>SHL-1</td>
<td>0.20</td>
<td>1.26</td>
<td>0.21</td>
<td>0.18</td>
<td>0.00</td>
</tr>
<tr>
<td>c100</td>
<td>SHL-2</td>
<td>0.20</td>
<td>0.96</td>
<td>0.44</td>
<td>0.35</td>
<td>0.00</td>
</tr>
<tr>
<td>c200</td>
<td>SHL-3</td>
<td>0.21</td>
<td>1.04</td>
<td>0.72</td>
<td>0.66</td>
<td>0.03</td>
</tr>
<tr>
<td>c300</td>
<td>SHL-4</td>
<td>0.21</td>
<td>1.31</td>
<td>1.17</td>
<td>1.05</td>
<td>0.12</td>
</tr>
<tr>
<td>c350</td>
<td>SHL-5</td>
<td>0.21</td>
<td>1.30</td>
<td>1.21</td>
<td>1.32</td>
<td>0.16</td>
</tr>
<tr>
<td>c400</td>
<td>SHL-6</td>
<td>0.21</td>
<td>1.34</td>
<td>1.32</td>
<td>1.50</td>
<td>0.12</td>
</tr>
</tbody>
</table>

Column six of Table 1 describes the maximum inter-story drift ID attained in each simulation. For the seismic hazard levels SHL-1 and SHL-2, corresponding to “very frequent” and “frequent” earthquakes, ID remained under 0.5%. In the case of the “rare” earthquake, SHL-3, ID remained under 1.00%, and in the case of “very rare” earthquakes, SHL-4, SHL-5 and SHL-6, ID remained under 1.50%. These values were considerably lower than the ones established in FEMA 356 for enhancing rehabilitation objectives and similar return periods (1%, 2% and 4%, respectively).

To calculate the stiffness and the capacity of the test specimen, the structural model was idealized with a single mass $m=11295\text{kg}$ and one degree of freedom (the horizontal displacement). This value, $m=11295\text{kg}$, is the sum of the mass of the floor diaphragm with the added weight, plus the added weight put on top of the columns. The dynamic equilibrium is governed by:

$$m\ddot{u} + c\dot{u} + F_s = 0$$  \hspace{1cm} (1)

where $\ddot{u}$ is the absolute acceleration, $c$ is the damping coefficient, $\dot{u}$ is the relative velocity and $F_s$ is the restoring force of the structure. Since $m$ is known and $\ddot{u}$ was calculated from the measurements obtained during the experiments, the total shear force $F_{s,B}$ exerted by the inertial force, $F_{s,B} = m\ddot{u} = -(c\dot{u} + F_s)$, can be readily obtained from the tests. In the instants of zero velocity ($\dot{u} = 0$), the damping force $c\dot{u}$ is null and $F_{s,B}$ is equal to the base shear force supported by the structure, $Q_B$. Values of $F_{s,B}$ when $\dot{u} = 0$ and thus $F_{s,B} = Q_B$, were calculated for each seismic simulation. These values are plotted with symbols in Fig. 5, against the horizontal displacement of the mass $m$, $\delta_f$. Also, an approximated envelope curve of these points is plotted in Fig. 5, which represents the capacity curve of the structure.
Figure 5. Relation between base shear force and top displacement $\delta_T$, with indication of points with zero damping and their envelope

**Response of the hysteretic dampers**

Unlike the RC frame, the brace dampers experienced large plastic deformations. The axial force acting in one of the brace dampers, $N$, was measured and plotted in Fig. 6 against the lateral displacements of the floor mass, normalized by the height of the story. As it is shown, the shape of the hysteretic loops is similar to a rhomboid for large plastic deformations and presents minimum sliding phenomena. This indicates that the improved joint structure is effective in mobilizing the energy dissipation capacity of the brace damper.
Assessment of the relative displacements between the proposed joint structure and the main frame

Fig. 7 presents a comparison between the displacement measured in one of the main frames (LVDT-2 as referred in Fig. 4) and, the horizontal component of the displacement measured in the brace damper (LVDT-5). Similar results and close peak values were obtained for each case. This corresponds with the negligible sliding effects shown in the load-displacement curves of the damper, previously explained.
This paper presents the shaking table tests of a previously damaged two-fifths-scale RC frame, which was seismically upgraded with hysteretic dampers. Special attention was placed on the design and assessment of the connection between the brace-type damper and the beam-column joint, by using an improved joint structure aimed at minimizing the effects on the existing stress condition of the joint. This solution consists in placing shear-key plates which restrain the movements of the end-plate welded to the brace-damper, being the shear-key plates the ones fixed to the concrete frame with anchors. In addition, we placed a low-friction material (Teflon) in the contact region between the restraining plates and the end-plate of the damper, with the purpose of reducing perpendicular forces acting on the edge of the restraining plate. As a result, tension axial forces in the anchors and bending moments in the plate were minimized. The test specimen was subjected to six seismic simulations in a

CONCLUSIONS
sequence of increasing peak ground acceleration, characteristic of different seismic hazard level at the site of Granada (from “very frequent” to “very rare” earthquakes), which can be considered as representative of the Mediterranean area. Based on the results of the tests, the following conclusions can be drawn:

1. The fundamental period $T_1$ and the damping ratio $\xi$ of the tested specimen suffered few changes over the sequence of seismic simulations. This means that the damage on the main frame did not increase after being seismic upgraded with the dampers and subjected to new seismic simulations.

2. The maximum inter-story drifts $ID$ recorded in the seismic simulations corresponding to “very frequent” and “frequent” earthquakes were under 0.5%. In the case of the “rare” earthquake $ID \leq 1.00\%$, and in the case of “very rare” earthquakes $ID \leq 1.50\%$. These values were considerably lower than the corresponding ones established in FEMA 356 (2000) for enhanced rehabilitation objectives (respectively, 1%, 2% and 4%).

3. The improved joint structure used in this study revealed an overall satisfactory performance, remaining undamaged over the sequence of simulations. The comparison of the horizontal displacement component measured in the dampers and in the RC frame, showed basically the same values. This means that minimum relative displacements developed between the main frame and the end-plate of the dampers.

4. The force-displacement curves of the brace damper presented minimum sliding effects, confirming the good correspondence between the horizontal displacements of the RC frame and the horizontal component displacement of the end-plate of the brace damper. This fact ensures an effective transmission of seismic loads between the main frame and the damper, thus avoiding undesirable pinching effects in the response of the damper, which could reduce its energy dissipation capacity.

AKNOWLEDGEMENTS

This work received financial support from the Spanish Government under project BIA-2011-26816 and from the European Union (FEDER).

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