SUMMARY

This paper presents the results of an experimental study on the determination of damping characteristics of bare, masonry infilled, and carbon fiber reinforced polymer retrofitted infilled reinforced concrete (RC) frames. It is well known that the masonry infills are used as partitioning walls having significant effect on the damping characteristics of structures as well as contribution to the lateral stiffness and strength. The main portion of the input energy imparted to the structure during earthquakes is dissipated through hysteretic and damping energies. The equivalent damping definition is used to reflect various damping mechanisms globally. In this study, the equivalent damping ratio of carbon fiber reinforced polymer retrofitted infilled RC systems is quantified through a series of 1/3-scaled, one-bay, one-story frames. Quasi-static tests are carried out on eight specimens with two different loading patterns: one-cycled and three-cycled displacement histories and the pseudo-dynamic tests performed on eight specimens for selected acceleration record scaled at three different PGA levels with two inertia mass conditions. The results of the experimental studies are evaluated in two phases: (i) equivalent damping is determined for experimentally obtained cycles from quasi-static and pseudo-dynamic tests; and (ii) an iterative procedure is developed on the basis of the energy balance formulaion to determine the equivalent damping ratio. On the basis of the results of these evaluations, equivalent damping of levels of 5%, 12%, and 14% can be used for bare, infilled, and retrofitted infilled RC frames, respectively. Copyright © 2013 John Wiley & Sons, Ltd.

KEY WORDS: masonry infilled frames; CFRP retrofitting; damping; equivalent damping; energy methods

1. INTRODUCTION

Past earthquakes and research demonstrated that the masonry infill walls have advantages in the improvement of energy dissipation as well as increase of stiffness and strength properties of reinforced concrete (RC) structures when they are placed regularly throughout the structure and/or they do not cause shear failures of columns, [1]. Damping in RC structures arises through energy dissipation by various mechanisms such as cracking of concrete and sliding between structural and nonstructural elements. Because it is very difficult and also unpractical to directly calculate the damping, experimental research is essential in order to determine a range of such energy dissipation characteristic. A review is given in the following:

Buttmann [2] conducted an experimental study on the specimens with the dimensions of 100 × 200 × 11.5 cm and 24 cm. The horizontal sinusoidal excitation applied to the specimens was
generated by a dynamic oscillator with a maximum power of 20 kN. The experimental study yielded critical damping ratios of 11% for shear walls and 24% for masonry walls. Farrar and Baker [3] performed an experimental study on 1/3-scaled low aspect ratio RC shear walls. It was concluded that within elastic range of the testing, damping ratio was found to be 2%, and when the damages increased and re-bars yielded, this value increased up to 22%. Fardis and Panagiotakos [4] evaluated pseudo-dynamic tests (PsD) test results that were conducted in ELSA Laboratory by Negro and Verzeletti [5]. It was concluded that the infills resulted damping after the first cracks observed. It was stated that the hysteretic energy dissipation occurred through the masonry infills. Also, the response spectra of an elastic single degree of freedom system infilled frame, despite infill’s apparent stiffening effect on the system, a reduction in the spectral displacement and forces were obtained mainly through high level damping. Hashemi and Mosalam [6, 7] conducted shake table tests on 3/4-scaled, three dimensional infilled RC frames. The tests resulted nearly four times higher structural stiffness, shortened natural period nearly 50%, increased damping coefficient from about 4% to 12%, and also increased the energy dissipation capacity of the system.

Costa et al. [8] performed in situ tests on masonry walls of abandoned traditional houses. Five specimens were tested aiming at characterizing the out-of-plane behavior of stone masonry walls and strengthening solutions recommended for post-earthquake interventions. Even for small drift values as 0.1%, the hysteresis is significant leading to an equivalent hysteretic damping value of 12% mainly explained by permanent deformations developed at the joints already for small displacement levels. The evolution of hysteretic damping is almost linear with the evolution of drift up to the formation of a complete diagonal crack to the foundation, which occurred for the drift cycle of 0.75%. This led to significant residual deformations along the wall and an equivalent hysteretic damping level of 26%. The final part of the test (drift of 1.0 and 1.25%) shows a constant hysteretic damping level close to 25% as a result of the severe damage observed and permanent deformations of the wall.

Sofronie [9] indicated that the masonry walls act as active dampers when strengthened with FRP by increasing frictional forces between wall elements resulting in higher damping. Santa-Maria et al. [10] conducted experimental studies on masonry walls under the effect of monotonic and cyclic loadings. The masonry specimens were retrofitted by horizontal, vertical, and diagonally braced FRP. Especially horizontally retrofitted specimen displayed a great increase in damping. Elgawady et al. [11] investigated the behaviors of seven specimens of 1/2-scaled FRP retrofitted masonry walls under cyclic displacement reversals. FRP caused a great increase in lateral stiffness, strength, and energy dissipation capacity. The damping values were also determined for each specimen at varying drift levels. FRP confinement provided increase in damping ratios. Some of the specimens were retrofitted after being tested, and these specimens produced higher values of structural damping.

Although various experimental studies have been conducted on the determination of damping characteristics of masonry walls and masonry infilled RC frames; for the quantification of equivalent damping, there is an apparent gap in the literature on carbon FRP (CFRP)-retrofitted infilled frames. There is a necessity about the damping characteristic, which is particularly important for the accurate estimation of seismic forces, of CFRP-retrofitted infilled RC frames for the development of realistic structural models.

In this paper, a new concept based on the energy balance is proposed to quantify the damping characteristic of the specimens. The scope of this experimental study is limited to 16 specimens of 1/3-scaled one-bay, one-story RC frames subjected to quasi-static (QS) and PsD tests. The test results are used in the quantification of equivalent damping ratios for bare, masonry infilled, and CFRP-retrofitted masonry infilled RC frames.

2. EXPERIMENTAL STUDY

The experimental study is conducted on sixteen 1/3-scaled one-bay, one-story RC frame, which is taken out from a three-span and five-story RC building. The specimens were built according to the
old construction practice, which had several variances with the current seismic design code of Turkey [12]. Dimensions and reinforcement details of the specimen are illustrated in Figure 1(a). Longitudinal reinforcement ratio in columns and beam is 1%, whereas transverse reinforcement ratio is around 0.4%. No confinement reinforcement in and around beam-column connections are used. Compression strength of concrete is obtained 19 MPa from the standard cylinder tests, which corresponds to the strength at the day of testing. Yield strength of reinforcements is 420 and 500 MPa for eight and 6 mm diameters, respectively.

The clay type brick used in the infill wall has a dimension of 87 × 84 × 56 mm, Figure 1(b). Both sides of infill wall were plastered having a thickness of 10 mm. Compression tests of the masonry wallets with the dimensions of 350 × 350 × 56 mm resulted compression strengths of 5.0 and 4.1 MPa in the two perpendicular directions. The diagonal tension (shear) test defined in ASTM E519–02 [13] was applied, and the shear strength of 0.95 MPa is reached [14]. As per the technical data provided by the manufacturer, the unit weight of the CFRP is 300 g/m², the fiber density is 1.79 g/cm³, and the modulus of elasticity of CFRP is 230 GPa, where the tensile strength and ultimate elongation capacities are 3900 MPa and 1.5%, respectively. Special anchorages were provided along the CFRP sheets at approximately quarter distances of the diagonal with the length of the 24 cm, which will be enough to cover the CFRP strips applied on both sides of the infill. The CFRP sheet was rolled with enough amount of epoxy and was installed in the infill through the bricks, Figure 1(c).

The four groups of specimens used in the study are shown schematically in Figure 2. Two alternative CFRP retrofitting scheme are applied to the infilled frames.

A servo-controlled 280-kN capacity hydraulic jack is used for the lateral loading. The specimens are fixed to the rigid steel beam of the test frame, Figure 3. No axial forces were applied to the columns to attain fairly simple testing setup, particularly for the PsD tests. To prevent the potential out of plane
deformations, four restrainers were used in the testing setup. There is small distance between the restrainers and the beam, and grease was applied to the surface of the restrainers.

In QS tests, eight specimens were tested in two groups using two different drift-based reversed cyclic loading patterns, namely, one-cycled and three-cycled displacement history cases, Figure 4.

The acceleration record used in the PsD tests were derived from the BOL090 component of October 12, 1999 Düzce Earthquake, which has PGA = 0.822 g. The part between 8 to 18 s of the record was modified to comply the acceleration design spectrum defined in Turkish Earthquake Code [12] for seismic zone 1 and firm type soils (Z2), Figure 5. The Oasis Sigraph software (Oasys Ltd., Newcastle-Upon-Tyne, UK) [15].

Figure 2. The specimens.

Figure 3. The testing set-up.

was used for this process. The target acceleration record of PGA = 0.4 g is called as the design earthquake. The other two records, which are PGA = 0.2 g and PGA = 0.6 g, were derived from the design earthquake by linear scaling.

The power spectra of the original and modified records are compared in Figure 6. The imparted energies, which are the area enclosed by the power spectra, are 0.109 and 0.156 units for the original and modified records, respectively, Kuwamura et al. [16]. So, the modified acceleration record used in PsD tests imparted more energy to the specimen with respect to the original one.

Mass intensity was one of the main parameters of PsD tests. The values practiced are $M_1 = 0.0085 \text{ kN.s}^2/\text{mm}$ and $M_2 = 0.0221 \text{ kN.s}^2/\text{mm}$. They are representing the lower and upper story masses, which are scaled down from the prototype structure. The experimental details can be found in Ozkaynak’s PhD dissertation [17] and Ozkaynak et al. [18].

A high sensitive optical displacement transducer is used in the application of target displacement to the specimens in PsD tests. Also, high sensitive load cell was instrumented for all the experiments. The restoring force corresponding to a particular displacement increment is evaluated within a number of buffering force.

The PsD tests were conducted by slow mode. Application of the target displacement, measuring of restoring force, measuring of displacements and deformations throughout the specimen, and solution of dynamic equilibrium equation for the next step elapsed 10–15 s for each point of the acceleration record. No data filtering was used.
The typical lateral load versus top displacement curves obtained in QS tests for bare, infilled, and retrofitted cases are shown in Figure 7. The increase of strength and stiffness from bare to infilled and retrofitted specimens was clearly seen from the curves. Also, the level of damage within the inelastic range is significant indicating progression of damping through energy dissipation.

The behavior of the test specimens for the PsD tests is given in Figure 8 for the parameters of $M_2 = 0.0221 \text{kN} \cdot \text{s}^2/\text{mm}$ and PGA = 0.4 g.

In case of the bare frame, the sources of energy dissipation mechanisms could be clearly observed from the progression of damages through the increments of drifts. Flexural types of cracks were observed at both ends of the columns. The crack distribution photos of bare frame at the initial stage and end of the test are illustrated in Figure 9(a) and (b), respectively.

In case of the infilled frame, flexural cracks were first formed on column ends, then slight separation of infill wall from RC members were observed, Figure 10(a). It was followed by infill cracking in both diagonal directions. Finally, corner crushing in infill wall and corner separations were observed, Figure 10(b).

In case of the retrofitted specimens, the first observed damages were the flexural cracks on columns. The succeeding damages observed in further steps of the tests were diagonal cracks occurred on infill walls. Toward end of the tests, crashing of plaster and infill damages were localized around the near vicinity of CFRP anchorages, Figure 11.

3. EVALUATION OF QUASI-STATIC AND PSEUDO-DYNAMIC TEST RESULTS IN TERMS OF DISSIPATED ENERGY IN SUCCESSIVE CYCLES

Typical behavior of a structure or structural member having energy dissipating capability, subjected to cyclic loading is schematically illustrated in Figure 12. The energy dissipated in the structure is given
by the area $E_H$ enclosed in the hysteresis loop. *Equivalent damping ratio* is defined according to Chopra [19] in terms of dissipated energy ($E_H$) and strain energy ($E_{SD}$), hereafter referred to as energy ratio method. The equivalent damping ratio $\zeta_{eq}$ is given by Eq. 1.
Figure 10. Typical damages of infilled specimens in one-cycle quasi-static tests.

Figure 11. Typical damages of the retrofitted specimen in one-cycle quasi-static tests.
3.1. Evaluation of quasi-static test results

The attained equivalent damping ratios with increasing drifts in QS tests are discussed here. At the beginning of one-cycle static tests, the obtained equivalent damping ratio for bare frame was around 5%, whereas the resulting damping ratios of infilled and retrofitted specimens were close to 10–12%, indicating that the effect of retrofitting was not apparent, Figure 13.

\[
\zeta_{eq} = \left(\frac{1}{4\pi}\right)\left(\frac{E_H}{E_{S0}}\right)
\]  

(1)

Figure 13. Equivalent damping variations of quasi-static tests results.
For the three-cycle QS tests, the equivalent damping ratio was calculated on the basis of the average of the results of three cycles. At the initial stage, the obtained equivalent damping ratio for bare frame was around 5%, whereas the resulting damping ratios of infilled and retrofitted specimens were close to 9–10%, and again, indicating that the effect of retrofitting was not apparent.

The ratio of the damping values calculated for the one-cycle tests to the one obtained from the first cycle of the three-cycle test is given in Figure 14 in terms of drift ratios. There are two distinct regions in the graphics. Here, up to 2.0% drift represents relatively undamaged stage, whereas beyond 2.0% is the damaged stage. Specifically for the cross-braced frame, the average ratio, $r_{av}$, was around unity where it was determined as 1.31 for the second region. This can be evaluated as in the damaged stage one-cycle test results with 30% greater value than that of three-cycle test.

Depending on the QS test results, one can be concluded that prior to inelastic range, damping ratios for bare, infilled and retrofitted frames can be assigned to 5%, 9–12%, and 9–12%, respectively. However, within the inelastic range, which is initiated beyond 2% drift, the equivalent damping ratios for bare, infilled, and retrofitted frames can be taken in the range of 8–10%, 10–13%, and 13–15%, respectively.

3.2. Evaluation of pseudo-dynamic test results

The results of PsD tests were used for the determination of damping properties of bare, infilled, and CFRP retrofitted infilled specimens. Complete closed cyclic loops were extracted from the load–displacement relations of PsD test results. By using the energy ratio method, average equivalent damping ratios for $M_1$ and $M_2$, and also for different PGA intensities were determined.

In Eq. 1, the term involving the area of hysteresis loop is directly affected by the fluctuation of lateral load within a small load range coupled with high number of loops during the experiment. This may cause sudden increase or decrease of the area of the loops compared with other successive loops. Consequently, the obtained damping values diverge in a wide range of scatterness. Histogram analysis of the scattered data showed the distribution of damping ratios in terms of recurrence number. This statistical analysis method led us to define the average equivalent damping ratio using geometric-mean in lieu of arithmetic mean.

![Figure 14](https://example.com/image1.png)

**Figure 14.** Equivalent damping ratios for the one-cycle to first loop of three-cycle quasi-static test varying with drift ratios.
The drift versus equivalent damping ratio graphics were plotted in the drift ranges of ±1.5% in order to make good comparison with high mass and intensity cases. Especially, for M1 case of the infilled specimen, the points gravitate within ±0.1% drift region. So, the values are seen as if they were concentrated in the vicinity of origin.

3.2.1. Bare frame. The analyses results for bare frame are shown in Figure 15. It is seen that for increasing PGA levels, the average equivalent damping ratio shown with a broken line is also increasing. The equivalent damping ratio scatters between 5% to 20% with an average value of 8% to 13% for two mass conditions, $M_1$ and $M_2$.

3.2.2. Infilled frame. Equivalent damping ratio distributions and the average values are illustrated in Figure 16. For various PGA intensities, the observed equivalent damping ratios are between 5% to 25%, whereas the average damping is in the vicinity of 12% for the two mass cases, $M_1$ and $M_2$.

<table>
<thead>
<tr>
<th>PGA</th>
<th>$M_1$ Inertia Mass</th>
<th>$M_2$ Inertia Mass</th>
</tr>
</thead>
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<tr>
<td>0.2g</td>
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<td><img src="image2" alt="plot" /></td>
</tr>
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<td>0.4g</td>
<td><img src="image3" alt="plot" /></td>
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</tr>
<tr>
<td>0.6g</td>
<td><img src="image5" alt="plot" /></td>
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</table>

Figure 15. Equivalent damping ratio variation for bare frame.
3.2.3. Cross-braced frame. Equivalent damping ratio variations and their average values are plotted in Figure 17. For various PGA intensities, the observed equivalent damping ratio was between 5% to 25%, whereas the mean damping values range 10–13%, for two mass cases, $M_1$ and $M_2$.

3.2.4. Diamond cross-braced frame. Equivalent damping ratio variations and their mean values are plotted in Figure 18. For various PGA intensities, the observed equivalent damping ratio was between 5% to 25%, whereas the mean damping value is in the vicinity of 13%, for two mass cases, $M_1$ and $M_2$.

Displacement responses obtained from low mass ($M_1$) and low intensity conditions are somewhat less than that of high mass ($M_2$) and high intensity conditions. For this reason, the obtained equivalent damping ratios are gravitated around the low drift ratios as seen in Figures 16–18.

From the results of tests conducted with two different methods, it can be concluded that the equivalent damping ratios of the retrofitted infilled frames were greater than those of bare and infilled frames. In all tests, the diamond cross-braced frame had higher damping ratios than the

<table>
<thead>
<tr>
<th>PGA</th>
<th>$M_1$ Inertia Mass</th>
<th>$M_2$ Inertia Mass</th>
</tr>
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<td>0.2g</td>
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<td>0.4g</td>
<td><img src="image3" alt="Graph" /></td>
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</tr>
<tr>
<td>0.6g</td>
<td><img src="image5" alt="Graph" /></td>
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</tr>
</tbody>
</table>

Figure 16. Equivalent damping variation for infilled frame.
cross-braced frames. Furthermore, an average value of equivalent damping ratio for the retrofitted infilled frames can be proposed as 13%.

The equivalent damping ratios that were obtained from PsD and QS tests are compared in Table I and illustrated in the graphical form in Figure 19.

4. PROPOSED APPROACH TO DETERMINE THE EQUIVALENT DAMPING RATIO BY USING ENERGY BALANCE METHOD

Early studies on energy-based evaluation methods initiated with the work of Zahrah [20] who performed numerical analyses to show how the energy terms in a simple structure is extracted from the displacement response of the structure under earthquake excitation. The energy considerations on structural systems and evaluation of structural performance based on energy concepts have been
the scope of several studies including those works that were conducted by Shen and Akbas [21], Chou
and Uang [22], and Nurtug and Sucuoglu [23].

Negro and Verzeletti [5] investigated a four story infilled RC structure by subjecting it to PsD tests
on the basis of which the structural performance evaluation was accomplished using energy

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>Quasi-static Test</th>
<th>Pseudo-dynamic Test</th>
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<tbody>
<tr>
<td></td>
<td>One cycle</td>
<td>Three cycle</td>
</tr>
<tr>
<td>Bare frame</td>
<td>11</td>
<td>8</td>
</tr>
<tr>
<td>Infilled frame</td>
<td>13</td>
<td>13</td>
</tr>
<tr>
<td>Cross-braced frame</td>
<td>13</td>
<td>14</td>
</tr>
<tr>
<td>Diamond cross-braced frame</td>
<td>14</td>
<td>13</td>
</tr>
</tbody>
</table>

Table I. Equivalent damping ratios (%) obtained from quasi-static and pseudo-dynamic tests.

Figure 18. Equivalent damping ratio variation for diamond cross-braced frame.
considerations. Mosalam et al. [24] performed PsD tests on a two-bay, two-story steel structure. The test results were evaluated in terms of energy concepts. It was concluded that the infill walls increased the structural damping from 2% to 12%.

The energy imparted into the structure during earthquake is divided into several mechanisms, namely, elastic strain, kinetic, hysteretic, and damping energies. The equation of motion of a single degree of freedom system represents the excitation of ground and the response of structure in terms of forces in time domain, Eq. 2. Integration of the equation of motion with respect to the relative displacement of the system, \( u(t) \), yields the following equation [25];

\[
\int m \dddot{u}(t) \, du + \int c \ddot{u}(t) \, du + \int f_s \, du = -\int m \dddot{u}_g(t) \, du
\]  

(2)

where the terms are named as kinetic energy \( (E_K) \), damping energy \( (E_D) \), and absorbed energy that is composed of elastic strain energy \( (E_S) \), hysteretic \( (E_H) \) energy and input energy \( (E_I) \), respectively. Thus, Eq. 2 can be re-expressed in the following form:

\[
E_K + E_D + E_S + E_H = E_I
\]  

(3)

Care should be paid to the third term on the left hand side of Eq. 2, which includes the energy of elastic and plastic strains. The elastic strain energy \( (E_s) \) is formulated with respect to the initial stiffness \( (k) \) of the structure, as follows:

\[
E_S = \frac{(f_s(t))^2}{2k}
\]  

(4)
The plastic strain energy, namely, hysteretic energy \( (E_H) \) is calculated as cumulative area enclosed by the load–displacement curves.

The displacement history is directly obtained as the response of test specimen due to the applied input acceleration in PsD tests. The velocity and acceleration response histories are then derived from the displacement history by using seven-point stencil central-differences.

![Figure 21. Energy distributions and obtained equivalent damping ratios for \( M_1 \) case.](image-url)
For an assumed equivalent damping ratio, the energy terms defined in Eq. 3 are calculated and the global energy balance is checked. This iterative process is carried out till the energy balance is satisfied with a predefined tolerance. Consequently, $E_K$, $E_D$, $E_S$, and $E_H$ energy terms are discretely determined. A general flowchart of the algorithm is illustrated in Figure 20.

<table>
<thead>
<tr>
<th>PGA=0.2g</th>
<th>PGA=0.4g</th>
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</thead>
<tbody>
<tr>
<td>Bare Frame</td>
<td>Bare Frame</td>
</tr>
<tr>
<td>$\zeta = 10%$</td>
<td>$\zeta = 12%$</td>
</tr>
<tr>
<td>Infilled Frame</td>
<td>Infilled Frame</td>
</tr>
<tr>
<td>$\zeta = 9%$</td>
<td>$\zeta = 11%$</td>
</tr>
<tr>
<td>Cross-braced Frame</td>
<td>Cross-braced Frame</td>
</tr>
<tr>
<td>$\zeta = 11%$</td>
<td>$\zeta = 12%$</td>
</tr>
<tr>
<td>Diamond cross-braced Frame</td>
<td>Diamond cross-braced Frame</td>
</tr>
<tr>
<td>$\zeta = 10%$</td>
<td>$\zeta = 12%$</td>
</tr>
</tbody>
</table>

Figure 22. Energy distributions and obtained equivalent damping ratios for $M_2$ case.
The well-known relation given in Eq. 5 is used to calculate the damping coefficient $c$ as a basis for this iterative scheme.

$$c = 2\zeta m\omega$$  

The proposed energy balance methodology was carried out to PsD tests for $M_1$ and $M_2$ cases and acceleration intensities of 0.2, 0.4, and 0.6 g. The energy distribution graphics and the obtained equivalent damping ratios are presented in Figures 21 and 22 for $M_1$ and $M_2$ cases, respectively.

The average equivalent damping ratios that are determined by using both energy ratio and energy balance methods are summarized in Table II. The results given for PsD tests belongs to the design earthquake (PGA = 0.4 g). In the energy ratio method, equivalent damping ratio of the specimens were originated as the geometric mean of damping ratio obtained for each loop as indicated in Figures 15–18.

However, the energy balance method takes the frequency content and the entire duration of the record into account as per Figures 20 and 21. Hence, in the evaluation of the PsD test results, the developed energy balance method is more reliable. For the design earthquake (PGA = 0.4 g), the energy ratio method yields relatively high values compared with the energy balance method.

According to all of the performed QS and PsD test results, the minimum value for the equivalent damping ratio of filled and CFRP-retrofitted filled frames would be 10% to 13%.

As it is seen from Table II, the average equivalent damping ratios for the retrofitted frames by using QS and PsD tests results are scattered in the range of 10% to 14%. This result is affiliated with the damage conditions observed on the specimens.

### Table II. Equivalent damping ratios (%) obtained from quasi-static and pseudo-dynamic tests.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Energy ratio method</th>
<th>Energy balance method</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Quasi-static tests</td>
<td>Pseudo-dynamic tests</td>
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<td>13</td>
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5. CONCLUSIONS

The results of the experimental study have been evaluated for bare, filled, and retrofitted filled RC frames in order to quantify the equivalent damping ratios in terms of experimental parameters that were used in QS tests such as one-cycle and three-cycle loading patterns and in PsD tests such as inertia masses and various PGA levels. In the analysis, the energy ratio method and energy balance method have been used to determine the equivalent damping ratios for each parameter and also their trends were discussed.

The equivalent damping ratios obtained from the energy balance method are smaller than those derived from energy ratio method.

The following conclusions are drawn from the QS Tests:

1. The equivalent damping for bare frame varies between 8% to 11% depending on the damage level.
2. The equivalent damping ratio derived for filled frame is 13%. Therefore, the effect of infill walls on the damping can be clearly seen from the results.
3. The cross-braced and diamond cross-braced frames has the biggest damping ratio, which is up to 14%.
The following conclusions are drawn for the PsD Tests:

1. In the comparison made for the design earthquake (PGA = 0.4 g), equivalent damping ratios of all the specimens are in the range of 10–15%. The value for bare frame accounts for the large hysteretic cycles prior to the collapse.

2. The equivalent damping ratio for infilled frame is obtained in the range of 9% to 11%. This result is consistent with the existing literature.

3. The average damping characteristics of the retrofitted frames are scattered in the range of 10% to 14% for the altered mass and PGA intensities. Although bare frame failed during the design earthquake (PGA = 0.4 g), both types of the retrofitted frames could withstand at the end of the earthquake of PGA = 0.6 g. So, for the retrofitted frames equivalent damping ratio turns out to be sustainable through the seismic action.

On the basis of the overall evaluation, equivalent damping ratio of 10–13% is recommended as a sustainable range for the CFRP retrofitted RC frames.

ACKNOWLEDGEMENTS

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