Damped Interconnection-Based Mitigation of Seismic Pounding between Adjacent R/C Buildings

S. Sorace and G. Terenzi

***Abstract*—A representative case study of potential earthquake-induced pounding between adjacent R/C frame buildings with insufficient separation gaps is examined in this paper. The height of the two examined buildings is the same, but their response is affected by considerable torsional pounding effects. An upgraded version of the traditional linear viscoelastic model for the numerical time-history analysis of the dynamic impact problem is proposed and implemented in the finite element model of the buildings. The results of the assessment enquiries carried out in current conditions, and a damped interconnection-based mitigation solution based on the incorporation of pressurized fluid-viscous dissipaters across the inadequate separation gaps, are presented. Evaluations of the benefits provided by the retrofit intervention, and some of its technical installation details, are finally offered.**

***Index Terms*—Damped interconnection, impact models, mitigation strategies, seismic pounding.**

I. Introduction

Earthquake-induced pounding between closely spaced buildings is one of the highest sources of seismic vulnerability, as it can cause severe damage to non-structural and structural members, and even contribute to structural collapse [1], [2]. Pounding impacts derive from the out-of-phase vibrational response of the colliding structures induced by their different dynamic characteristics, when their separation joints at rest are not wide enough to accommodate the maximum relative displacements. Insufficient separation between adjacent buildings is typical of old city centers, where masonry buildings are normally in full contact along the height, as well as of modern urban blocks, where buildings were designed without any seismic provisions or by referring to earlier editions of the current seismic Standards, and thus generally with inadequate separation.

Prevention of pounding in new structures is easily attained by adopting properly sized gaps, although with some limitations for tall buildings, deriving from the loss of useful floor space and the technical set up of proportionally sized expansion joints.

On the other hand, mitigation of pounding hazards in existing buildings is usually a very demanding issue. A traditional mitigation strategy is represented by a general stiffening of the individual potentially colliding buildings, aimed at reducing their absolute and relative displacements. However, whatever the stiffening method chosen, this strategy represents the most expensive and invasive approach to the issue, as it involves a complete seismic retrofit of the structural systems. Alternative strategies are based on local interventions in the predictable contact areas. The first solution consists in rigidly linking the adjacent structures by means of coupling beams [3] or shock transmitters [4], the latter being preferred especially for buildings with a wide plan, as they provide stiff connections during earthquakes, while avoiding the rising of significant forces related to the thermal elongation effects. While this rigid interconnection-based approach, which is derived from similar mitigation solutions adopted for bridge structures, allows preventing collisions, it can generate considerable increases in seismic story shears, and thus in the stress states of the structural building members. Also, the dynamic response of the joined structures appreciably differs from the response in the original separated configuration, and sometimes it can cause unfavorable effects as compared even to the most demanding out-of-phase pounding response conditions. A careful numerical evaluation of the consequences of the interconnection interventions is required to identify the best linking layout.

The second solution is represented by the introduction of "sacrificial" elements, also named "crash box interfaces" [4], e.g. crushable parapets and carters, or strong collision walls acting as bumpers [5], all capable of protecting the contact-prone structural members. In case of pounding, these elements are subject to remarkable damage and require post-earthquake repair (collision walls) or substitution (crushable devices), at rather high costs and with a interruption in the use of at least some portions of the buildings. This implicitly fails to meet the basic requirements imposed by the last generation of international seismic Standards for what concerns the Operational and Immediate Occupancy performance levels, in spite of the fact that structural and non-structural members may formally meet them, thanks to the protective action of the "sacrificial" elements.

The third mitigation strategy refers to the concept of structural interconnection too, like for the rigid linking approach recalled above, but it is based on the installation of energy dissipating devices across the separation gaps, with the aim of substantially reducing the severity of collisions rather than preventing them [6]. The advantages are represented by the remarkably lower interaction forces transmitted as compared to the rigid link configurations, thanks to the dissipative action of the dampers. Furthermore,

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a lower dynamic coupling between the connected buildings is determined. On the other hand, like for any type of interconnection-based intervention, an agreement between the owners of the buildings is legally required, in consideration of the mutual alterations made (which can be rated, in terms of town planning rules, as a transformation of two originally separate units into a new whole unit). From a numerical modeling viewpoint, the incorporation of dampers accentuates the non-linear characteristics of the contact problem, which makes it solvable only by means of a time-history dynamic analysis approach. The critical design aspects consist in determining the optimal damping properties of the devices, as well as their best layout in the collision zones. The remaining design variables are the same as for a rigid linking problem, i.e. related to the dimensions of the existing separation gaps, as well as to the geometric and material characteristics of the adjacent buildings (with equal height and aligned floors; with different height and aligned floors; with equal or different height, but not aligned floors; with any of these configurations, and significantly differing floor masses; or else, multiple buildings in a row; corner buildings in potential collision with buildings in orthogonal rows; buildings with an asymmetric structure, causing torsional pounding, etc).

This paper offers a synthesis of a research study dedicated to the analysis of pounding between reinforced concrete (R/C) frame buildings and its mitigation by a damped interconnection strategy based on the incorporation of pressurized fluid viscous (FV) dampers as protective devices. The analytical contact-force models proposed in the literature are briefly recalled, and a modified version of the classical linear viscoelastic model is introduced. This model is applied to the analysis of a representative case study, where the two potentially colliding R/C structures have the same height, but whose response is affected by considerable torsional pounding effects. Based on the results of the pounding assessment analysis, a damped interconnection mitigation solution is suggested by illustrating the benefits induced by the retrofit intervention and some of its technical installation details.

II. Contact Models for Numerical Analysis of Pounding

Two conceptual models have been developed to simulate structural pounding, i.e. the stereo-mechanical approach and the contact element approach, respectively. The former refers to the traditional theory of impact for particles [7], and is based on the principles of conservation of energy and momentum. Impact is evaluated by the coefficient of restitution r, which accounts for the energy dissipation related to the plastic effects occurring during the collision, defined as follows:

$$r = \frac{v_1' - v_2'}{v_1 - v_2} \tag{1}$$

where v_1, v_2 are the approaching velocities, and v_1', v_2' are the post-impact (rebound) velocities. The stereo-mechanical theory is not appropriate for developing a time-history analysis of multi-degree-of freedom structural systems, as it does not simulate the structural response during contact, by assuming a negligible duration of it. In fact, this is an essential phase for the computation of pounding forces, as well as for the influence exerted on the global response of the colliding structures, especially in the frequent case of multiple simultaneous contacts. Moreover, the stereo-mechanical contact model cannot be directly implemented in commercial finite element calculus programs. Therefore, application of this approach is generally confined to research studies focused on the impact of simple bodies, which can be schematized as single-degree-of-freedom systems, and analyzed by specifically developed software.

The contact element approach offers a straightforward idealization of the pounding problem, as it corresponds to the intuitive interpretation of the phenomenon. Impact is simulated by a contact element that is activated when the separation gap between the structures shrinks, which allows solving the problem within the framework of an ordinary response analysis. The contact element is obtained by combining in parallel a spring and a viscous damper. The stiffness of the spring is typically assumed to be equal to the axial stiffness of the contacting floor diaphragms (or the stiffness of specific floor portions or members, in case of localized impacts).

The spring is generally assumed to be linear elastic or non-linear elastic. In the latter case, reference is commonly made [5], [8] to the Hertz model, which expresses the contact force as a n-power law of the relative displacement between the colliding members, with the n exponent fixed at 3/2. Although the non-linear model corresponds to the physical expectation that the contact area will increase as the contact force grows, extensive computational studies [8] have shown that the displacement response of the colliding systems is scarcely influenced by n, and thus that similar numerical results are obtained as compared to the linear model too.

The damper element is associated to the elastic spring, either of linear or non-linear type, in order to account for the energy dissipation occurring during impact. If an elastic spring and a linear viscous dashpot are jointly assumed, the model coincides with the classical linear viscoelastic Kelvin-Voight rheological scheme (upper image in Fig. 1, where m_1, m_2 are the masses of the colliding structures, and *sep-gap* is the separation distance at rest). The damping coefficient of the linear dissipater, c_1, can be related to the coefficient of restitution r by equating the energy losses during impact [9], obtaining the following expressions:

$$c_1 = 2\xi\sqrt{k_1 \frac{m_1 m_2}{m_1 + m_2}} \tag{2}$$

$$\xi = -\frac{\ln r}{\sqrt{\pi^2 + (\ln r)^2}} \tag{3}$$

where k_1 is the spring stiffness, ξ is the impact damping ratio, and $\ln(\cdot)$ is the natural logarithm function. This formulation acceptably agrees with the results of tests on simple impacting systems, except for the fact that the dashpot element is active not only in the approaching phase, but also in the separation time interval. As a consequence, it counters the relative bounce motion pulling the structures together in the early separation motion, which is the opposite of the real physical pounding.

In order to bypass this incongruence, a gap element can be incorporated in series with the damper [5], [10], so that the latter is activated only at the approaching stage. This varied scheme, also called Impact Kelvin model, solves the drawback of the original Kelvin-Voight model in terms of numerical response, but not from a conceptual viewpoint.

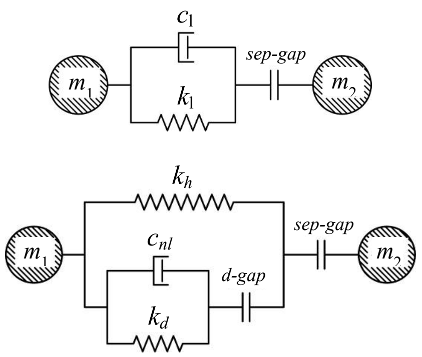

Fig. 1. Rheological schemes of Kelvin-Voight linear elastic and Jankowski non-linear elastic impact models.

A gap element placed in series with the damper (*d-gap* in the lower image in Fig. 1) is included also in the non-linear viscoelastic model proposed by Jankowski [5], where the spring is assumed to be non-linear and responding to the Hertz law with n=3/2, and the damping coefficient is transformed into a non-linear function of the time-varying interpenetration depth of the deformed colliding structures, $\delta(t)$. The damper impact force is kept as a linear function of the interpenetration velocity $\dot{\delta}(t)$.

The expressions of non-linear damping coefficient c_{nl} and Jankowski impact damping ratio ξ_J are as follows:

$$c_{nl}(t) = 2\xi\sqrt{k_h\sqrt{\delta(t)}\frac{m_1 m_2}{m_1 + m_2}} \tag{4}$$

$$\xi_J = \frac{9\sqrt{5}}{2}\frac{1-r^2}{r[r(9\pi - 16) + 16]} \tag{5}$$

where k_h is the stiffness of the Hertzian impact spring (which has the dimensions of a force divided by a 3/2-power law of displacement).

In the rheological scheme of Jankowski model, the elastic spring with k_d stiffness introduced in parallel with the damper is aimed at driving the latter to its pre-impact position before a new contact occurs. The analytical model expressed by (4) and (5) is an extension of the Kelvin-Voight model defined by (2) and (3), without the physical incongruence observed for the latter at the rebound phase, thanks to the presence of the *d-gap*. Furthermore, it provides a more careful description of the energy dissipation mechanism involved in pounding, due to the spring and damper being non-linear. However, as the spring and damper reaction forces don't reach their respective peaks simultaneously, the impact force–time curve does not vary smoothly when passing from the approach phase to the rebound one. This discontinuity does not correspond to the physical expectations about the time-evolution of the collision force. Moreover, the dependence of c_{nl} on time makes the numerical time-history analysis more burdensome.

Other more elaborated impact models have been proposed in literature, among which a non-linear viscoelastic scheme incorporating a Hertzian damper (also named Hertz-damp model) [8]. This scheme, extrapolated from different engineering research areas, such as robotics and multi-body systems, has been later modified to overcome an incongruence in the impact damping ratio estimate [10].

Due to their accentuated degree of non-linearity, these models are affected by notable uncertainties in the calibration of relevant characteristic parameters, and require a great computational effort. Therefore, as long as experimental research cannot demonstrate the greater reliability of the most complex non-linear models and at the same time cannot allow their better parameter tuning, the simplest linear viscoelastic Kelvin-Voight–like assembly can be still suggested for use in the time-history analysis of pounding structures.

Anyway, specific modifications capable of removing the pulling (tensile) damping force at the rebound phase, not based on the mere numerical artifice represented by the incorporation of an in series gap element, as discussed above, are required to improve the conceptual basis of the linear viscoelastic model.

The study summarized in this paper is carried out within this research framework. The modification introduced as compared to the classical analytical elaboration of the model presented in [9], consists in a different hypothesis about the instant of separation between the colliding structures.

In [9] the instant coincides with the condition: $u_1(t)$=$u_2(t)$, where $u_1(t)$, $u_2(t)$ are the displacements of the colliding members. This condition does not consider that the contacting surfaces get deformed during impact, and thus separation is anticipated. Here, instead, separation is assumed to occur when the impact forces get annulled, i.e. when the following condition is reached:

$$k_1\big(u_1(t) - u_2(t)\big) + c_1\big(v_1(t) - v_2(t)\big) = 0 \tag{6}$$

In this hypothesis, named: $m = \frac{m_1 m_2}{m_1 + m_2}$, $\omega^2 = \frac{k_1}{m}$, $\omega_d = \sqrt{1-\xi^2}$, the relation between r and ξ results to be as follows:

$$r = -\frac{e^{-\omega\xi\,\mathrm{arctg}\left(\frac{2\xi\sqrt{1+\xi^2}}{1+2\xi^2}\right)}}{\omega_d}\sin\left(\omega_d\,\mathrm{arctg}\left(\frac{2\xi\sqrt{1+\xi^2}}{1+2\xi^2}\right)\right) \tag{7}$$

In order to obtain the dual form typical of (3) and (5), where the damping ratio is expressed as a function of the restitution coefficient (and not vice versa), as (7) cannot be inverted to analytically derive this form, an approximated ξ–r

relation can be determined by numerical interpolation of this equation. The best fitting relation is

$$\xi = \frac{1}{4}\left(r^{-0.85} - 1\right) \tag{8}$$

Relations (3), (7) and (8) are plotted in Fig. 2, which shows a satisfactory correlation between the analytical and interpolated expressions of the modified Kelvin-Voight model proposed in this section. Remarkable differences with the original model [9] are noticed in the [0-0.5] r sub-range, where the curves of the modified model approach infinity as r tends to zero (theoretical condition of perfectly plastic impact). This trend is consistent with the physical interpretation of impact, and also characterizes relation (5) of the non-linear viscoelastic Jankowski model. The three relations provide nearly coincident ξ values in the [0.8-1] sub-range, and rather similar values in the [0.6-0.8] range, which includes the value of the damping ratio most commonly adopted, i.e. 0.65, which represents the basic choice also in the case study analyses presented in the next section.

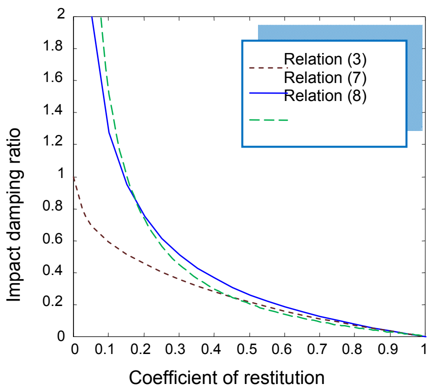

Fig. 2. ξ–r relations (3) and (8), and r–ξ relation (7).

The modified Kelvin-Voight model removes the pulling effect of the original model and, unlike Jankowski scheme, provides a smooth (continuous) impact force-time response curve over the approach and rebound collision phases.

III. Analysis of Case Study Pounding

The case study examined herein is represented by two adjacent six story R/C frame buildings sited in Pordenone, Friuli region – Italy, designed and built in the early 1960s (Fig. 3). The town of Pordenone lies in a medium seismicity area, characterized by the following site-peak ground accelerations prescribed by the new Italian Seismic Standards [11] for the four assumed reference design earthquake levels (frequent—FDE, with 81% probability of being exceeded over 50 years; serviceability—SDE, with 63%/50-year probability; basic—BDE, with 10%/50-year probability; and collapse prevention—CPE, with 5%/50-year probability), and B-type soil conditions (deposits of very thick sand, gravel, or very stiff clay, several dozens of meters thick): a_{FDE}=0.065 g; a_{SDE}=0.084 g; a_{BDE}=0.236 g; and a_{CPE}=0.298 g.

The two buildings have the same interstory heights, and the same overall structural height (with a small difference in the total architectural height, due to the presence of a parapet at the roof level of the right building in Fig. 3). The left building includes a penthouse on the top floor, which covers about half of the surface in plan. The second half is a terrace, situated on the front façade, covered by a light metal and glass structure. The main skeleton frames are parallel to the longitudinal direction in plan x, which also constitutes the pounding direction. The separation gap along the height is equal to 20 mm, determined by the thickness of the wooden planks making up the formwork of the columns of the right building, which was built two years later than the left building. This is a recurrent configuration for a large stock of R/C structures built in Italy during that decade, in the absence of reference Seismic Standards.

Fig. 3. General views of the main façades of the buildings, and detailed view of the joint zone on top of the ground floor.

The finite element model of the two structures joined by contact elements assembled according with the linear viscoelastic impact rheological scheme discussed in the previous section, characterized by the r–ξ relation expressed by (7), is displayed in Fig. 4. The model was generated with the SAP2000NL commercial calculus program [12], which allows computing dynamic response by a Fast Non-linear Analysis approach that is an alternative solution to the traditional step-by-step time integration approach, with remarkable savings in processing delays.

At an early stage of the assessment enquiry, the modal analysis of the two structures was carried out separately. Rather similar modal characteristics emerged, highlighted by a purely torsional first vibration mode, with period equal to 1.98 s and effective modal mass equal to 18% of the total seismic mass, for the left building, and to 1.79 s and 25%, for the right one. The second and third modes are mainly translational along the y and x axes in plan, respectively. Relevant periods and masses are as follows: 1.47 s and 72% (y), 1.16 s and 63% (x), for the left building; 1.6 s and 49% (y), 1.18 s and 82% (x), for the right building.

Consistently with these modal characteristics, the response history analyses carried out with sets of seven artificial accelerograms generated from the pseudo-acceleration response spectra referred to the four normative earthquake levels mentioned above, highlighted torsion-dominated pounding effects. The highest demand was computed in both structures along the perimeter frame of the main façade, which is opposite the C-shaped R/C wall enclosing the elevators.

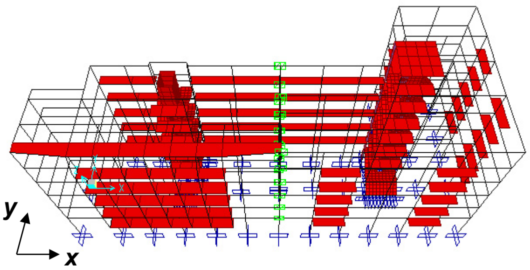

Fig. 4. Finite element model of the two structures, including linear viscoelastic impact elements governed by (7) across the separation gap.

As way of example of the results of the dynamic analyses, the impact force time-history of the façade frames obtained from the most demanding input motion scaled at the CPE level intensity, is plotted in Fig. 5.

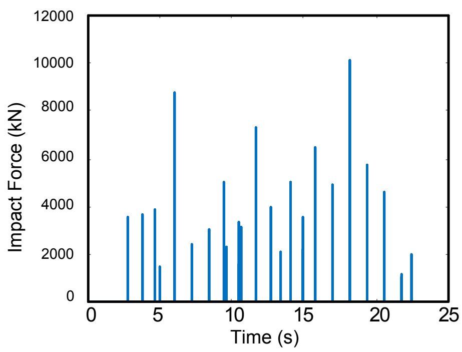

Fig. 5. Impact force time-history of the façade frames obtained from the most demanding input motion scaled at CPE level intensity.

Peak forces were found to be greater than 10,000 kN, and about 20% and 40% lower in the other two colliding frame alignments. This causes unsafe conditions for 53 columns (out of 139 in total) for the left building, and 77 columns (out of 154), for the right one. Severe damage is also noticed for most beams and the R/C shaped walls.

IV. Characteristics of Fluid Viscous Dissipaters Adopted for Pounding Mitigation

Two types of FV dissipaters were adopted in the mitigation hypothesis formulated for this case study. The first type is represented by spring-dampers made of an internal cylindrical casing filled with a compressible silicone fluid pressurized by a static pre-load applied upon manufacturing; of a piston moving in this fluid; and of an external casing (Fig. 6). The operating mechanism is based on the silicone fluid flowing through the thin annular space found between the piston head and the internal casing [13]-[15]. The inherent re-centering capacity of the device is ensured by the initial pressurization of the fluid [13], [16].

The total dynamic reaction force exerted by the device is the sum of $F_d(t)$ damping and $F_{ne}(t)$ non-linear elastic reaction forces corresponding to their damper and spring functions, respectively. $F_d(t)$ and $F_{ne}(t)$ can be expressed analytically as follows [17], [13]:

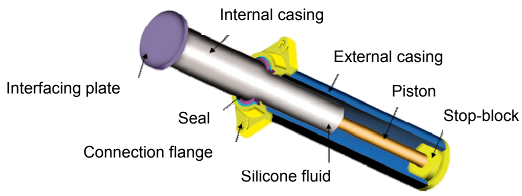

Fig. 6. Cross section of a pressurized FV spring-damper.

$$F_d(t) = c\,\mathrm{sgn}(\dot{x}(t))\left|\dot{x}(t)\right|^{\alpha} \tag{9}$$

$$F_{ne}(t) = k_2 x(t) + \frac{(k_1 - k_2)\,x(t)}{\left[1 + \left|\dfrac{k_1 x(t)}{F_{0d}}\right|^{R}\right]^{1/R}} \tag{10}$$

where c=damping coefficient; sgn(·)=signum function; |·|=absolute value; α=fractional exponent, ranging from 0.1 to 0.2; F_{0d}=static pressurization pre-load; k_1, k_2=stiffness of the response branches situated below and beyond F_{0d}; and R=integer exponent, set as equal to 5 [13], [18]-[20]. The finite element model of FV spring-dampers is obtained by combining in parallel a non-linear dashpot element and a non-linear spring element with reaction forces given by (9) and (10), respectively. Both types of elements are currently incorporated in commercial structural analysis programs, like the SAP2000NL code used in this study [12]. In this assembly, the static pre-load F_{0d} is imposed as an internal force to a bar linking the two elements. In order to simulate the attainment of the spring-damper strokes, the device model can be completed by adding a "gap" element and a "hook" element, aimed at disconnecting the device when stressed in tension, and at stopping it when the maximum displacement in compression is reached, respectively [21]-[24].

The second type of FV device is a simple damper, without the spring function. In this case, the piston crosses the external casing on both sides. The response of the damper is described by (10) too, and its finite element model is obtained by the same assembly as described above, but not including the spring component.

V. Mitigation of Case Study Pounding

The basic objective of the mitigation hypothesis formulated in this section was to prevent collisions up to the CPE level of seismic action. The design solution consisted in incorporating four FV spring-dampers and seven FV pure dampers. The former were placed across the two perimeter frames on the second and third stories, so as to produce a small increase in the separation gap at rest (about 10 mm). This allows extending the available free displacement at the approaching phase, and thus the response cycles of the devices and their dissipative action. The spring function was not required on the upper stories, where the seven pure dampers were put across the perimeter frames too (fourth and fifth stories), on both sides of the penthouse terrace (fifth), and on the central frame (sixth). The positions of the eleven devices are highlighted with rhomboidal (spring-dampers) and triangular (pure dampers) arrows in Fig. 7, where an elevation view of the structural model and of the fifth story plan are drawn. The maximum energy dissipation capacity and stroke of the selected devices are as follows [25]: 50 kJ and 120 mm—spring-dampers; 57 kJ and 100 mm—pure dampers.

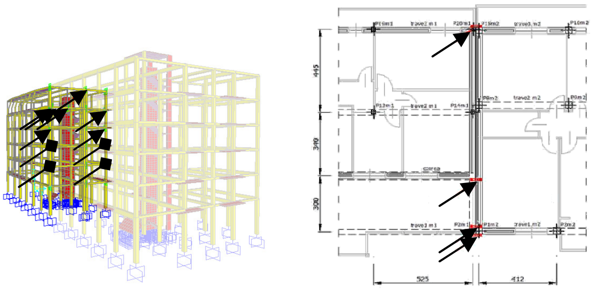

Fig. 7. Positions of the FV devices in elevation and on the fifth story plan.

Renderings of the installation of two FV spring-dampers on the façade, before and after their covering with metallic carters, and of one of the two pure dampers situated on the penthouse terrace of the left building, are illustrated in Fig. 8.

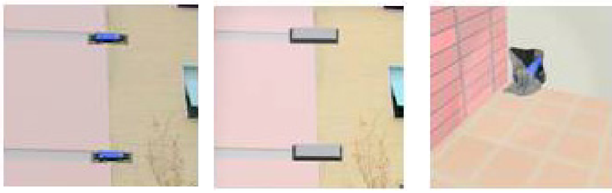

Fig. 8. Renderings of the installation of some FV devices.

The envelope of the maximum relative displacements between the two structures obtained from the most demanding CPE-scaled input motion is plotted in Fig. 9. Thanks to the protective action guaranteed by the FV devices, the displacements are constrained below 20 mm up to the sixth story. This allows meeting the targeted no-collision objective, also without considering the increase in gap depth produced by the spring-dampers located on the third and fourth stories.

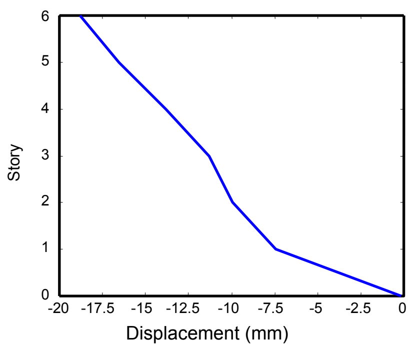

Fig. 9. Envelope of the maximum relative displacements between the two structures obtained from the most demanding CPE-scaled input motion.

The number of columns in nominally unsafe conditions is reduced to only 2 and 5 for the left and right building, respectively. This is a consequence of the retrofit intervention, which not only prevents the structures from pounding, but also remarkably reduces their response as compared to the theoretical case where the separation gap would be enough to accommodate their free relative oscillations. Indeed, in this case the unsafe columns would be 29 (left) and 43 (right). Similar improvements are also noticed for the beams and the C-shaped walls.

VI. Conclusion

The analysis of seismic pounding represents one of the most topical research fields in earthquake engineering. Experimental studies are still required, especially on large-scale structural prototypes, to definitely validate the analytical models used to simulate impact between colliding buildings. Theoretical improvements of these models, as well as updated criteria for their numerical implementation, should also be developed further.

Concerning pounding mitigation, a variety of highly protective, limitedly invasive and relatively low-cost solutions can nowadays be obtained by incorporating passive energy dissipaters. The most proper choice of the damping devices, as well as their optimal sizing and installation procedures represent challenging topics for researchers and designers.

The study summarized in this paper was aimed at offering some contributions both from an analytical modeling and a technical mitigation viewpoints. The modified version of the linear viscoelastic model obtained by equating the contact forces of the colliding structures at the instant of impact, rather than their displacements, allows avoiding a spurious pulling rebound force. At the same time, unlike the non-linear viscoelastic Jankowski rheological scheme, the updated linear model provides a smooth impact force–time response curve over the approach and rebound collision phases.

The interconnection-based solution devised for pounding mitigation, based on the incorporation of fluid viscous dissipaters across the separation gaps, offered positive indications in the case study examined here. This was assessed by achieving the highly demanding performance objective of no collisions for the seismic action scaled up to the intensity of the collapse prevention earthquake level, starting from a minimal at-rest depth of the existing gap between the two considered buildings.

Furthermore, it was observed that, in addition to the effective pounding prevention obtained, the incorporation of FV devices remarkably reduced the response of the buildings as compared to their theoretically independent (non-pounding) response. Based on this observation, the mitigation intervention proposed in this study can be viewed as a global seismic retrofit strategy for adjacent structures featuring inadequate separation gaps.

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