Damped interconnection-based mitigation of seismic pounding between a R/C tower and a masonry church

Damped Interconnection-Based Mitigation of Seismic Pounding between a R/C Tower and a Masonry Church

F. Pratesi[a], S. Sorace[b] and G. Terenzi[a]

[a]*Department of Civil and Environmental Engineering, University of Florence*
Via di Santa Marta 3, 50139 Florence, Italy
[b]*Department of Civil Engineering and Architecture, University of Udine*
Via delle Scienze 206, 33100 Udine, Italy

Abstract. Damped interconnection represents an advanced strategy to mitigate the effects of seismic pounding between adjacent structures built at poor mutual distance. The effects of pounding can be particularly severe in slender R/C structures, including civic or bell towers. An emblematic case study falling in this class of structures, i.e. a monumental R/C bell tower constructed in the early 1960s in Florence, is analyzed in this paper. In order to assess the effects of pounding, a non-linear dynamic finite element enquiry was carried out by simulating collisions between the tower and the adjacent masonry church with a multi-spring-damper viscoelastic contact model, originally implemented in this study. The survey results show that pounding affects the seismic response of the two buildings as early as an input seismic action scaled at the amplitude of the normative basic design earthquake level. A damped interconnection-based retrofit hypothesis to prevent pounding is then proposed, which consists in linking the two structures by means of a pair of fluid-viscous dissipaters. Thanks to the added damping produced by these devices, the impacts are totally annulled, bringing the structural members of the tower to safe levels.

Keywords: Added Damping, Passive Control, Seismic Pounding Mitigation, Non-linear Fluid-Viscous Dissipaters, R/C Towers, Structural Assessment, Seismic Retrofit.
PACS: 46.70.-p

INTRODUCTION

One of the greatest sources of vulnerability of structures designed before the advent of modern Technical Standards is represented by seismic pounding, which occurs when the distance between adjacent buildings is not wide enough to avoid collision during their earthquake-induced motion. The damaging effects of pounding can be particularly severe in slender R/C structures, including various types of civic or bell towers. Indeed, these buildings are characterized by high horizontal translational deformability at least in one of the two main directions in plan, and by limited structural redundancy. In this respect, the case study analyzed in this paper is emblematic, being represented by a R/C bell tower constructed in the early 1960s to replace the existing 19th century tower of the Chiesa del Sacro Cuore in Florence (Figure 1). The new tower was built at a very narrow distance from the façade of the church due to the lack of reference Technical Standards at the time, as typical of a large stock of modern heritage R/C structures [1]. As a consequence of the little width of the gap, averagely equal to 20mm along the height, the two structures appear to be remarkably pounding-prone.

MODAL ANALYSIS OF THE BELL TOWER AND THE CHURCH

A complete finite element model of the church and the bell tower was generated by the SAP2000NL commercial calculus program [2], based on the original design documentation collected through record research, as well as on supplementary laser measurements [3]. The model of the tower was constituted by a complete mesh of frame elements. The model of the church included frame elements, for the wooden roof and the R/C dome, and shell elements, for the masonry walls. A modal analysis of the two structures was initially carried out by the model, the results of which show a first mode of the tower alone, mixed translational along the direction orthogonal to the façade (y)–rotational around the vertical axis (z), with vibration period of 1.94 s and effective associated masses equal to 80% of the seismic mass of the tower along y, and 22.4% around z. The corresponding shape is plotted in the right image of Figure 1. The second mode, concerning the bell tower alone too, is mixed translational along the direction parallel to the façade (x)–rotational, with period of 1.39 s and associated masses equal to 93.1% along x, and 66.9% around z. The third mode is translational along x–rotational, with period of 0.39 s and associated masses

11th International Conference of Numerical Analysis and Applied Mathematics 2013
AIP Conf. Proc. 1558, 2466-2469 (2013); doi: 10.1063/1.4826040

equal to 3.4% along x, and 2.7% around z. The fourth, and last significant, mode is translational along y–rotational, with period of 0.38 s and associated masses equal to 14.9% along y, and 3.9% around z. The church structure features all mixed translational–rotational modes too, the former of which with period of 0.48 s. The first modes that include a significant translational contribution in y direction are the fifth and sixth ones, with vibration periods of 0.21 s and 0.17 s, equal to about 1/9 and 1/11 of the first period of the tower along the same axis. This highlights very different dynamic properties of the two structures along the potential pounding direction, as expected from their structural characteristics.

FIGURE 1. Views of the bell tower and the church and first mode shape of the finite element model of the two structures.

NON-LINEAR DYNAMIC ANALYSIS OF POUNDING

A special "multi-spring-damper" viscoelastic contact model constituted by an in-series assemblage of m linear dampers and m associated springs was devised in this study to simulate the effects of pounding in finite element time-history analyses. The response of the model is based on the sequential activation and disconnection of the dampers and relevant springs, following the variation of the interpenetration depth $\delta(t)$ (with t=time variable) between the colliding structures. This way, the resulting equivalent damping coefficient of the assemblage, c_{eq}, becomes a function of $\delta(t)$, which allows reproducing in piece-wise linear form any relation between c_{eq} and $\delta(t)$ likely to be selected in the analysis. The special relation proposed in [4]:

$$c_{eq}(t) = 2\xi\sqrt{k_H\sqrt{\delta(t)}\,\frac{m_1 m_2}{m_1 + m_2}} \tag{1}$$

was particularly adopted in this enquiry, where m_1, m_2=masses of the impacting structures; k_H=stiffness of the impact force transmitting spring; and ξ= impact damping ratio, defined in [4] as the following function of the coefficient of restitution r:

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

Consistently with model [4], the impact force transmitting spring is assumed to be non-linear and governed by Hertz analytical law, i.e. expressed as a n-power law of the relative displacement between the colliding members, with the n exponent fixed at 3/2 for pounding computation. The version of the multi-spring-damper model with m=5 components is drawn in Figure 2a. The activation of each damper, with damping coefficient c_i — where i=1,..,5 in this case, and i=1,..,m in general — is governed by a gap (named gap_{-Ci} in Figure 2a), to which an initial opening is assigned. As the gap closes, the damper starts to react, adding its response to the already activated dampers. Each damper is driven to its pre-impact position by the associated linear spring, with stiffness k_{di}. The contact element is completed by the impact-transmitting Hertzian spring described above, placed in parallel to the multi-spring-damper assemblage; the existing gap at rest between the two adjacent structures (named *rest-gap*); and the gap that disconnects the damper in the rebound phase (*reb-gap*). A numerical enquiry carried out by varying the number of linear dampers from 3 to 9 highlighted that the 5-damper assemblage in Figure 2a is capable of bearing the best balance between simulation capacities and computational times. Therefore, this version of the multi-spring-damper model was incorporated at the interface between the bell tower and the church for the time-history analysis of pounding, in the positions marked by the pairs of joints denoted by letters A-A' through E-E' in Figure 2b. These joints represent the potential physical impact spots situated on the four rear columns and at the top of the rear arcade beam of the tower, and the corresponding spots on the church façade.

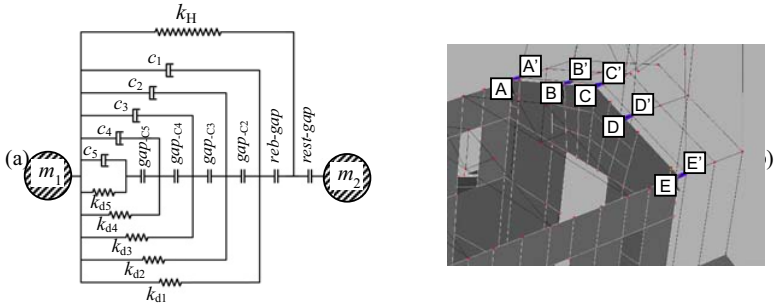

FIGURE 2. Multi-spring-damper contact model and positions of the five contact elements incorporated in the finite element model of the two structures.

The non-linear dynamic analyses were carried out by assuming seven artificial ground motions generated from the pseudo-acceleration elastic response spectrum of the Italian seismic Standards for the city of Florence, scaled at the amplitude of the basic design earthquake level (BDE, with a 10% probability of being exceeded over the reference period of 50 years fixed for the building), and the local soil conditions of the church site. As way of example of the results of the numerical enquiry, Figure 3 includes plotting of the interpenetration depth and contact force time-histories obtained from the most demanding of the seven motions for the element linking joints C and C', for which the maximum values of both quantities were surveyed. These graphs highlight maximum δ values equal to 5.6 mm, and peak impact forces of about 165 kN. Similar results are obtained for the other contact elements, assessing severe pounding response conditions. These data are reflected in the stress states of the tower members, and particularly of the columns, where maximum percent increases of about 33% in bending moment, 25% in shear, and 9% in normal force, are observed in pounding conditions. This causes the current safety margins of the columns to annul totally as early as the BDE level of seismic action, and prompts to adopt a pounding mitigation strategy.

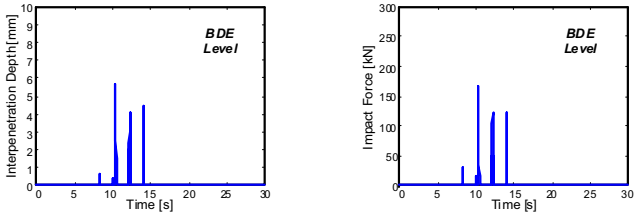

FIGURE 3. Interpenetration depth and impact force time-histories obtained from the most demanding input motion for C-C' joints.

DAMPED INTERCONNECTION-BASED MITIGATION STRATEGY

The pounding mitigation strategy selected for this case study, recently proposed for application to adjacent frame buildings [5], consists in linking the potentially colliding structures with high-capacity fluid-viscous (FV) dampers. This class of dissipaters has been the subject of a wider research activity developed over the last two decades by the second and third author, which included several FV damper-based seismic protection technologies [6-15]. The analytical expression of the damping reaction force F_{FV} exerted by the dissipaters is [6]:

$$F_{FV}(t) = c_{FV} \cdot sgn(\dot{d}(t)) \cdot \left|\dot{d}(t)\right|^{\alpha} \tag{3}$$

where d=displacement; $\dot{d}$ =velocity; c=damping coefficient; $sgn(\cdot)$=signum function; $|\cdot|$=absolute value; and α=fractional exponent ranging from 0.1 to 0.2. In order to keep the architectural intrusion of the intervention to the minimum, only two FV dissipaters were installed, and namely in the positions marked by the pairs of joints A-A' and E-E' in the Figure 2b. The design analysis led to select the following properties of the two devices: c_{FV}=600 kN(s/m)$^{\alpha}$: α=0.15; maximum reaction force $F_{FV,max}$=500 kN; stroke s=±50 mm; maximum damping energy capacity E_d=100 kJ. A new set of non-linear dynamic analyses was carried out to evaluate the benefits of the intervention, removing the five contact elements from the computational model and substituting them with the two FV dampers, so as to reproduce the new structural configuration. The results are summarized in Figure 4, which includes plotting

of the time-history of relative displacements of the bell tower with respect to the church (measured again at the top C-C' position in Figure 2b), obtained from the most demanding input ground motion scaled at BDE amplitude.

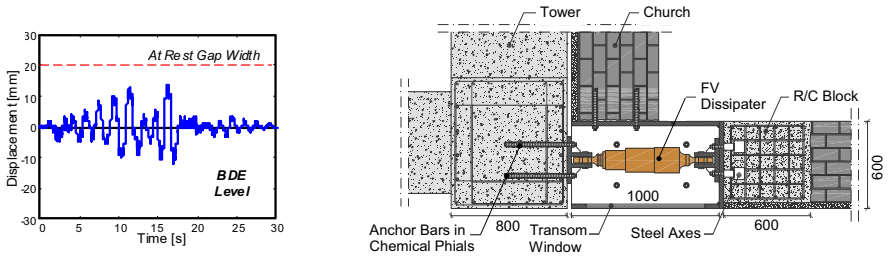

FIGURE 4. Relative displacement time-history obtained from the most demanding input motion and installation detail of one of the two dampers (plan view).

The graph shows that the maximum value of the relative displacement is equal to 13.8 mm. As this value is lower than the assumed gap depth at rest, pounding does not occur. Concerning the technical installation of the dampers, it is very simple and non-invasive from an architectural viewpoint, as shown in the design drawing in Figure 4, displaying a plan view of the intervention.

Based on the results of this study, the proposed damped-interconnection strategy appears to be an effective pounding mitigation strategy also for special slender R/C structures, such as the one examined in this paper.

ACKNOWLEDGMENTS

The study reported in this paper was sponsored by the Italian Department of Civil Protection within the Reluis-DPC Project 2010/2013. The authors gratefully acknowledge this financial support.

REFERENCES

1. S. Sorace and G. Terenzi, Structural assessment of a modern heritage building, *Engineering Structures* **49**, 743-755 (2013).
2. Computers and Structures Inc., SAP2000NL. Structural Analysis Programs, Berkeley, CA: CSI, 2012.
3. F. Pratesi, "La chiesa del Sacro Cuore a Firenze. Analisi storico-critica, vicende del cantiere, studio del martellamento strutturale e proposta di adeguamento sismico" [in Italian], Degree Thesis: University of Florence, 2012.
4. R. Jankowski, Non-linear viscoelastic modelling of earthquake-induced structural pounding, *Earthquake Engineering and Structural Dynamics* **34**, 595-611 (2005).
5. S. Sorace and G. Terenzi, Advanced protection of seismic pounding between adjacent buildings, *WASET*, **71**, 178-184 (2012).
6. S. Sorace and G. Terenzi, Non-linear dynamic modelling and design procedure of FV spring-dampers for base isolation, *Engineering Structures* **23**, 1556-1567 (2001).
7. S. Sorace and G. Terenzi, Seismic protection of frame structures by fluid viscous damped braces, *Journal of Structural Engineering, ASCE* **134**, 45-55 (2008).
8. S. Sorace, G. Terenzi, G. Magonette and F.J. Molina, Experimental investigation on a base isolation system incorporating steel-Teflon sliders and pressurized fluid viscous spring dampers, *Earthquake Engineering and Structural Dynamics* **37**, 225-242 (2008).
9. S. Sorace and G. Terenzi, Analysis and demonstrative application of a base isolation/supplemental damping technology. *Earthquake Spectra* **24**, 775-793 (2008).
10. S. Sorace and G. Terenzi, Fluid viscous damper-based seismic retrofit strategies of steel structures: General concepts and design application, *Advanced Steel Construction* **5**, 322-339 (2009).
11. S. Sorace and G. Terenzi, The damped cable system for seismic protection of frame structures—Part I: General concepts, testing and modeling, *Earthquake Engineering and Structural Dynamics* **41**, 915-928 (2012).
12. S. Sorace and G. Terenzi, The damped cable system for seismic protection of frame structures—Part II: Design and application, *Earthquake Engineering and Structural Dynamics* **41**, 929-947 (2012).
13. S. Sorace, G. Terenzi and G. Bertino, Viscous dissipative, ductility-based and elastic bracing design solutions for an indoor sports steel building, *Advanced Steel Construction* **8**, 295-316 (2012).
14. S. Sorace, G. Terenzi and F. Fadi, Shaking table and numerical seismic performance evaluation of a fluid viscous-dissipative bracing system, *Earthquake Spectra* **28**, 1619-1642 (2012).
15. S. Sorace and G. Terenzi, Dissipative bracing-based seismic retrofit of R/C school buildings, *The Open Construction & Building Technology Journal* **6**, 334-345 (2012).