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SHAKING TABLE TESTING OF AN RC FRAME WITH DISSIPATIVE BRACINGS

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SUMMARY

Dissipative Bracings (DB) have been used over the last decade, mostly in North America and Japan, for controlling the seismic response of framed buildings through the absorption of a portion of the input energy. However, most of these applications, as well as numerical and experimental research, concern steel structures. In Europe, on the contrary, the main interest in DB is for retrofit of RC-framed low-rise buildings designed according to non-seismic specifications or old seismic codes. These structures are more rigid than tall steel frames, and thus require DB able to dissipate energy at very small displacements. The research presented here is aimed at verifying experimentally the design methods used to optimise the location and characteristics of DB for installation in RC buildings, as well as evaluating the relative performance of three damping technologies, i.e. non-linear fluid viscous damping, elastomeric viscoelastic damping, and steel hysteretic damping. Shaking table tests are under way on two identical one-bay, 2 storey, full-scale spatial RC-frame structures, one bare and the other equipped with DB. The frames are designed to be representative of existing structures built before the introduction of recent seismic codes based on ductility requirements. The results will make an important contribution to the future extension of the European Building Code for seismic regions (EC8) to cover structures with auxiliary damping devices.

INTRODUCTION

Dampers have been used over the last three decades for controlling response of buildings and other structures to external dynamic loadings. One of their early applications in buildings has been the installation of Elastomeric Visco-Elastic Dampers (EVED) in the twin towers of World Trade Centre in New York for reducing displacements and accelerations, and thus increasing human comfort, due to wind excitations (Mahmoodi [1]). Since then the use of dampers has been extended to the field of

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seismic engineering. Some of the early studies on the use of dampers in Dissipative Bracings (DB) for seismic protection of framed buildings are described by Pall [2], Filiatrault [3] and Giacchetti [4]. The basic role of dampers incorporated into a structure is to absorb a portion of the input energy, thereby reducing the energy dissipation demand on primary structural members and minimising possible structural damage. The principal types of damper currently available on the market are EVED, Fluid Viscous Dampers (FVD), Steel Hysteretic Dampers (SHD) and Friction Dampers (FD).

In Europe, despite the high level of research by its industry and research institutions, dampers have been mainly applied to bridges, whereas in North America and Japan there have been many applications of DB in buildings. This is believed to be due to the fact that most European buildings have RC frames with a small number of storeys. Such structures are more rigid than those with high steel frames, and thus require dampers able to dissipate energy at very small displacements. Furthermore, installation of DB in existing RC frames is more problematic than in steel frames (e.g. due to tension in the columns). The difficulties have so far hindered the application of DB to the protection of RC frame structures. One of the very few exceptions is the seismic retrofit of a RC framed school damaged by the Umbria-Marche 1997 earthquake with EVED (Antonucci [5]). This type of intervention is expected to be very competitive, both technically and economically, for the seismic retrofitting of the many RC framed buildings, particularly those in Europe and the Mediterranean area, constructed before the introduction of modern seismic codes.

Thus, despite some numerical and experimental studies on RC structures protected by DB having already been performed (e.g. Valente [6], Fuller [7]), there is a need to investigate further and optimise the use of dampers in RC framed buildings.

This paper is related to a study aimed at verifying experimentally the design methods used to optimise the location and characteristics of DB for installation in RC buildings, as well as evaluating the relative performance of three damping technologies, i.e. non-linear fluid viscous damping, elastomeric viscoelastic damping, and steel hysteretic damping. Shaking table tests are under way on two identical one-bay, 2 storey, full-scale spatial RC-frame structures, one bare and the other equipped with DB. The frames are designed to be representative of existing structures built before the introduction of recent seismic codes based on ductility requirements. The paper reports the preliminary results of this research, in particular the numerical analyses aimed at the selection of the optimal design parameters for each type of DB.

DISSIPATIVE BRACINGS

The DB considered in this study have one of three different types of constitutive behaviour: viscous, visco-elastic and elasto-plastic. From the various technologies having one of these types of behaviour, those more suitable for application to RC frames have been selected. In particular, amongst viscous dampers, those with highly non–linear behaviour, i.e. maximum energy dissipation efficiency, have been selected. Elasto-plastic hysteretic behaviour may be obtained by yielding of steel elements; a lot of different shapes of element have been tried. Most steel hysteretic dissipating elements commercially available operate by bending, but this type is not very suitable for application to RC structures owing to its relatively low elastic stiffness (and consequently high yielding displacement). Torsion-based steel hysteretic dampers could be suitable owing to their high elastic stiffness (Dusi [8]). However, here the focus is on steel hysteretic dampers operating by compression/tension, i.e. the Buckling-Restrained Braces (BRBs).

Non-linear fluid viscous dampers

As previously said, the Fluid Viscous Dampers (FVD) selected for this study are highly non-linear, i.e. with a force vs. velocity constitutive law of the type $F=C\cdot V^{\alpha}$, with $\alpha=0.15$ (see Castellano [9]). This non-linear behaviour allows the greatest dissipatory energy efficiency compared with linear FVDs or FVDs with a higher exponent α value. In fact, it guarantees significant energy dissipation even at low displacements and low velocities. Figures 1 and 2 show the behaviour of the FVD with

different exponent α values in terms of force vs. displacement and force vs. velocity graphs. Due to this behaviour, non-linear FVDs are particularly suitable for use in DB for the seismic protection of RC frames that cannot reach high inter-storey drifts.

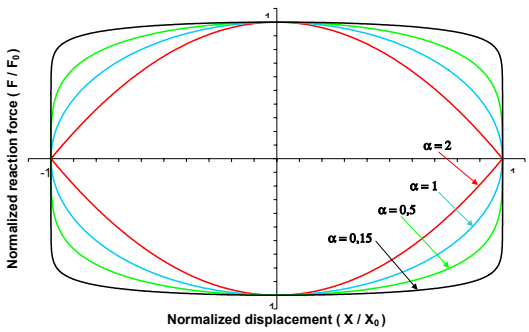

Figure 1 – Force vs. displacement curves of FVDs with different values of the exponent α.

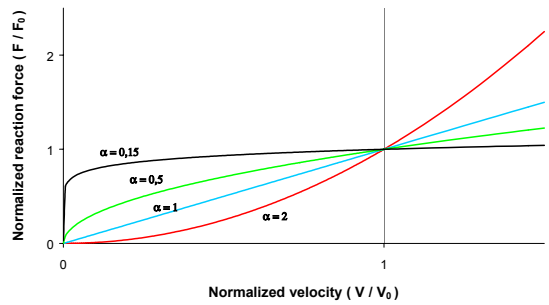

Figure 2 - Force vs. velocity curves of FVDs with different values of the exponent α.

Elastomeric visco-elastic dampers

Material

The visco-elastic material should have a shear modulus and damping that do not change excessively with temperature. This is more important than a very high level of damping. The excessive dependence of some commercially available compounds not only restricts the range of service temperatures but may also lead to heat build-up resulting in a loss of performance during an earthquake. A material selection was made on the basis of the best overall performance. Figure 3 shows the variation of the complex shear modulus (G*) and loss factor (tanδ) with temperature for the material used in the prototype devices. The shear modulus varies by about a factor of 3 over the range –20 to 40°C. The loss factor (twice the critical damping) varies between 0.45 to 0.6 over the full test temperature range.

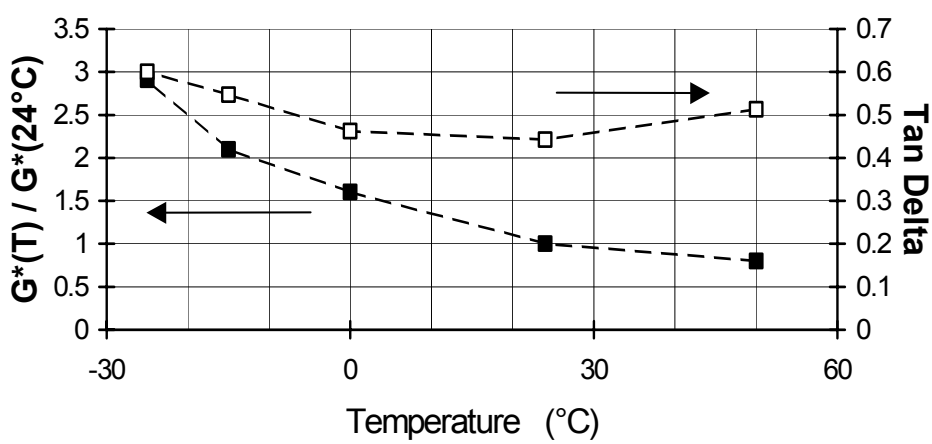

Figure 3 - The effect of temperature on the dynamic properties of the high damping material for prototype devices. Dynamic shear modulus normalized with respect to modulus at 24°C. Measurements at 50% strain, 1Hz, 3rd cycle.

Device characteristics

The design chosen for the prototype damper is shown schematically in Figure 4. It consists of a single layer of elastomer, 7mm thick and of plan dimensions 240x170mm, bonded between steel plates. The design strain in the rubber is intended to be about 100% in simple shear. The dampers

dampers were scragged, and heat treated before testing. This procedure greatly reduced the high stiffness otherwise observed in the first quarter cycle of loading; it did, however somewhat reduce the level of damping compared with the unscragged material. The dynamic stiffness and loss factor of the dampers have been measured over a range of frequencies and amplitudes. Figure 5 shows the variation with amplitude from tests carried out at a frequency of 0.1Hz. The changes in dynamic properties produced by varying the frequency over two orders of magnitude are modest (see Table 1).

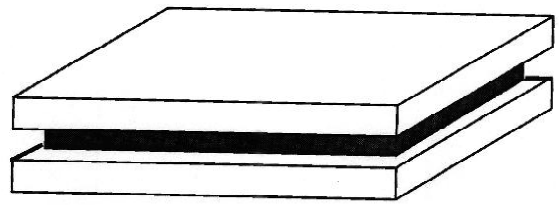

Figure 4 - Schematic View of EVED Device.

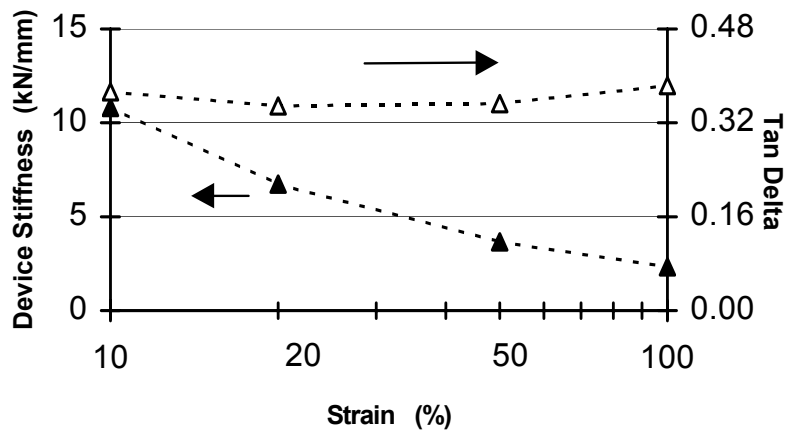

Figure 5 - Effect of strain amplitude in the rubber on the dynamic stiffness and loss factor of the prototype device. Data are for 3rd cycle.

Table 1 - Effect of frequency on EVED device stiffness and loss factor

Frequency Hz	Stiffness kN/mm	Loss factor
0.01	3.1	0.35
0.1	3.4	0.35
1	4.1	0.34
Strain amplitude 50%. Tenth cycle data		

Typical shear force-displacement loops obtained are shown in Figure 6. This data is for the small device, based on the prototype damper, used in the retrofit of a school (Antonucci [5], Figure 7). A difference in stiffness is seen for the first three cycles. Further cycling produced little change in the dynamic properties, however.

Preliminary tests

A two-storey, two-thirds scale mock-up of part of a reinforced concrete frame building of non-seismic design was constructed for pseudo-dynamic tests at JRC, Ispra, Italy (Fuller [7]). The mock-up (10m long, 4m wide and 5.2m high) represented two bays of 5m in the direction of testing and one bay across its width. The dampers were installed as pairs, one in each bay along the two longitudinal facades. The peak inter-storey drift of the mock-up at the design level earthquake (0.3g PGA) with the

devices in place was 8mm compared with 36mm without the devices. The inter-storey drift thus remained within the elastic deformation range with the devices inserted, despite the frame itself being of non-seismic design. The overall peak forces in the protected frame were similar to those in the bare frame. A high proportion of the force (55%) in the protected frame, however, was borne by the devices. The added stiffness raised the natural frequency of the mock-up from 1.7Hz to 3.5 – 5Hz (the figure depending upon amplitude because of the non-linearity introduced by the devices). The energy content of the seismic input was much higher at the higher frequencies and this explains the similarity in the peak forces induced in the bare and protected frame. Without the high damping obtained with the devices, the increased stiffness of the protected frame would have led to higher force levels. This preliminary experimental evaluation suggested that installation of EVED can successfully transform a non-seismic design RC frame to one able to restrict the response during a substantial level earthquake to elastic deformations.

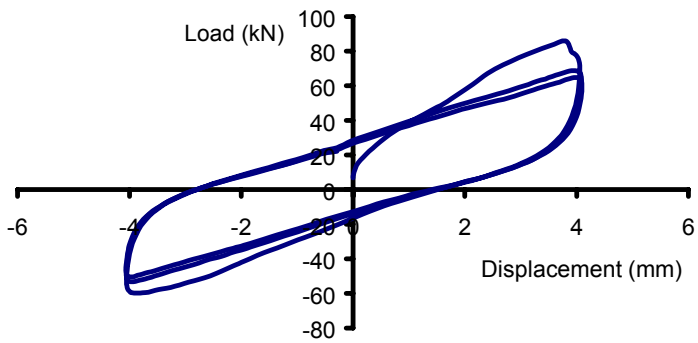

Figure 6 - Force-displacement loops obtained for a pair of EVED tested in a double-shear arrangement. Frequency 0.1Hz. Elastomer strain amplitude 100%

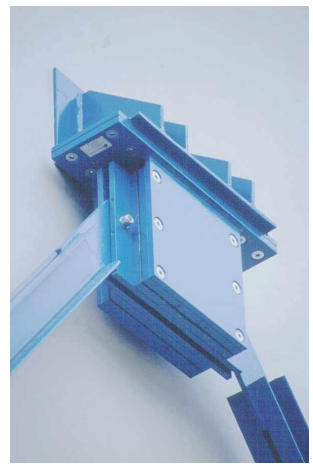

Figure 7 - An EVED installed in an RC framed building.

Buckling-restrained braces

Buckling-restrained braces are a special implementation of steel hysteretic dissipative devices, specially designed for use in framed buildings. They ensure reliable absorption of earthquake energy through tension-compression yielding.

Axial loading is the ideal way to accomplish hysteretic energy dissipation because all the resisting sections work at a uniform level. However, in general, it is not used because of Euler buckling. BRBs overcome the Euler buckling issue and allow the dissipation of energy through stable tension-compression cycles within the plastic range. As the Euler equation shows:

$$P_{eu} = \frac{\pi^2 \cdot E \cdot J}{l_0^2} \qquad (1)$$

a way to increase to increase the critical load P_{eu}, beyond increasing inertia J (that is to say reinforcing the brace), is to diminish the effective length l_0, i.e. "restraining" the buckling. Several variations of BRBs, based on different principles to restrain buckling and using different materials and geometries, have been proposed and studied extensively since 1988 (Watanabe [10]), and the same have been used in hundreds of buildings in Japan and the United States (Brown [11]), but not yet in Europe. The BRB type considered here achieves a reduction of the effective length l_0 by encasing the brace (named the core member) into a buckling-restraining system comprising an outer steel tube filled with concrete or mortar (Figure 8). A slip interface between the core member and the surrounding material ensures that only the steel core carries compressive and tensile loads. In this manner, the filling material prevents core buckling (because $l_0 \rightarrow 0$, $P_{eu} \rightarrow \infty$) and provides a hooping effect: virtually,

strut buckling resistance finds its limit only in the outer tube resistance to internal pressure exerted by the filling material. To simplify things, it can be thought that the buckling resistance of the brace equals at least the buckling resistance of the outer tube. The material and thickness of the slip interface must be carefully selected: it must absorb brace expansion under compressive loads and minimize friction, so as to allow relative movement between the members. Conversely, if it is too scarcely stiff and too thick, the constraining effect of the filling material becomes fruitless and the steel core buckles within this layer.

A lot of tests have been carried out on BRBs of different materials and geometries, to check design criteria and optimize hysteretic behaviour. Figure 9 shows typical hysteretic loops resulting from low-cycle fatigue tests, at 33%, 67% and 100% of the design displacement. The behaviour is very stable, and the fatigue life is high. There is of course, as expected, a difference between the plastic stiffness under compression and under tension, but it is very small.

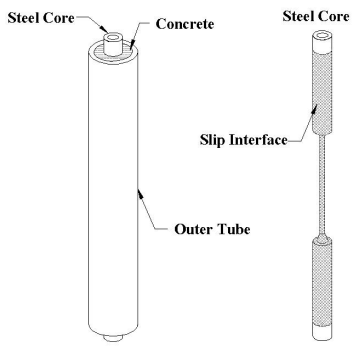

Figure 8 - An assembled BRB on the left and its steel core member on the right.

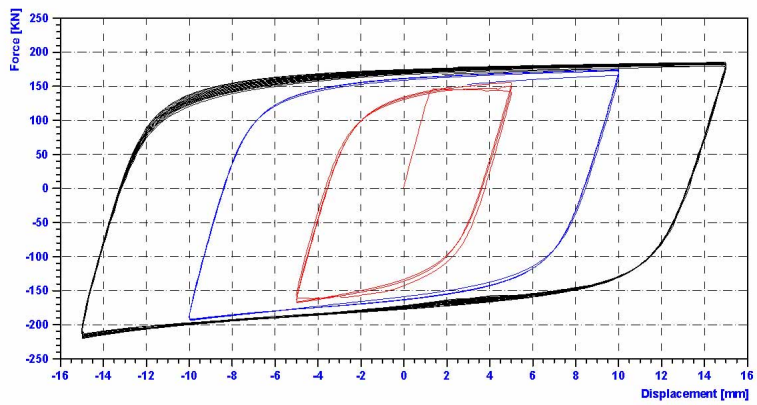

Figure 9 – Experimental hysteretic loops of a BRB at increasing displacements.

An advantage of BRBs resides in the possibility of producers to be able to independently control strength, stiffness and yield displacement and ductility by varying the cross-sectional area of the steel core, the yield strength of the steel and the length of the steel core that is allowed to yield. This provides design engineers with an opportunity to accurately tailor the force-displacement relationship of their lateral force-resisting elements to suit the needs of the application in question.

DESIGN OF THE BARE FRAME

The choice of geometry, materials and constructive details of the RC frame mock-up to be used in this experimental study has been subordinated to a number of factors. The primary objective was to create a mock-up whose seismic, non-linear response would adhere as much as possible to that of a class of real structures designed to either resist vertical loads only or accommodate lateral excitations according to inadequate seismic codes. The structural type in question is one associated with shear–type, multi-storey buildings characterised by poor strength due to scarce materials, great lateral flexibility, low level of ductility, and an uncontrolled strength hierarchy such as to yield to the soft-storey failure mechanism. In addition, both the geometrical and mechanical limits of the shaking table facility incurred some restrictions as well.

The reference shear-type, multi-storey building was simulated by a one-bay by one-bay, two-storey, RC mock-up whose structural elements at each storey are four 26x26 cm columns placed on a 4x4 m square grid, 26x40 cm two-way beams, 12 cm thick floor slabs, with a 3.30 m inter-storey height (Figure 10).

Figure 10 – The frame mock-up on the shaking table at CEA facilities.

The mock-up was designed on the basis of an old Italian seismic code. A design spectrum with PGA=0.07g was considered for the design. In order to be representative of the real buildings constructed about 20 to 30 years ago, the concrete strength was limited to a design value of 17 MPa. As far as the type of re-bars is concerned, a yielding value of 500 MPa was imposed by the manufacturer, allegedly due to the unavailability of weaker materials. The effect of live loads was simulated by applying a concrete ballast placed on each floor and rigidly connected to the slabs. In this way a uniform target load of 5.0kN/m2 was achieved. The design of the constructive details was aimed at driving the formation of plastic hinges in the desired critical regions (i.e. the columns) and at giving a response factor of about 2 to 2.5, as usual in buildings constructed about 20 to 30 years ago. A preliminary calibration based on push-over analyses was carried out to check for this behaviour.

The mock-up is being modelled using different softwares and methodologies, ranging from simple elastic linear analyses to very complex 3-D non-linear push-over analyses, to describe the frame's

dynamic and post-elastic behaviour. Both design and experimental values of material characteristics were inserted in the models. It is worthy of note that said experimental values proved better than the design values; particularly, the concrete strength at the first level (26 MPa, while the steel yielding stress is around 550 MPa). All the results of non-linear analyses show that the collapse mechanism will be of the soft-storey type, as expected. A number of analyses are underway to predict with the highest possible level of precision the evolution of plastic hinges in the frame during the tests. In effect, tests will be carried out with a sequence of artificial time-histories derived from EC8 spectrum with increasing PGA, i.e. 0.10 – 0.15 – 0.20 – 0.25g. The results of dynamic non-linear analyses with this sequence of time-histories show that the plastic mechanism starts with the yielding of steel bars in almost all the critical zones (completely, at the first story) followed by a progressive cover spalling at the bottom of the columns of the first story.

The results of the shaking table tests will be used to further improve said models, especially to understand the influence of confinement upon the inner core strength, as well as the influence of both the steel plinth's and the beam to column joint's stiffness upon the columns' flexibility, so that it will be possible to create a suitable means for further analysis with damping systems.

DESIGN OF THE FRAME WITH DISSIPATIVE BRACINGS

The mock-up that will be tested with DBs is identical to the frame that will be tested as is, in order to compare test results. A number of analyses are underway to optimise the three DB types for the frame mock-up, in an effort to allow the frame to sustain a design earthquake with PGA=0.25g without damage (non only to structural but also to non structural elements), and with at least PGA=0.35 g without collapse. The DB shape has been selected taking into account the ease and cost-effectiveness involved in the installation in actual structures with a large number of dampers. Thus, for DBs with FVD and EVED, the chevron brace shape was that of choice. Conversely, for BRBs, both the diagonal and chevron shapes were considered in preliminary analyses and then the diagonal configuration was selected thereafter because it was the most cost-effective. Only the results with the diagonal shape will be presented here.

DBs are inserted in each bay along one direction only because the shaking table tests will be unidirectional. In order to simplify the installation and substitution of different DB types during the testing campaign, DBs with FVD and EVED will be installed and tested in one direction, and BRBs in the orthogonal direction. At this stage, DBs at the first and second floor have the same characteristics. However, further analyses are underway to further optimise matters by changing the DB characteristics in each floor.

Each DB type is characterised by different damper behaviour; thus, the optimisation process is based on different design parameters, as well as different methods.

The results of numerical analyses that model the RC frame as elastic are presented in the following (with PGA=0.25 g). However, further analysis are under way, with more detailed models that take into account the non linear behaviour of the RC frame. Said models will be calibrated on the basis of the results of the tests on the bare frame, in an effort to predict test results with a high level of precision.

Optimization of viscoelastic dampers

The optimization of EVED characteristics for the frame mock-up is based on an energy oriented approach. This approach, proposed by Lin and Soong [12, 13], focuses on the modal strain energy method. It assumes that the behaviour of a structure equipped with supplemental dampers can be represented by the modal periods and shapes of the non-damped structure, using appropriate damping coefficients within the modal equations of non-damped motion. Equivalent structural damping can be expressed in energy terms as the ratio between dissipated energy and the total accumulated energy. Therefore, for a SDOF:

$$\xi = \frac{E_d}{4\pi E_{ms}} \tag{2}$$

where:

E_d = *energy dissipated by the system;*

E_{ms} = *total elastic energy accumulated by the system or strain energy*

For an MDOF system, expressions similar to (2) can be used, considering ξ as the modal damping, and E_d e E_{ms} as the energy dissipated and strain energy associated with each vibration mode. With this assumptions, it is possible to correlate the total structural damping to the stiffness of the frame, as well as the stiffness and loss factor of the DB. The latter, of course, depend not only on the EVED stiffness and loss factor but on the stiffness of the steel brace in series with the EVED as well.

Thus, the modal strain energy method is applied trough analytical formulae that correlate the global stiffness of the frame at one floor and the total characteristics of the DBs at that floor to the structural damping ξ. Furthermore, when the objective structural damping has been selected, it is possible to find the total characteristics of the DBs on a floor by floor basis.

To apply said method to the frame mock-up object of this study, the EVED loss factor was considered constant and equal to 0.34, i.e. the minimum value found in the tests discussed above. Thus, the design parameters were reduced to the horizontal stiffness of the chevron bracing K_b and the EVED stiffness K_d. A parametric analysis was then carried out varying said parameters.

Figure 11 shows the variation of the structural damping ξ with the damper stiffness K_d.for a fixed value of brace stiffness K_b. The importance of the brace stiffness is self-evident. In fact, if the braces cannot transmit the inter-storey drift to the dampers and lose part of it in their deformation, the efficiency of the damper+braces system decreases. Observing the K_b=20 MN/m curve, we can see that when the damper reaches a stiffness of about 30% of the braces stiffness, the curve tends to became flat and decrease. It has become now evident that it is not possible to reach any value of structural damping if we do not first select the correct brace stiffness. If we increase the brace stiffness, the efficiency of the damper+braces system increases and makes it possible to reach a higher value of structural damping with the same value of damper stiffness.

The effect of brace stiffness can also be observed in Figures 12 and 13. Figure 12 shows how, increasing the value of K_b, decreases the value of damper stiffness we need to reach for the target damping (10 % in this case). Figure 13 shows how structural damping increases as brace stiffness increases. The increase is not linear, so is not worth increasing K_b above certain values, because the associated decrease in damper stiffness is lowered.

On the basis of said data and a cost-effectiveness approach, the design parameters selected as optimal for the frame mock-up are K_b=80 MN/m and K_d=8 MN/m, which permit achieving a total structural damping of 10%. The shear stiffness of the frame at each floor is K_s=13680 kN/m. Thus, the selected brace has a stiffness which is about 5.8 times greater. The braces will be created using a couple of UPN120s (that yield approximately the desired stiffness).

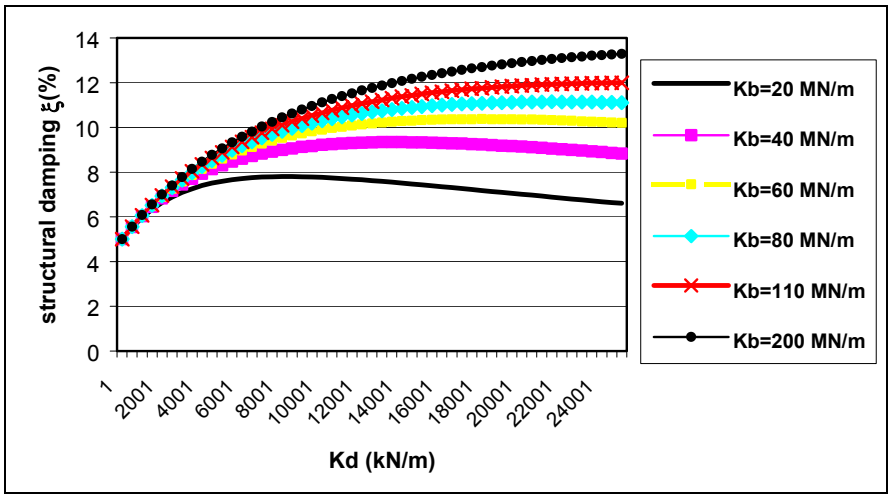

Figure 11 – Parametric curves for optimization of DBs with EVEDs.

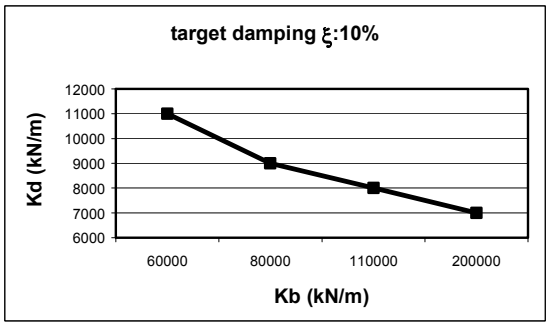

Figure 12

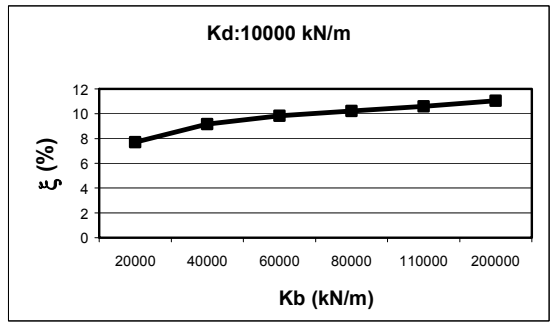

Figure 13

Optimization of viscous dampers

The viscous damper behaviour could be modelled with very good approximation utilizing a Maxwell model made comprising an elastic spring and dashpot in series. The stiffness value for the elastic spring, owing to the compressibility of the fluid, can be calculated to reach the value of maximum force at displacement equal to 5% of the maximum displacement. In any case, when the elastic stiffness value is high enough, the results of the analysis show that taking this into account is not that important in terms of the total response of the structure. Dashpot parameters are the damping constant C and the exponent α. The latter was considered equal to 0.15, because, as already stated, highly non-linear FVDs were selected for this study. As far as the braces are concerned, the same bracing system will be used during tests for EVEDs and FVDs. Thus, brace stiffness was also considered constant and equal to the stiffness of the bracing system selected as optimal in the optimization of EVEDs. Therefore, the only FVD design parameter to be optimized is the damping constant C.

This optimization was carried out through dynamic step-by-step nonlinear analyses in which the non-linearity was concentrated on the damper devices, while the RC frame was considered linear. Structural response parameters considered for the optimisation were the peak acceleration at the first and second floor, peak inter-storey drift at both levels, peak of the base shear, and the ratio between the energy dissipated by the dampers E_d and the input energy E_i.

Figures 14, 15, 16 and 17 show the parametric curves obtained through said analyses. In general, the higher C is, the better the structural response (lower floor accelerations, inter-storey drift and base shear, higher ratio E_d/E_i). However, for a high value of C, further increases do not make any marked improvement in structural response. Furthermore, when the inter-storey drift is too small, the damping efficiency becames smaller. 15 $kN(s/m)^{0.15}$ is the minimum value for C that can guarantee a peak

interstorey drift lower than 6.6 mm, i.e. lower than the 2 ‰ limit given by modern standards in order to avoid any damage to non structural elements. Conversely, the 4 ‰ limit – i.e. 13.2 mm in this case - that can guarantee from damage in the masonry infills, was never reached for all the C values considered. On the basis of these considerations, the C value selected as optimum was 15 kN(s/m)$^{0.15}$.

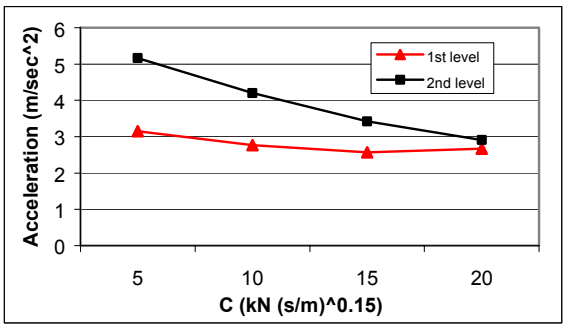

Figure 14 – Floor acceleration vs. damping constant C.

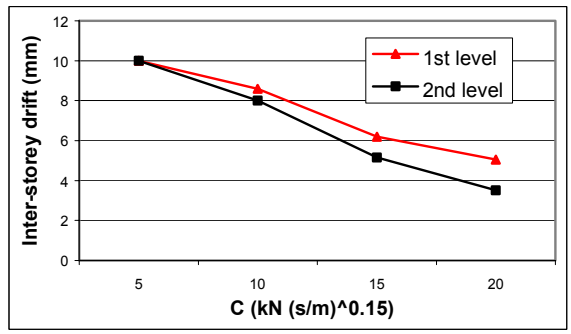

Figure 15 – Interstorey drift vs. damping constant C.

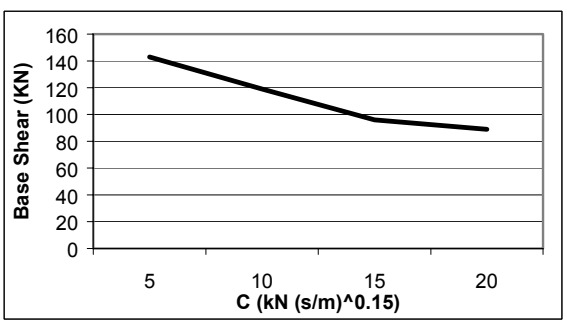

Figure 16 – Base shear vs. damping constant C.

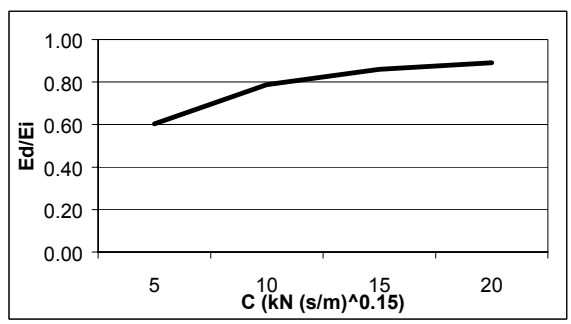

Figure 17 – Ratio of dissipated to input energy vs. damping constant C.

Optimization of buckling-restrained braces

The optimization of BRBs was carried out with the same method used for the FVDs, i.e. dynamic step-by-step nonlinear analyses. In this case, damper behaviour is approximated well by a bilinear force-displacement hysteretic loop and can thus be defined by three parameters: the elastic stiffness K_e, the post-elastic stiffness, and the yielding force F_y.

The ratio of post-elastic to elastic stiffness can vary within a rather limited range, approximately 0.01 to 0.03. Preliminary analyses and experience with other structures have shown that the structural response is not very sensitive to variations in the value of the ratio within said range. Thus it was considered to be constant and equal to 0.01 for these analyses.

The results of the parametric analysis show that structural response is quite sensitive to variations in K_e and F_y values (Figures 18, 19 and 20). As already observed in DBs with EVEDs, the higher the elastic stiffness, the better the response. Nonetheless, with high values of elastic stiffness, the improvement in the response is not as evident. The Figures show that optimum structure behaviour can be achieved using BRBs with a yielding force value between 10 and 15 kN. Taking into account technical feasibility and cost-effectiveness, a BRB with K_e=20 MN/m and F_y= 15 kN was selected as optimal.

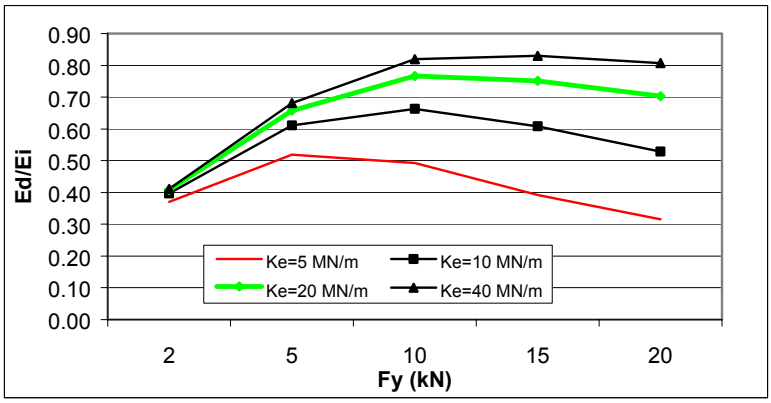

Figure 18 – Ratio of dissipated to input energy vs. yielding force.

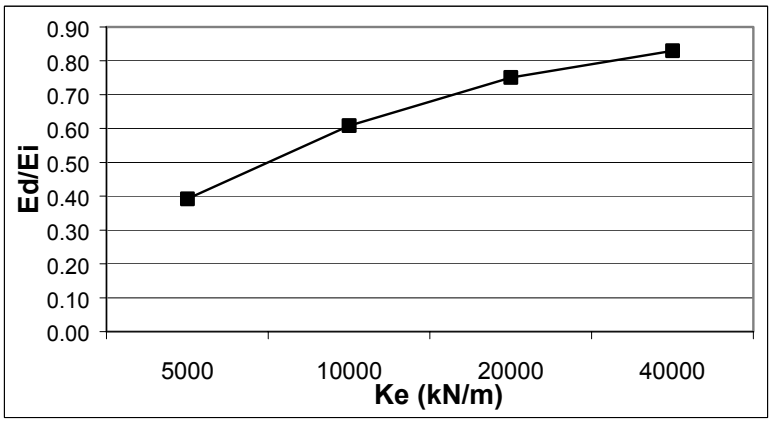

Figure 19 – Ratio of dissipated to input energy vs. elastic stiffness.

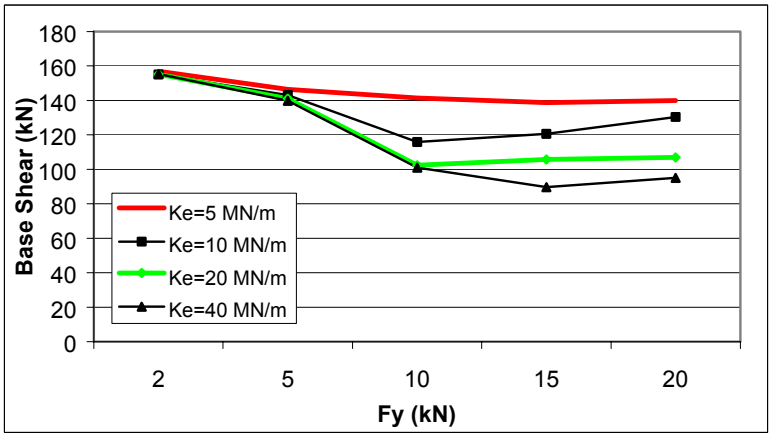

Figure 20 – Ratio of base shear vs. yielding force.

Comparison of different DB types

The responses of the frame mock-up equipped with three DB types, each with the optimal design parameters given in the previous paragraph are compared here below. In order to compare results obtained with the same type of analysis, step-by-step dynamic FEM analyses were carried out even for the case of DB with EVED, modelling the EVED force vs displacement loop with a bilinear hysteretic curve.

Figures 21, 22, 23 and 24 show a comparison of the values of dissipated energy to input energy ratio, the input energy, the peak base shear, and the peak horizontal force in the DB. The behaviour of the structure equipped with the three different DBs looks quite similar looking to the ratio Ed/Ei and the maximum base shear value. Greater differences become evident in the absolute value of the input energy and in the maximum force. The latter is lower in the DBs with higher dissipative efficiency, i.e. BRBs and FVDs. The total peak force taken by the four DBs is at least about 40 % of the base shear.

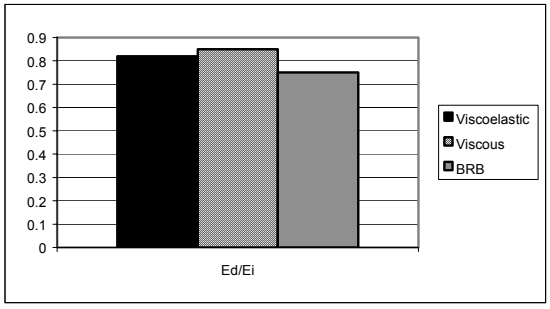

Figure 21 – Comparison of dissipated to input energy ratio.

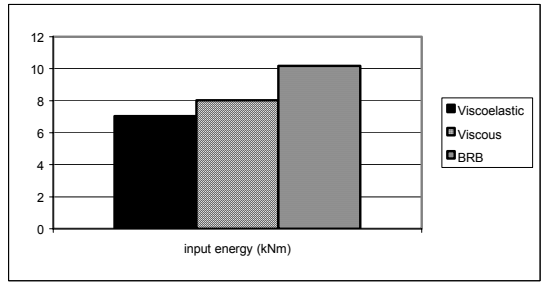

Figure 22 – Comparison of input energy .

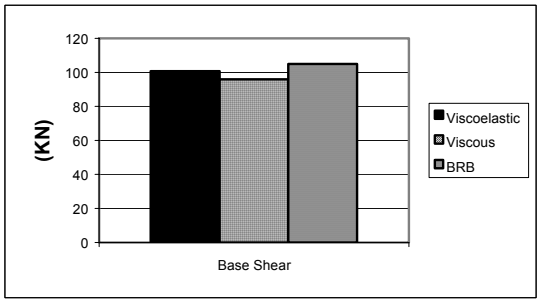

Figure 23 – Comparison of base shear.

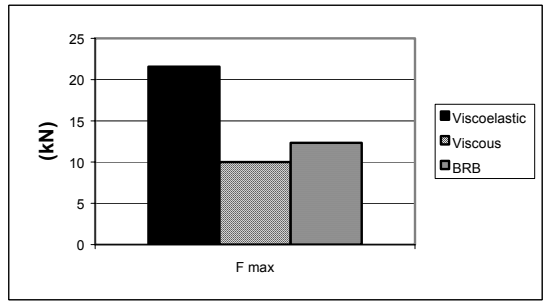

Figure 24 – Comparison of maximum horizontal force in each damper.

CONCLUSIONS

Preliminary results of the research described here show the effectiveness of dissipative bracings in improving the seismic response of RC frames built without capacity design approach. The lack of ductility of said frames is compensated by the energy dissipation in the dissipative bracings, that can also provide for a strong reduction of inter-storey drift. Consequently, DBs are useful to not only reduce damage to structural elements, but to infills as well. The comparison of different DB types shows specific advantages of each, and permits performing a cost-benefit analysis. The use of different models to analyse the behaviour of a structure equipped with DB will permit giving design engineers guidelines for the design of RC frames with DBs, through a comparison with test results. Final results of this research will make an important contribution to the future extension of the European Seismic Building Code (EC8-Part 1) to cover structures with auxiliary damping devices.

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