Seismic upgrading of an old industrial masonry building by dissipative steel roofing

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ABSTRACT: The seismic upgrading carried out on an existing masonry one-storey industrial building is dealt with in this paper. Due to the large span of the building and to the absence of intermediate walls, a steel reticulated diaphragm integrated by energy dissipation devices placed at supports of roof trusses has been introduced in order to provide a significant amount of energy dissipation. To this purpose, both oleodynamic and plastic threshold devices have been used, so to have a satisfying response of the building at all the relevant limit states taken into consideration, namely daily and seasonal thermal changes on the roof; low-to-moderate intensity earthquake; severe earthquake. A comprehensive study of the structure seismic response, carried out by means of a dynamic time-history analysis, shows the effectiveness of the adopted solution.

1 INTRODUCTION

Object of the study is a single-storey industrial masonry building. Due to the necessity of undergoing a complete transformation, comprising the partial reconstruction of some walls and the substitution of the whole roof, a particularly effective and economic system of seismic protection based on energy dissipation devices has been studied. The system is conceived in such a way to maximise the performance of the structure in all operational conditions. It is based on the diaphragm effect of a reticulated steel roof, fitted with a particular system of energy dissipation devices.

To this purpose, both viscous and plastic threshold devices have been used, together with a steel gridwork with stiffness optimised in order to maximise the device effectiveness. The construction has been analysed via a time-history procedure, which pointed out the effect of the main mechanical parameters of both devices and roof gridwork on the global behaviour of the structure (Mazzolani & Mandara 2004).

The building under consideration consists of a tuff masonry regular construction completed by a top concrete tie-beam. Relevant $L_1 \times L_2$ sizes are $58 \times 32$ m in plant, with an internal clearance of 6.5 m (Figure 1). A particularity of the building is to have the longitudinal

Figure 1. Overall view of the structure during and after the completion of the works performed by the R.C.M. Constructions S.r.l.
walls of different height (6.5 m and 8 m), in consequence of the slope of foundation soil. In addition, all the walls have several openings, sometimes of large sizes, that reduce their in-plane stiffness. A special trussed structure, placed at the level of the lower rafter of roof trusses, is fitted in order to offer adequate stiffness, so as to behave as a diaphragm under seismic actions. Such roof structure has been provided with a system of seismic protection consisting of oleodynamic viscous dampers (Figures 2 and 3a), used in combination with controlled yielding steel devices (Figures 3b and 4). The viscous device consists of a conventional double-chamber oleodynamic piston, without by-pass valve (Figure 3a). The plastic threshold device, on the contrary, is obtained from a steel round bar and is characterised by the typical “handle-bar” form, connected to both roof trusses and lower masonry wall (Figures 3b and 4). Its energy dissipation takes place due to alternate plasticity in bending. The main mechanical features of both devices are summarised in Figure 3.

Two objectives can be achieved through this intervention, both of them very important in the view of the optimisation of the structural behaviour at serviceability and ultimate state limit, respectively. Thanks to the use of oleodynamic viscous devices, in fact, conditions of provisional restraint are obtained, allowing
the device to behave as a perfect slider under load applied slowly (e.g. thermal changes), but opposing a remarkable reaction and energy dissipation under quickly applied load (e.g. seismic actions). Such aspect optimises the behaviour of the roof under seismic actions (quick actions), for which the system demands the maximum redundancy, by eliminating, at the same time, thermal co-actions in the roof. A further amount of dissipated energy is eventually given by hysteretic plastic threshold devices, that are involved under very strong earthquakes, only, with significant beneficial effects on the behaviour of the whole construction.

Both choice and set-up of this combined system of seismic protection has been made on the basis of an accurate analysis of the dynamic behaviour of the construction, described in the following paragraph. The analysis led to define the optimal features of the devices, as well as their quite unusual configuration. In particular, the results of numerical analysis have brought to the adoption of the aforementioned “mixed” system of protection, based on the combined effect of different types of devices.

2 NUMERICAL ANALYSIS

The structure has been studied through dynamic analysis in the time domain by means of the F.E.M. code SAP 2000NL (CSI 2000), that provides a good interpretation of the behaviour of structures equipped with both viscous and plastic threshold devices (NLLINK). The F.E.M. model of the building is shown in Figure 5. The structure has been reproduced in the least details, including the openings in the building walls and the whole steel skeleton of the roof. Three historical seismic inputs have been assumed, considered acting in the direction transverse to the building, namely Calitri (1990), El Centro (1940) and Taiwan (1999), scaled to 0.28 g and 0.40 g. Based on the measured value of the device damping exponent ($\alpha \approx 0.9$) (Figure 3), linear dampers have been considered in the analysis, for which $\alpha = 1$.

The behaviour of materials has been assumed linear elastic ($E = 2000 \, \text{N/mm}^2$, $\gamma = 20000 \, \text{kN/m}^3$) with a 3% equivalent modal damping. As shown in previous studies (Mandara & Mazzolani 2001a, b) such assumption does not lessen the reliability of results achieved on the structural response. On the contrary, it puts into evidence the beneficial effect of dampers on the global performance of the structure, mainly resulting in a reduction of displacement under seismic action and, hence, in an increase of the level of the action that can be absorbed by the construction in purely elastic range. To this purpose, it has been already observed (Mazzolani et al. 2003a, b, c) that, with an appropriate sizing of devices, the behaviour of the structure is such to remain entirely in elastic range under earthquakes of intensity equal to or higher than the one specified in seismic codes.

In previous papers (Mandara & Mazzolani 2001a, b; Mazzolani & Mandara 1994; Mazzolani & Mandara 2002a, b; Mazzolani et al. 2003a), the authors underlined the effectiveness of viscous devices when placed at the base of roof trusses in long-bay single storey buildings ($L_1/L_2 > 2+3$), in combination with a rigid top diaphragm. In such conditions, for an earthquake acting in direction transverse to the building, the use of energy dissipation devices can yield a reduction of the structural response of both longitudinal and transverse walls. This result can be further improved by means of additional plastic threshold devices, placed in different points of the structure, for example between roof and transverse walls, in order to limit the transmission of actions on walls to a given value, increasing dissipated energy at the same time.

In the case under consideration the $L_1/L_2$ aspect ratio is lower than 2, exactly it is equal to 1.81. In addition, due to the large span of the shed, achieving
a truly rigid roof diaphragm would be really difficult, without a very complex and expensive design of the structure. It would result therefore difficult the optimal exploitation of the system and the achievement of the maximum performances. For this reason an alternative configuration of the devices has been adopted: instead of viscous devices placed on both long walls and plastic threshold devices on short walls, hysteretic controlled-yielding devices illustrated in the previous paragraph have been put on the wall 1, only (the one of greatest height), leaving the viscous dampers on wall 2 (Figure 5). Such decision has been taken after a comparative evaluation of the seismic behaviour of the construction in both configurations.

The corresponding comparison is represented in Figures 6 and 7 as a function of the damper viscous constant $c$, for hysteretic devices with a plastic threshold $F_y = 100$ kN. In general, it can be observed that the additional use of hysteretic devices makes the value of top wall displacements practically independent of the viscous constant $c$. In particular, for $c$ values higher than 1000 kN/ms$^{-1}$ (this value guarantees a satisfactory response reduction in all walls) the results achievable with the “mixed” solution are better in comparison with the purely “viscous” solution. Values of $c$ lower than 1000 kN/ms$^{-1}$, even though favourable to walls 2, 3 and 4, could involve excess of displacements on the top of wall 1 (Figure 6a), which is more vulnerable due to its greater height.

The choice of the optimal plastic threshold $F_y$ has been made examining the response of the structure for different values of $c$, for PGA = 0.28 g (Figure 8). It can be observed that, in all cases, the value $F_y = 100$ kN represents a transition threshold between two different types of behaviour, characterised by a
very different response in terms of displacement. The existence of such marked transition is also visible in Figure 9, showing the case relative to PGA = 0.40 g in comparison with PGA = 0.28 g. The use of plastic threshold device turns out to be useful also in the limitation of the maximum force transmitted by the viscous devices. Figures 10 and 11 show the values of such force as a function of both viscous constant and plastic threshold of devices under the three considered recordings scaled at PGA = 0.28 g and 0.40 g. The benefit is evident for values of c higher than 1000 kN/ms\(^{-1}\). Also, for \(F_y \leq 100\) kN the maximum force in the viscous devices tends to be practically independent of the PGA (Figure 11a).

Forces acting in viscous devices are in close relationship with the diaphragm stiffness. Figure 10b, relative to a diaphragm with double the stiffness of the real one, shows that in practice also the force magnitude in the devices is doubled, which inevitably results in a corresponding increase of the force acting on underlying masonry walls. In such conditions, actions transmitted to the transverse walls are not significantly reduced. Figure 12 shows the displacement values in such walls as a function of the plastic threshold \(F_y\) for c = 10000 kN/ms\(^{-1}\) for a diaphragm of infinite rigidity.

Such considerations emphasise the advantage involved by the additional use of plastic threshold devices, resulting in a significant reduction of forces transmitted by viscous dampers (Figure 10a,b), as well as in making such force less dependent on the magnitude of the seismic action (Figure 11a). In Figure 11b the time history of the force in the viscous devices is
shown for two of the examined cases. Eventually, both “viscous only” and “viscous + plastic” solutions are compared to each other in Figure 13, where the values of top wall displacements are represented in the form of histogram, against the solution in which a fixed
connection between roof and walls is adopted. Average values involved by the three considered earthquake recordings have been allowed for.

The comparison shows, in general, the validity of such strategies of seismic protection, and particularly the advantage of the “mixed” provision. Compared with the fixed connection, in fact, an average reduction of top wall displacements around 30% is obtainable on walls 2, 3 and 4. For wall 1 only, that is where plastic threshold devices are installed, the “viscous only” solution provides slightly better results for the represented case (PGA = 0.28 g).

3 REMARKS ON DEVICE SIZING

From the practical point of view it is necessary to define the damper characteristics, in terms of both viscosity and mechanical resistance. Due to the small aspect ratio of the building ($L_1/L_2 \approx 1.8$), it is not possible to find an optimal viscosity value for the dampers which minimises the structural response. This is also evident when using viscous dampers only (Figures 6 and 7). On the other hand, the use of additional plastic threshold devices makes the structural response even less influenced by the viscous properties of devices, as already shown by Mandara & Mazzolani 2001a,b. Concerning the examined case, graphs point out that values of $c$ in the order or higher than $1000 \div 10000 \text{ kN/m} \text{s}^{-1}$ involve a response reduction of longitudinal walls whose magnitude is comparable to the one that can be obtained with a fixed connection between roof and vertical walls (see Figures 6 and 7 for $c = 1000000 \text{ kN/m} \text{s}^{-1}$). A value of $c = 15000 \text{ kN/ms}^{-1}$ has been chosen in the end, in order to allow for a possible aging of the fluid (siliconic oil) contained in the device, which could involve a consequent reduction of its viscosity. Higher values of the viscous constant would involve excess of force transmitted across the device and, hence, to the masonry structure (Figures 10 and 11).

The choice of the device maximum capacity has to be done according to the magnitude of forces which can arise in the device during the most severe design earthquake. A device with inadequate capability could drop its effectiveness because of possible fluid leakages due to an excess of internal pressure, or could fail due to the attainment of its mechanical limits. In such circumstances the connection between walls and roof structure would be drastically impaired. The devices adopted in this case has a operational pressure of 35 MPa, with the possibility to bear up to 70 MPa without fluid losses through gaskets and rings. With the siliconic fluid which they are filled with, having a kinematic viscosity of 100 cSt, such limits correspond to transmitted forces of 140 and 280 kN, respectively. The particular mechanical structure of the device, entirely made of stainless steel to avoid problems of corrosion, is of course sized to withstand even higher loads.

The above values result in good relationship with the data obtained from numerical analysis, even though, in some conditions, the safety margin with respect to earthquake time history data would seem, at a first glance, rather poor in comparison with the maximum allowable design pressure of the devices (200 kN). From the examination of Figures 10a and 11a, in fact, it is possible to observe that, also adopting additional
plastic threshold devices with $F_y = 100$ kN, the maximum values of force acting in viscous dampers is in the order of 250 kN, for PGA ranging between 0.28 and 0.40 g.

Nevertheless, the following considerations can be made, that justify the choice of the adopted devices. First, forces transmitted by dampers have been evaluated under the assumption of purely elastic behaviour of masonry, which leads to overestimate the actions in the devices.

Also, values in Figure 10 are instantaneous peak values, attained a very limited number of times during the seismic event. This can be observed in Figure 11b, in which the time-histories of the force in the dampers are represented for Calitri and Taiwan earthquakes, both scaled to PGA = 0.28 g. It is well known that the strength of such devices under impulsive loads are significantly greater than under statically sustained actions. Furthermore, one should consider that values in Figure 10 refer to the most heavily loaded device, that is the one placed at midspan of wall 2, all the remaining devices being, as a matter of fact, less stressed.

All these considerations, together with economic reasons, have led to the sizing of viscous devices. It has to be observed that using a diaphragm of greater rigidity would have increased the magnitude of force in the dampers, as shown in the Figure 10b for a diaphragm of double the stiffness of the real one, without a remarkable effect on the value of transverse wall displacements (Figure 12).

As far as the plastic threshold value of hysteretic devices is concerned, an interesting comparison, even though approximate, can be made on the basis of the equivalent static analysis procedures set out in seismic codes (e.g. the Italian Seismic Code). For a 2nd category seismic zone, the equivalent static forces in the devices is given by $F_y = kW = \beta(s-2)/100 \times Y_{n} \times V_m$, in which $k$ is the coefficient of seismic intensity, $W$ is the "seismic weight" of the masonry panel, $s$ the seismic degree (equal to 9 for 2nd category zones), $\beta$ is the behavioural coefficient for masonry buildings (assumed equal to 4) and $V_m$ is the masonry volume pertaining to the single device. Considering a 8 m high masonry wall, one may obtain $F_y = 106$ kN, which results in a good agreement with the value suggested by numerical analysis (100 kN) for PGA = 0.28 g ± 0.40 g. Such values of PGA do comply with the spectral acceleration value for masonry buildings ($\beta = 4$) located in 2nd category seismic zones.

The plastic threshold devices, obtained from a solid $\varnothing 40$ steel Fe430 round bar, are designed in such a way to yield under a load of 100 kN, ensuring a good ductility under cyclic actions. Adopting a steel with $f_y = 350$ N/mm$^2$ the fully plastic moment $M_p$ is equal to 3738 kN mm. Considering that the rod works in two sections and assuming clamped end restraint conditions, the length necessary to obtain such value of the bending moment is about 150 mm, neglecting the effect of shear (see Figure 4 and relevant scheme in Figure 3b). A special system of connection by means of bolted plates, makes for an easy substitution of devices damaged by earthquake.

4 CONCLUSIONS

The study case described herein represents an example of profitable use of passive control systems for the improvement of the seismic performance of an existing building. The intervention, carried out on an old industrial masonry shed, has concerned the roof structure only, which has been equipped with a combined system of energy dissipation devices. As widely demonstrated herein and elsewhere, the effect of energy dissipation due to dampers is basically to reduce the extent of displacements in the structure under dynamic input. This means that the application of dissipative devices produces an increase of the maximum earthquake intensity which can be resisted by the structure without damage. Numerical analyses have shown that such a "mix" results in a reduction in terms of top wall displacement in average of 30% compared with values obtainable with a fixed roof-to-wall connection. In this way, the diaphragm effect has been conveniently exploited, minimising the effects involved by its inherent deformability. Also, the use of oleodynamic devices has allowed to eliminate the effects of daily and seasonal thermal changes, which are not negligible due to the large span of the building. From the economic point of view, the adoption of such "mixed" solution resulted convenient in comparison with conventional strengthening operations and with the "viscous only" solution. In addition, it is completely reversible, and this could represent an useful issue for such operations when applied to historical or monumental constructions.

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