VISCOUS DISSIPATIVE, DUCTILITY-BASED AND ELASTIC BRACING DESIGN SOLUTIONS FOR AN INDOOR SPORTS STEEL BUILDING

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Received: 21 December 2011; Revised: 28 June 2012; Accepted: 31 July 2012

ABSTRACT: Three bracing solutions are developed for an indoor sports facility representative of the most recent architectural design trends for recreational and commercial steel buildings. A viscous-dissipative bracing system incorporating pressurized fluid viscous spring-dampers is assumed as anti-seismic technology for the first solution. Thanks to the protective capacities of this technology, non-structural and structural operational performance levels are pursued for the building up to the maximum considered design earthquake level. A traditional concentric X-shaped configuration is selected for the second and third solution, which differ from each other in the design strategy adopted. In the first case, a ductility-based solution is chosen, by adopting a basic behaviour factor equal to 4, reduced by 20% to take into account the structural irregularity caused by the eccentric position of the intermediate floor of the building. In the second case, the same performance objectives as originally formulated for the viscous-dissipative design hypothesis are assumed, fixing the behaviour factor at 1. The resulting dimensions of the members, and the total weight and cost of the steel structure are compared for the three layouts. A substantially higher seismic performance at all normative design levels at a comparable cost; and a cut in the cost by about 50%, with an improved look of the structure due to the remarkably greater slenderness of the constituting members, come out for the viscous-dissipative bracing design as compared to the ductility-based and elastic X-bracing solutions, respectively.

Keywords: Steel structures, Seismic protection, Fluid viscous dissipative bracing, Damping, Concentric bracing, Ductility-based design, Elastic design

1. INTRODUCTION

The most recent trends in the design of steel buildings favour transparency, lightness and aerial effects, which are obtained by including large glazed façades, by increasing the free spans of floors and roofs, and by implementing innovative architectural shapes and finishes. These emerging trends compel structural designers to reduce further the architectural impact of load-bearing systems, while at the same time meeting the high performance levels required by the latest generation of Technical and Seismic Standards on steel construction.

A satisfactory balance between the issues of high structural performance and limited architectural impact is a very demanding challenge for structural engineers, especially for buildings situated in medium-to-high seismicity zones. The normative solution to this problem was traditionally represented by the design of dissipative structures, i.e. structures that are able to dissipate energy by means of their ductile hysteretic (plastic) behaviour. According to this ductility-based design approach, the seismic forces for the verifications at the ultimate limit states (i.e. Life Safety — LS and Collapse Prevention — CP performance levels) are computed by scaling relevant pseudo-acceleration response spectra by behaviour factors proportional to the ductility resources of the structural systems. This allows keeping the size of structural members within acceptably small limits, while at the same time planning the plasticization, and thus significant damage, of their critical (dissipative) zones under the highest earthquake levels considered in the design process. This implies that considerable repair costs after medium-intensity seismic events, and in many cases the complete demolition and rebuilding of the structure after severe seismic events, must be accepted.

Over the past two decades, this traditional approach has been revised for strategic buildings, where appreciable damage even at the highest design earthquake levels must be avoided. This class of buildings includes hospitals, barracks, fire-stations, civil protection headquarters, airports, railway stations, industrial plants, broadcasting facilities, etc. Recently, the concept of "strategic" has also been extended to various types of public and private buildings that, whatever their specific function, are likely to play an important role in the immediate post-earthquake emergency phases, e.g. shelter to injured people, and temporary headquarters for the coordination of rescue, assistance and recognition activities. These buildings, which include schools, indoor sports facilities, gyms, exhibition halls, etc., continue to be designed for their normative nominal structural life and their specific use (with coefficients of use greater than the ones assigned to ordinary buildings, but lower than the ones of strategic buildings), but with a shift towards higher performance levels, as compared to the seismic response capacities required by the traditional ductility-based design approaches.

New solutions to the combined issues of enhanced seismic performance and reduced dimensional impact for contemporary steel buildings are now offered by advanced seismic protection technologies. Among these technologies, viscous dissipative bracing (VDB) systems — which dissipate seismic energy by means of viscous dampers mounted on the supporting braces, rather than by plasticization of the constituting structural members — generally provide the most viable solutions for steel buildings, as they visually resemble the bracing elements traditionally incorporated in steel structural skeletons to absorb seismic and wind loads. This allows keeping the traditional setup of steel structures substantially unchanged, while reducing, for a predetermined set of performance objectives, the member sections and/or the number of locations where braces are placed, as a consequence of a remarkable reduction in seismic effects produced by the dampers.

Within the wide class of VDB technologies [1-3], a special system incorporating pressurized fluid viscous (FV) devices has been studied for several years by the first two authors of this paper, also in the frame of international research projects. In particular, numerical and analytical modelling; experimental characterization and verification; definition of design procedures; and technical implementation of the protective system have been carried out within these activities [4-8]. Pilot applications have also been developed, with special reference to the seismic retrofit of steel school buildings [9-10].

The study of a novel application, concerning a steel fitness and indoor sports facility in Italy, well representative of the most recent architectural design trends, is presented in this paper. The building constitutes a detailed prototype case study developed within a National Research Project funded by the Italian Department of Civil Protection and dedicated to the advanced seismic protection of structures, with the aim of establishing a benchmark for scholars and designers interested in similar studies and of implementing real applications in the near future. The building is characterized by a single-span roof, open space interiors, continuous floor-to-roof glazed façades, and a perimeter arcade. The architectural design also imposes the smallest possible structural member sizes and the introduction of the lowest possible number of vertical bracing alignments, in order to limit the visual obstruction of the glass façades.

The seismic design hypothesis based on the installation of the VDB system is compared to two X-shaped (concentric) bracing solutions, carried out by following the standard normative ductility-based approach (DBXB hypothesis), and by pursuing an elastic response up to the maximum input earthquake level (EXB hypothesis), respectively. The DBXB solution makes a basis for comparison in terms of performance capacities starting from similar structural member sizes and costs, as determined by a reduction in seismic effects owed to the protective action of the FV dampers, for the VDB design, and by adopting the normative value of the behaviour factor, for the traditional ductility-based design. On the other hand, the EXB hypothesis is developed to compare sizes and costs with the VDB solution, by postulating identical structural and non-structural performance objectives and by pursuing them without scaling dynamic response, i.e. by assuming a behaviour factor equal to 1, in both cases.

The general characteristics of the case study building, the earthquake and performance levels assumed, their relevant limitations, and a synthesis and comparison of sizes, dynamic response and costs of the three design solutions, are presented in the following sections.

2. GENERAL CHARACTERISTICS OF THE CASE STUDY BUILDING

The external dimensions of the building are 60 m × 30 m in plan, with a height of 18 m. The arcade is 3 m-wide, which determines net internal dimensions of 54 m × 24 m. An intermediate floor is situated at a height of 6 m on one side, covering a smaller area in plan, equal to 20 m × 24 m. The structural plans of the roof and the floor are shown in Figures 1 and 2. The elevation views in the longitudinal (parallel to y axis in plan) and transversal directions are drawn in Figures 3 and 4. The A-A cross section traced out in Figures 1 and 2 is displayed in Figure 5. Two architectural renderings of the whole building, and three views of the interiors, are reproduced in Figures 6 and 7, respectively. These images show how the structure would look with the X-bracing solutions. Views of a typical internal joint of the roof trusses, identical to the corresponding joint of the floor trusses, and a truss/column/longitudinal beam/arcade beam connection on the roof, are displayed in Figure 8. The latter connection too is identical to the one at the floor level, except for the absence of the arcade beam joining the top of the arcade columns.

The shapes and dimensions of the beams (HEA 180 Italian profiles — roof, HEB 200 — floor), trusses (all tubular sections, sized 244.5 mm — diameter × 8 mm — thickness, for the upper and lower horizontal profiles, and 139.7 mm × 8 mm for the diagonal profiles — roof; 244.5 mm × 10 mm for the upper and lower horizontal profiles, and 139.7 mm × 10 mm for the diagonal profiles — floor), horizontal braces (168.3 mm × 10 mm — roof, 139.7 mm × 12.5 mm — floor), and stairs (columns in 219.1 mm × 5 mm tubular sections, beams in HEA 300 profiles), are identical for the three structural solutions. Indeed, their design is essentially governed by gravity loads, equal to 2.8 kN/m^2 (roof), 3.6 kN/m^2 (floor) and 2 kN/m^2 (stairs) — for dead loads, and 1.1 kN/m^2 (roof) and 5 kN/m^2 (floor and stairs) — for live loads. The arcade columns are not designed for seismic action, which is completely absorbed by the bracing systems, and thus their section (219.1 mm × 5 mm tubular profile) is equal for the three design hypotheses too. Second order effects are computed in the design of these slender columns.

The glazed façades are supported by vertical aluminium trusses, whose base is fixed to the foundation and whose top is hinged to the perimeter beams of the roof structure. The glass panes have 4 fixing points, and are joined each other by “spider”-type connections.

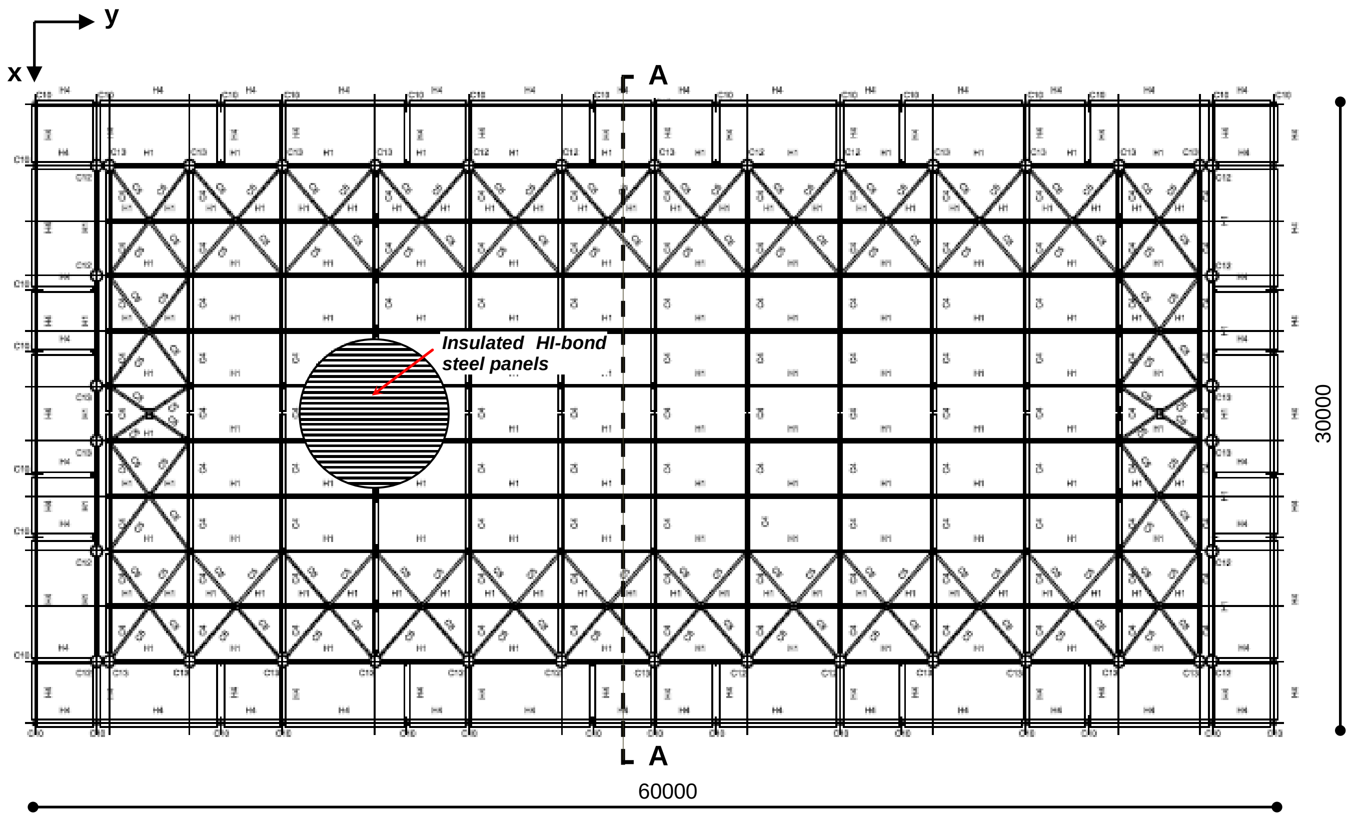

Figure 1. Structural Plan of the Roof

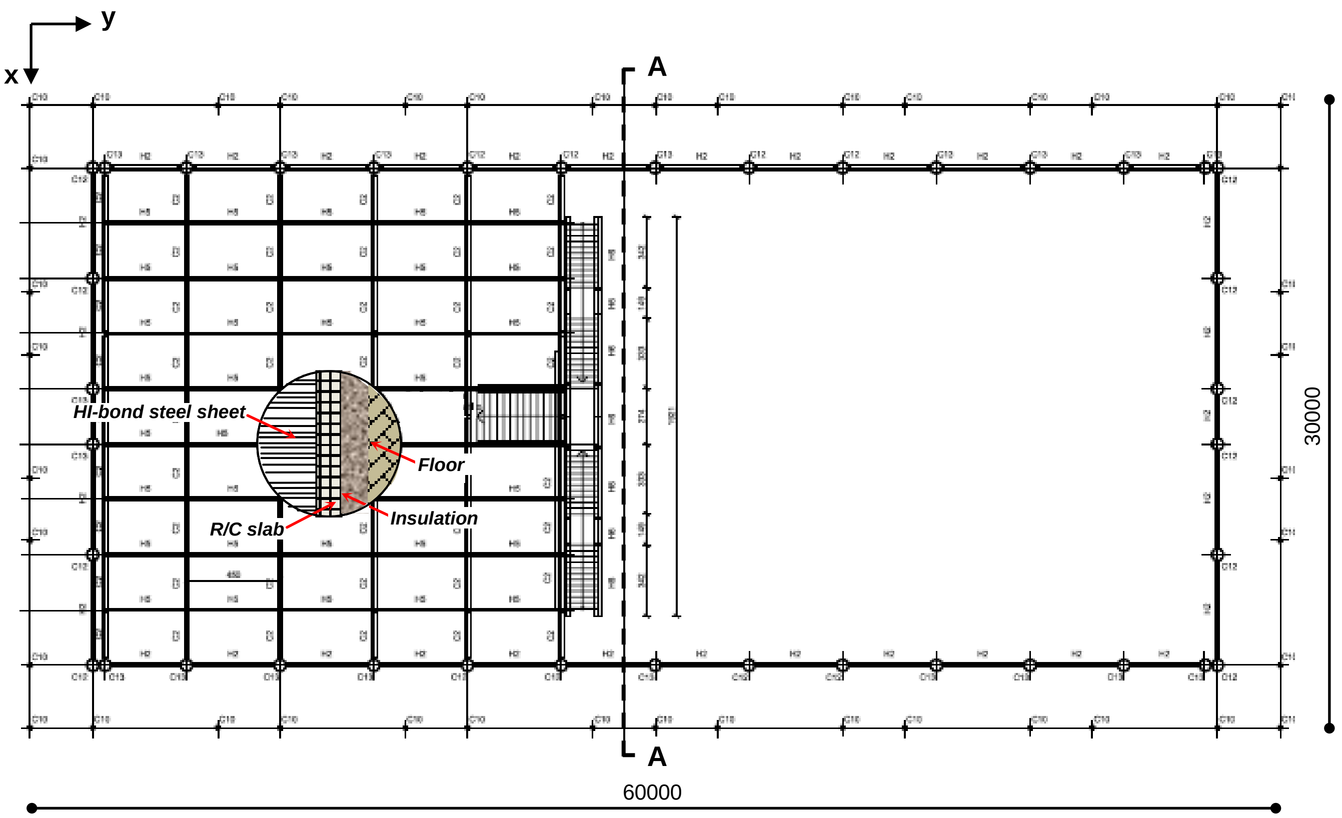

Figure 2. Structural Plan of the Floor

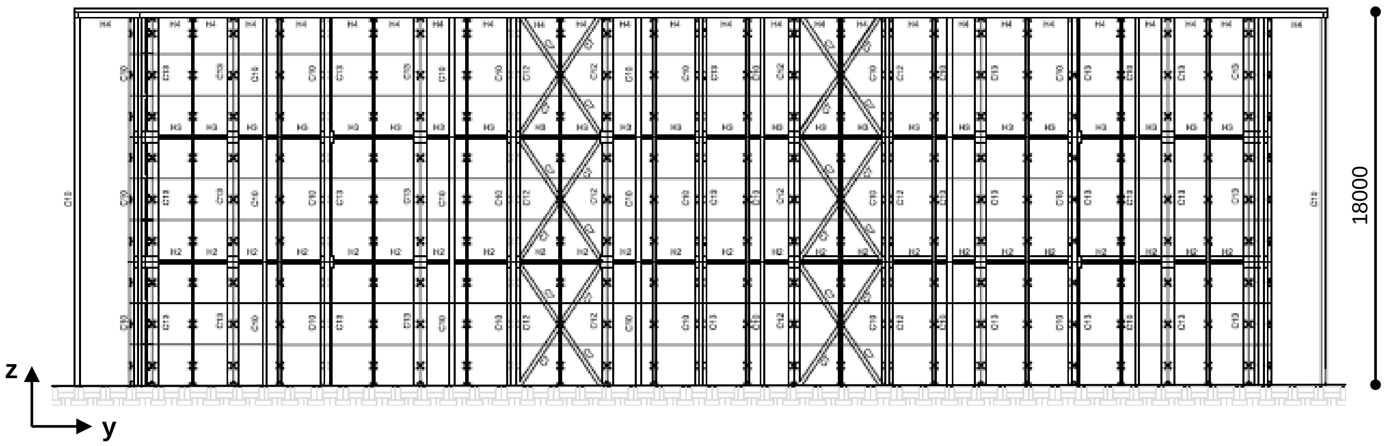

Figure 3. Longitudinal Elevation View (X-bracing)

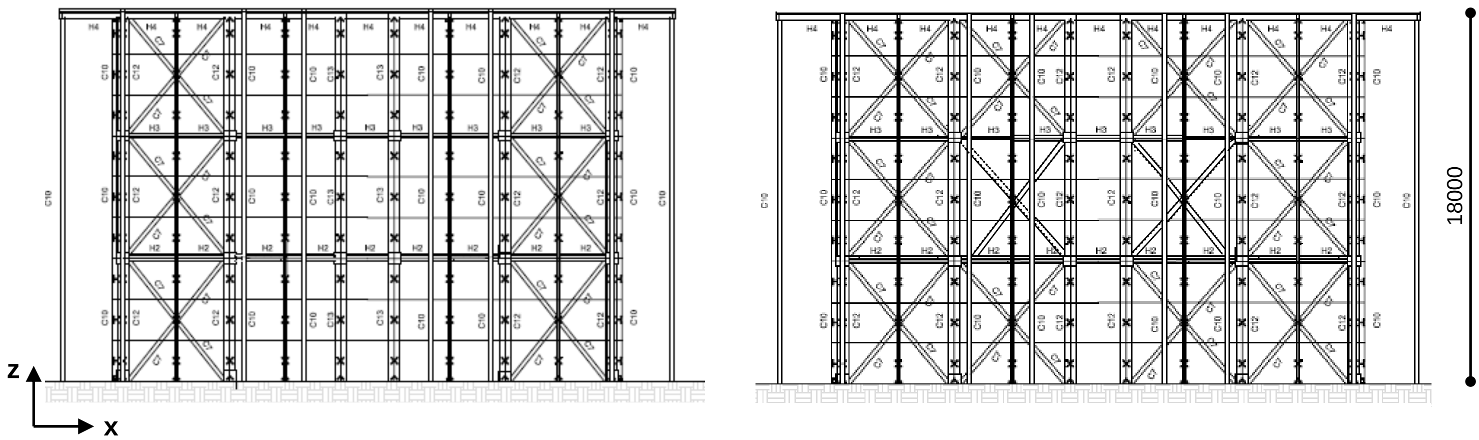

Figure 4. Transversal Elevation Views (X-bracing) – Left: Floor Side; Right: Opposite Side

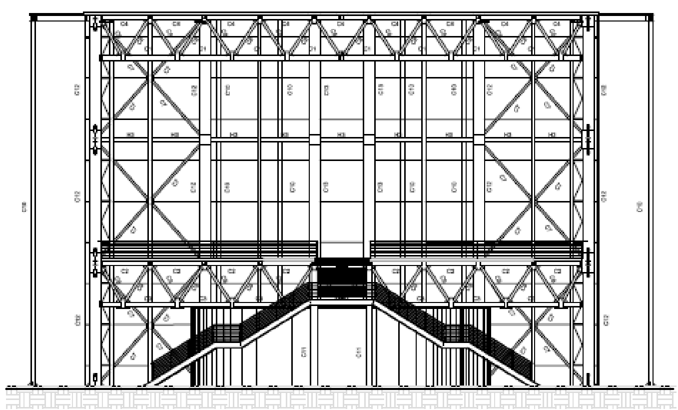

Figure 5. A-A Cross Section in Figures 1 and 2 (X-bracing)

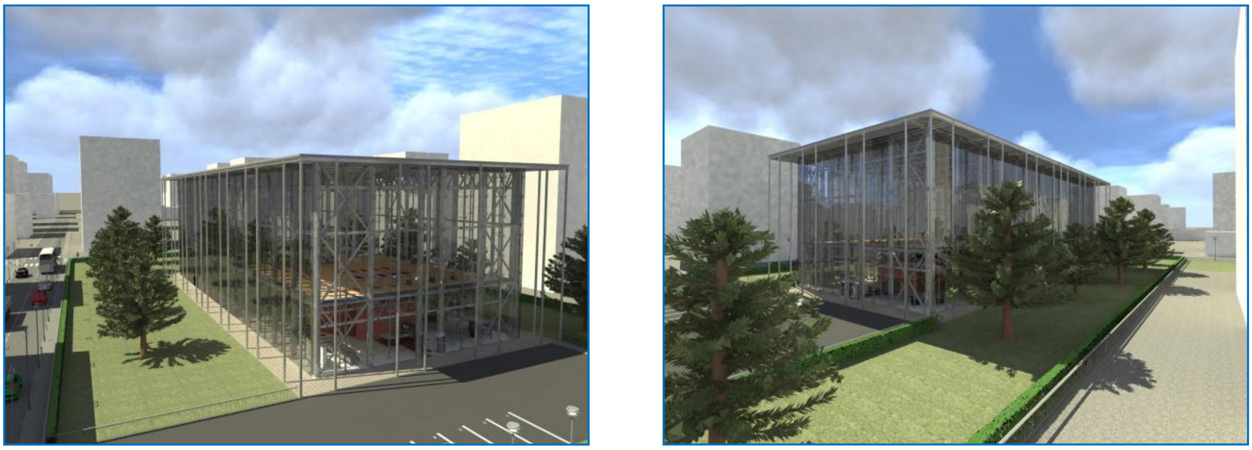

Figure 6. External Renderings of the Building on the Floor Side

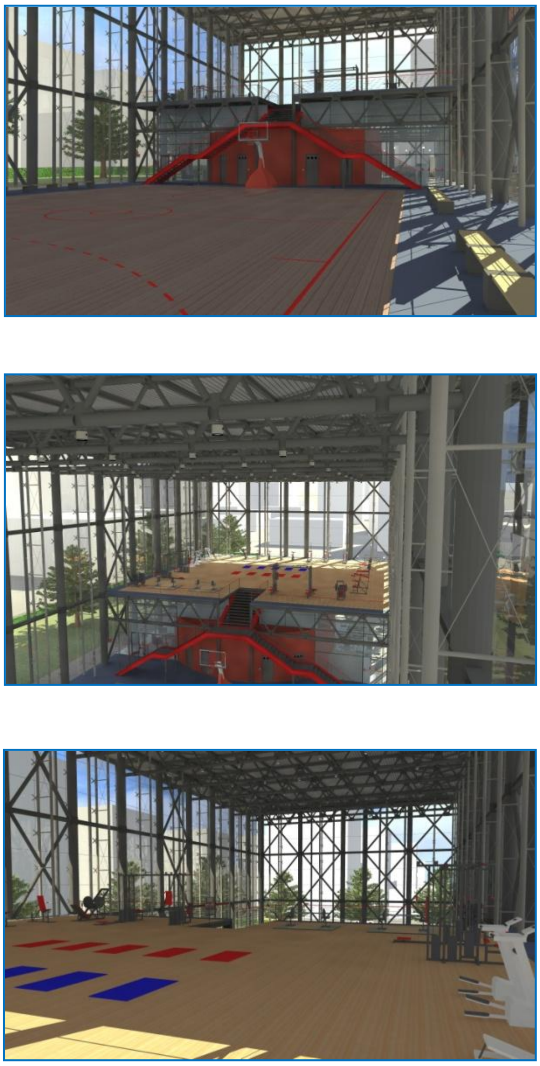

Figure 7. Internal Renderings of the Building

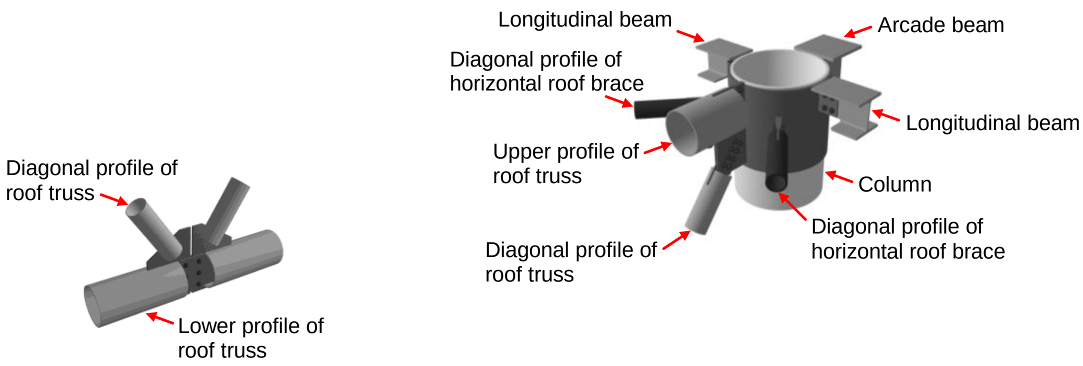

Figure 8. Views of an Intermediate Joint of Roof Trusses, and
a Truss-column-beams Connection at the Roof Level

3. DESIGN EARTHQUAKE LEVELS

The building is designed to be located in the town of Udine, Friuli region, Italy, to which a medium-high seismicity level, according to the site-dependent seismic classification of the new Italian Technical Standards [11], is assigned. The pseudo-acceleration elastic response spectra scaled at the amplitudes of the frequent design earthquake (FDE, with a 81% probability of being exceeded over the reference time period V_R, calculated as: $V_R=V_N \cdot C_U$, where V_N, C_U mean nominal life, and coefficient of use of the building, respectively); the serviceability design earthquake (SDE, with a 50%/V_R probability); the basic design earthquake (BDE, with a 10%/V_R probability); and the maximum considered earthquake (MCE, with a 5%/V_R probability), are displayed in Figure 9. The spectra are computed for the following parameters: V_N=50 years, C_U=1.5 (class of use III, corresponding to buildings with potential crowds), topographic category T1 (flat surface of the building site), and B-type soil (deposits of very thick sand, gravel, or very stiff clay, several dozens of meters thick). As anticipated in the introduction, the design analyses were developed in the DBXB design hypothesis by scaling the spectra referred to the BDE and the MCE by a behaviour factor $q=4\times0.8=3.2$, where a 0.8 penalty coefficient is assumed so as to take into account the structural irregularity caused by the eccentric position of the floor. The same spectra were not scaled (q=1) for the EXB and VDB solutions. The FDE and SDE-related elastic response spectra were mutually assumed for the three design hypotheses. The verifications of the structural members were carried out according to the very similar requirements of the Italian Standards [11] and Eurocode 3 [12].

By applying the mutual prescriptions of Standards [11] and Eurocode 1 [13], the wind action for the site of the building (altitude of 113 m above ground, and terrain category IV, corresponding to an area where at least 15% of the surface is covered with buildings whose average height exceeds 15 m) is quantified by a peak velocity pressure $q_p=q_b \cdot c_e(z)=0.69$ kN/m^2, where $q_b=0.39$ kN/m^2 is the basic velocity pressure, calculated as follows: $q_b=\rho \cdot v_b^2$, with ρ=air density=1.25 kg/m^3 and v_b=basic wind velocity=25 m/s, and $c_e(z)=1.76$ is the exposure factor, computed for the z height above ground of the building, equal to 18 m. Based on the geometric and structural characteristics of the edifice, the size factor c_s and the dynamic factor c_d (and thus their product, i.e. the structural factor $c_s \cdot c_d$, as defined in [13]) are fixed at 1. The most demanding combination of wind forces — represented by the external pressure-related force, $F_{w,e}$, the internal pressure-related force, $F_{w,i}$, and the friction-related force associated to the wind action parallel to the external surfaces, F_{fr} — is obtained for the x direction in plan. This combination is determined by the following parameters:

external pressure coefficient c_{pe}=0.8; internal pressure coefficient c_{pi}=-0.5, which generates suction forces on the façades with the same sign as the external pressure-induced forces; and friction coefficient c_{fr}=0.4. This gives rise to a total wind force on the building (equal to the maximum wind-induced base shear) F_w=998 kN, which is lower than the minimum earthquake-induced base shear, equal to 1140 kN, computed for the lowest earthquake level (FDE) and the VDB solution (greater minimum earthquake-induced base shear values are obtained for the DBXB and EXB design hypotheses). Therefore, the horizontal load-related design of the bracing systems is governed by seismic action.

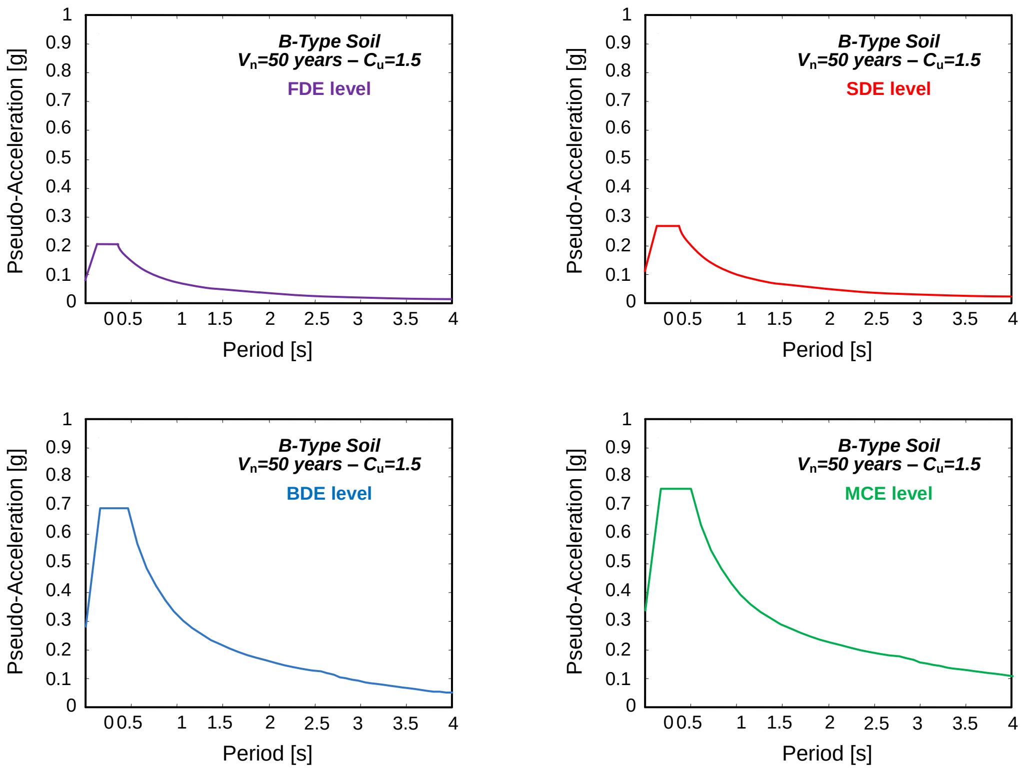

Figure 9. FDE, SDE, BDE and MCE-scaled Elastic Response Spectra

4. SEISMIC PERFORMANCE LEVELS AND LIMITATIONS

The response limits related to the highest seismic performance levels (Operational — OP, and Immediate Occupancy — IO) are generally calibrated with the presence of damageable non-structural elements and finishes directly interacting with the structural skeleton or, in any case, of displacement-sensitive components. The 4 point-fixed glass panes wrapping the building are among the most sensitive non-structural elements to in-plane and out-of-plane displacements. Manufacturers of glass panes installed with spider-type or similar joints suggest that, in order to avoid local damage to glass and connectors, the maximum relative in-plane and out-of-plane horizontal displacements between the lower fitting points of the two upper connectors and the upper fitting points of the two lower connectors be constrained within 5‰ of their mutual distance D (i.e. 9.3 mm for the 2 m high panes considered in this design, where D is equal to 1.85 m). Therefore, the 0.005D threshold can be fixed as the relative displacement limit associated to the non-structural OP performance level, $rd_{NS,OP}$, which also allows matching the slightly less prudential value suggested by the recently released Technical Recommendations on glass elements [14], equal to D/175 (0.0057D) for any type of 4-edge supported pane. Beyond this level of deformation, a first series of hairline cracks appears, normally confined at glass-connector interfaces, and reparable without interrupting the use of the building until the relative displacement of the pane approximates 0.01*D*. Therefore, this value can be adopted as the limit for the non-structural IO performance level, $rd_{NS,IO}$. Clearly visible and irreparable cracks are observed on the surface of the panes in the 0.01–0.015*D* range of relative displacement, and cracks extended to the entire surface, although at no risk of detachment, in the 0.015–0.02*D* range, with 0.02*D* marking the acceptable boundary for the LS performance level, $rd_{NS,LS}$. The glass panes become fallout-prone for relative displacements around 0.03*D*, which can be fixed as the upper limit for the CP performance level, $rd_{NS,CP}$.

Concerning structural performance, the limitations on deformations are implicitly dictated by the compliance with the ones postulated for non-structural levels. Moreover, response is assessed in terms of plasticization levels of the main members constituting the steel structure. For all design solutions, no plasticization is admitted for the structural OP and IO limit states. This requirement is extended to the LS and CP levels for the EXB and VDB design hypotheses. In the case of the DBXB solution, considerable buckling effects are expected in several braces at the LS limit state. Indeed, according to the design philosophy of Standards [11] and Eurocode 8 [15], as well as of any damage control-based approach to the seismic design of steel structures [16], plastic activity must be primarily concentrated in the bracing members, with a smaller involvement of columns and beams, so as to facilitate the possible post-earthquake reparation and/or substitution of damaged members. The level of plasticization must be compatible with the preservation of the original strength and stiffness of the structural system against vertical loads, as well as of a residual horizontal stiffness equal to about 50% of the original elastic stiffness. The CP-related requirements consist in keeping the vertical load-bearing capacity of the structural system, though under severely damaged conditions of the bracing members and moderate-to-medium damage of columns and beams; and an approximately 25% residual fraction of the original horizontal stiffness, with very low safety margins from collapse against seismic forces. For all design hypotheses and performance levels, no damage to foundations is allowed.

5. DESIGN CONCEPTION OF THE VERTICAL STRUCTURAL SYSTEM

The classical design of the vertical structures of steel buildings tends to separate the role of standard columns (the gravity load-bearing system) from the role of bracings (the seismic and wind load-resisting system). In the examined case study, this design conception is regularly applied to the VDB and DBXB design solutions, as an acceptably small number of vertical bracing alignments results to be necessary in both cases (2 on both building sides along x as well as y, for a total of 8 — VDB; 2 on the floor side and 4 on the opposite side along x, and 2 on both sides along y, for a total of 10 — DBXB), as illustrated in the next sections. This allows meeting the basic architectural requirement of limiting the visual impact of bracings on the glazed façades, mentioned in the introduction. On the other hand, this objective cannot be met for the EXB solution if the same clear distinction between gravity and seismic/wind load resisting functions is considered in this case too. Indeed, in this hypothesis, the incorporation of bracings should be necessarily extended to all vertical alignments along x, and to at least 6 out of the 12 alignments along y, as a consequence of the highly demanding approach followed in this special design (q=1). In this configuration, maximum 8 standard gravity-only load bearing columns would be kept, while the remaining 30 out of 38 would be involved in the bracing system. This poorly balanced proportion, along with the drastically prevailing sizes of the 30 columns belonging to the bracing alignments over the sizes of the 8 residual standard columns, would significantly impair the aesthetic quality of the building. Therefore, for the EXB design hypothesis the columns not incorporated in the bracing system were preferably involved in the absorption of the horizontal loads. This was obtained by designing semi-rigid connections, rather than hinged joints, for these columns. This way, braces could be placed exactly in the same alignments as selected for the DBXB solution. As a consequence, not only all architectural restraints were met, but also the structural design was balanced properly. Moreover, the cost of the building resulted to be lower, as detailed in section 8.3.

It should be remarked that this necessarily different conception of the vertical structures does not affect the comparisons between the EXB design hypothesis and the other two solutions. Indeed, according to the objectives of the study formulated in the introduction, the comparisons are carried out in terms of performance for similar costs (VDB *vs* DBXB), or of costs for similar performance (VDB *vs* EXB), for the global structural solutions adopted in the three design hypotheses, rather than specifically in terms of response of the three bracing systems.

6. VDB DESIGN SOLUTION

The FV spring-dampers incorporated in the VDB system, detailed in [17-22], produce their damping action by a compressible silicone fluid flowing through the thin annular space found between the piston head and the internal casing (Figure 10). The inherent re-centering capacity of the devices is ensured by the pressurization of the fluid imposed upon manufacturing by means of a static pre-load force. The $F_d(t)$ damping and $F_{ne}(t)$ non-linear elastic reaction forces corresponding to the damper and spring functions of the devices are effectively simulated by the following analytical expressions [17], [23]:

$$F_d(t) = c\,\mathrm{sgn}(\dot{x}(t))\left|\dot{x}(t)\right|^{\alpha} \tag{1}$$

$$F_{ne}(t) = k_2 x(t) + \frac{(k_1 - k_2)\,x(t)}{\left[1 + \left|\dfrac{k_1 x(t)}{F_0}\right|^5\right]^{1/5}} \tag{2}$$

where c=damping coefficient; sgn(·)=signum function; |·|=absolute value; α=fractional exponent, ranging from 0.1 to 0.2 [17]; F_0=static pre-load force; k_1, k_2=stiffness of the response branches situated below and beyond F_0.

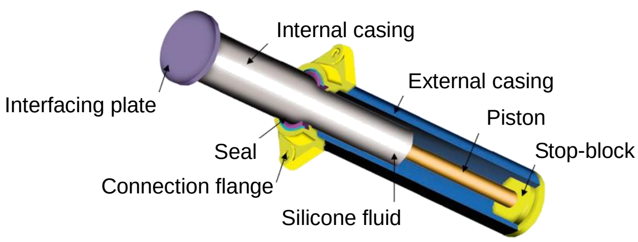

Figure 10. Cross Section of a Pressurized FV Spring-damper

The basic layout of the VDB system, illustrated in Figure 11, consists in a pair of interfaced FV spring-dampers installed in parallel with the floor-beam axis, at the tip of each couple of supporting steel braces. A half-stroke initial position is imposed on site to the pistons of both dissipaters by means of the connecting threaded steel bars, so as to obtain symmetrical tension-compression response cycles, starting from a compressive-only response of the single devices [17], [20]. Renderings of the installation details in the case study building are shown in Figure 12.

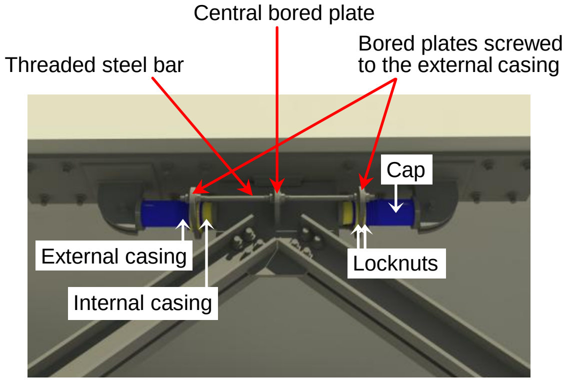

Figure 11. Typical Installation Layout of FV Spring-dampers in the VDB System

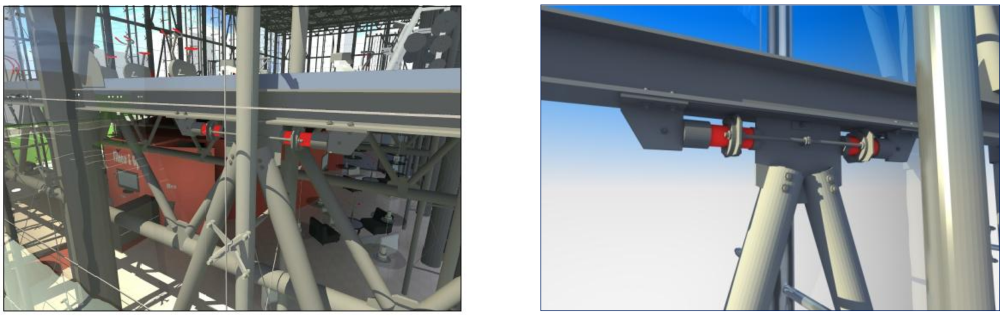

Figure 12. Renderings of the Installation of FV Spring-dampers in the VDB System of the Case Study Building

A preliminary design of the dissipative bracing system was developed by the general criterion formulated for applications to frame structures in [6], which consists in assigning the set of FV devices to be installed on a building story the capability of dissipating a prefixed fraction of the maximum seismic input energy computed by the numerical model of the structure on that story. By adapting the criterion to the special configuration of the building, which does not include whole intermediate floors, the energy balance was computed for the overall structure, rather than story by story, by assigning 90% of the total seismic input energy to the whole set of dampers to be incorporated in the system, along both x and y. Two types of spring-dampers were adopted, namely BC1FN and BC1GN type, selected from the manufacturer's basic catalogue [24]. The values of the mechanical parameters included in Equations (1) and (2) for these devices are: c=6.2 kN·(s/mm)$^{\alpha}$ (BC1FN), c=13.8 kN·(s/mm)$^{\alpha}$ (BC1GN); α=0.15 (BC1FN and BC1GN); k_2=1 kN/mm (BCIFN), k_2=1.25 kN/mm (BC1GN); k_1=20 k_2 (BC1FN and BC1GN); and F_0=90 kN (BC1FN), 150 kN (BC1GN). Further mechanical characteristics are: nominal energy dissipation capacity E_n=6 kJ (BC1FN), 12 kJ (BC1GN); maximum reaction force R_{max}=150 kN (BC1FN), 230 kN (BC1GN); and stroke d_{max}=65 mm (BC1FN), 80 mm (BC1GN). As anticipated in section 5, the devices were placed over eight symmetrical vertical alignments, sketched in Figure 13, each being subdivided in three levels, determined by the geometry of the braces along the height. These levels correspond to 1/3 of the building height (6 m, coinciding with the floor height), 2/3 (12 m), and the top (18 m). The torsional response effects caused by the eccentric position of the floor with respect to the x axis were compensated by incorporating at the first level of the alignments parallel to this axis the smallest (BC1FN) devices, on the floor side, and the biggest (BC1GN) devices, on the opposite side. BC1FN dissipaters were introduced at the second and third level on both sides, and BC1GN dissipaters were placed at all levels along y, for a total of 24 pairs of devices (10 pairs of BC1FN and 14 pairs of BC1GN elements).

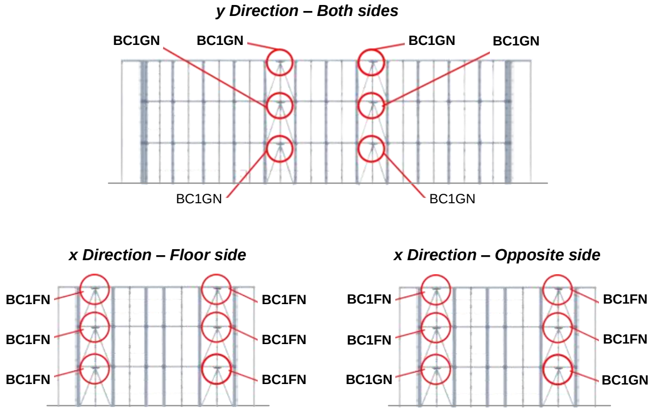

Figure 13. VDB Design Solution: System Alignments and FV Spring-damper Types

Table 1. VDB Design Solution: Profiles of the Main Structural Members

Tubular profiles (diameter×thickness) – (mm×mm)	
Standard columns	219.1×8
Columns included in bracing alignments	355.6×20
Braces	168.3×10
Italian H-shaped profiles	
Perimeter beams	HEB 160

The profiles adopted for the main members of the vertical structure are summarized in Table 1. These members include the internal columns, either of standard type or belonging to the vertical bracing alignments, the perimeter beams situated at the three levels of the same alignments, and the diagonal braces. As observed in section 2, all the other members, i.e. beams, trusses and horizontal braces of the floor and the roof, columns and beams of the stairs, and columns of the arcade, are equal for the three design solutions.

The modal parameters of the VDB-protected structure result as follows. The first mode is purely translational along y, with vibration period T_1 equal to 0.88 s and effective modal mass (EMM) equal to 77.3% of the total seismic mass of the building. The second mode is mainly translational along x, with a relatively low contribution of the rotation around the vertical axis z. The vibration period T_2 is equal to 0.72 s and the EMM is equal to 75.6% along x and 9.5% around z. The third mode is purely rotational around z, with T_3=0.45 s and EMM=38.1%. The fourth mode is again purely translational along y, and is sufficient to obtain a summed effective modal mass (SEMM) greater than 90% along the same axis. Indeed, this mode shows 18.9% EMM, along y, giving rise to 96.2% SEMM. At the same time, the third translational mode is needed, in addition to the second one, to reach a SEMM greater than 90% in the x direction (attaining a total value of 93.2%). Concerning the calculation of modal parameters, it should be noted that the spring-dampers respond with their second branch stiffness k_2 beginning from very low input seismic actions (i.e. with amplitudes just capable of overcoming the static pre-load F_0). Therefore, in order to compute the actual dynamic parameters characterizing the seismic response of the VDB-protected structure, the first branch of the spring component of the devices was removed from the finite element model of the structure when the modal analysis was carried out, so that the devices may respond with k_2 only [6-9]. As a consequence, due to the in-series connection of the spring-dampers to the supporting steel braces, the stiffening effects of the VDB system on the modal behaviour of the structure result to be very little, as k_2 is very low in comparison to the stiffness of the supporting braces (the modal parameters are nearly the same as the unbraced structure). This explains why the vibration periods are considerably higher than those calculated for the DBXB hypothesis, reported in the next section, although the sizes of the bracing system of this design solution are not much greater than the sizes selected for the VDB-protected structure.

7. DBXB AND EXB DESIGN SOLUTIONS

The profiles adopted for the main members of the vertical structure in the DBXB and EXB hypotheses are listed in Tables 2 and 3. As mentioned in section 5, the torsional effects were offset in these cases by doubling the bracing alignments along x on the opposite side of the floor (passing from two to four). This design choice helps to obtain a satisfactory modal behaviour, although not characterized by purely translational modes along x (similarly to the VDB solution), which could be reached by increasing further the number and/or the size of braces on the opposite side of the floor, but with negligible additional benefits on the modal response in plan and a considerable increase in costs.

Table 2. DBXB Design Solution: Profiles of the Main Structural Members

Tubular profiles (diameter×thickness) – (mm×mm)	
Standard columns	219.1×8
Columns included in bracing alignments	406.4×25
Braces	193.7×14.2
Italian H-shaped profiles	
Perimeter beams	HEB 200

Table 3. EXB Design Solution: Profiles of the Main Structural Members

Tubular profiles (diameter×thickness) – (mm×mm)	
Standard columns	508×40
Columns included in bracing alignments	610×40
Braces	273×20
Italian H-shaped profiles	
Perimeter beams	HEB 320

Based on these assumptions, for both design hypotheses the first mode results to be purely translational along y, with vibration period T_1 equal to 0.71 s (DBXB) and 0.52 s (EXB), and EMM equal to 74.2% (DBXB) and 76.2% (EXB) of the total seismic mass. Similarly to the VDB solution, the second mode is mainly translational along x, with T_2 equal to 0.48 s (DBXB) and 0.37 s (EXB), and EMM equal to 73.4% along x and 15.6% around z (DBXB), and 73.3% along x and 15.1% around z (EXB). The third mode is purely rotational around z, with T_3=0.35 s and EMM=33.5% (DBXB), and T_3=0.28 s and EMM=28.7% (EXB). The fourth mode is again purely translational along y, with EMM of 22.6% (DBXB) and 17.1% (EXB), which produces a SEMM of 96.8% (DBXB) and 93.9% (EXB) along the same axis, in combination with the first mode. In these two design hypotheses too, the third translational mode in x direction is required to reach a SEMM greater than 90%, with total values of 93.8% and 94.2%, for DBXB and EXB, respectively.

8. SYNTHESIS OF DYNAMIC ANALYSES, PERFORMANCE EVALUATIONS AND COST ESTIMATES

The finite element models used to carry out the dynamic time-history design analyses of the structure for the three bracing solutions were elaborated with the SAP2000NL commercial analysis program [25]. One of the three models (VDB configuration) is demonstratively shown in Figure 14. Four sets of seven artificial accelerograms, generated from the FDE, SDE, BDE and MCE-scaled response spectra in Figure 9, were assumed as inputs for the analyses. A schematic roof plan and the positions of the upper joints of the two most distant internal columns (denoted by letters A and B), which were selected as control points for the evaluation of the global response of the building, are sketched in Figure 15.

The evaluation of the performance of the glazed façades was developed by separate finite element models, which included all the constituting panes (reproduced by shell elements), the spider connections and the supporting aluminium trusses (simulated by frame elements). The maximum displacements of the upper ends of the trusses (hinged to the perimeter beams of the roof, as mentioned in section 2) deduced from the time-history analyses of the structure for the various earthquake levels were imposed to the corresponding top joints of the façade models. This allowed precisely computing the displacements of each pane, and comparing these values with the non-structural performance limitations assumed. A view of the model of a longitudinal glazed façade, and a demonstrative out-of-plane deformed shape resulting from one of the analyses carried out, are displayed in Figure 16.

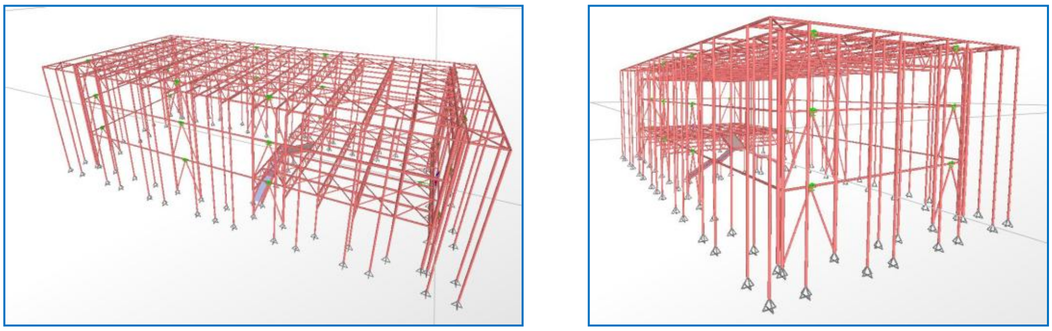

Figure 14. VDB Design Solution: Views of the Finite Element Model

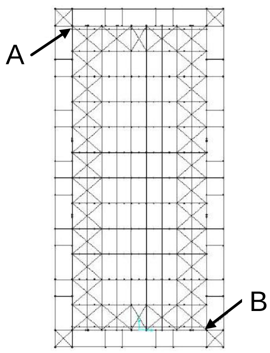

Figure 15. Reference Joints A and B

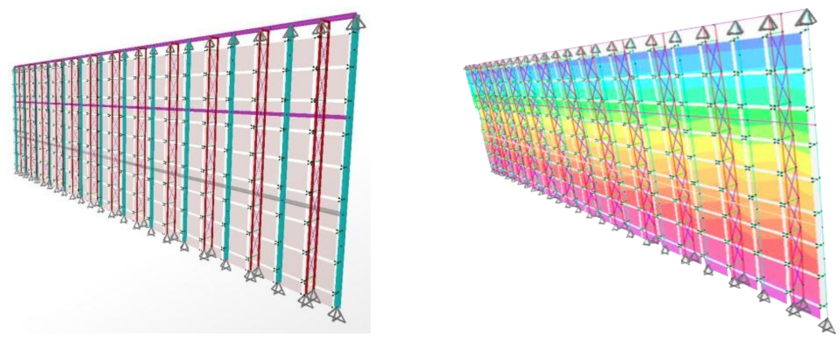

Figure 16. Finite Element Model of the Longitudinal Glazed Façades and Demonstrative Out-of-plane Deformed Shape Derived from Computation

8.1 Response and Performance of VDB Design Solution

As an example of the analyses developed on the structure protected by the VDB system, results extracted from the response to the most demanding of the seven input accelerograms scaled at the BDE level of the input action are reported in Figures 17 through 19. The former illustrates the displacement time-histories of control joints A and B in x and y direction. Differences no greater than 10% are observed in the peak values, which highlight the remarkable constraint of torsional components obtained with this design solution. The response cycles of a pair of BC1FN (floor side) and BC1GN (opposite side) devices situated at the first level of one of the two alignments parallel to x, displayed in Figure 18, exhibit peak displacements no greater than 20 mm. At the MCE level, the maximum displacements are again very low, as they do not exceed 27 mm. Similar data come out for the dissipaters placed on the second and third level along x, as well as for the most stressed devices incorporated in the alignments parallel to y. The energy time-histories plotted in Figure 19 show that around 90% of the total input energy is absorbed by the dissipaters (90.3% in x direction, and 91.5% in y), as targeted in the design of the system.

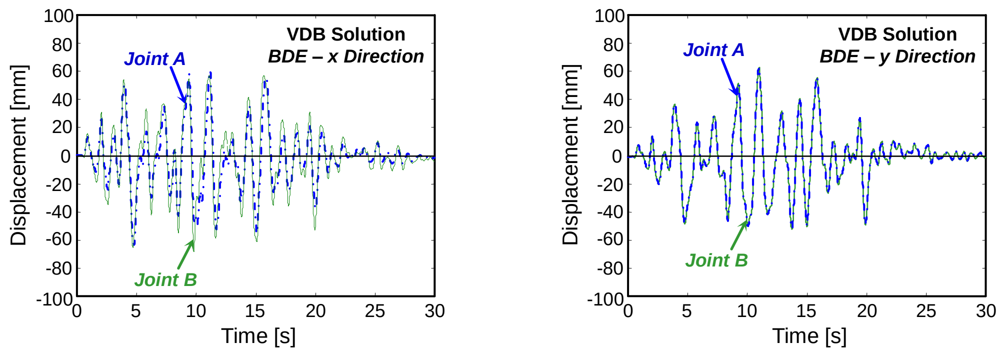

Figure 17. VDB Design Solution: Displacement Time-histories of Reference Joints A and B obtained from the Most Demanding BDE-scaled Accelerogram

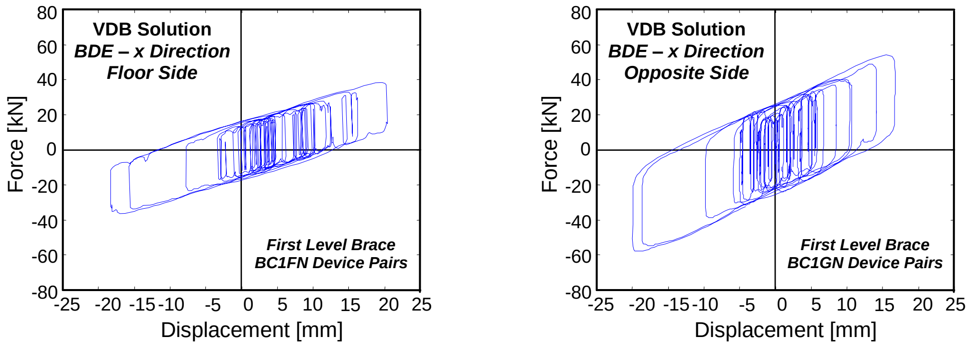

Figure 18. VDB Design Solution: Response Cycles of a Pair of BC1FN and a Pair of BC1GN Devices Situated at the First Level along x obtained from the Most Demanding BDE-scaled Accelerogram

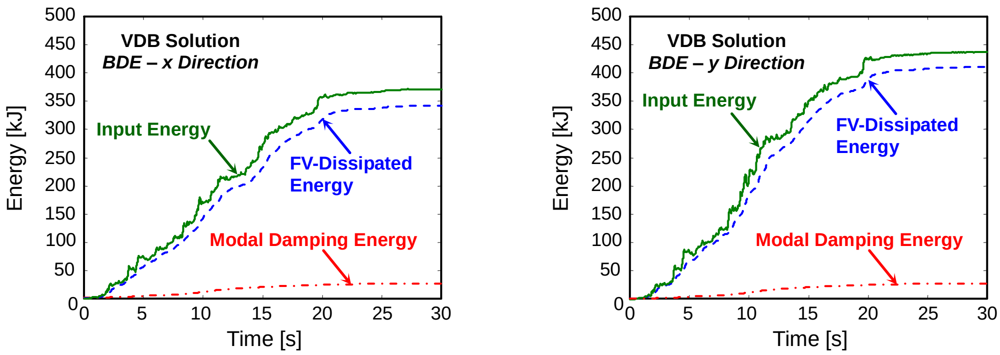

Figure 19. VDB Design Solution: Energy Time-histories of the Structure obtained from the Most Demanding BDE-scaled Accelerogram

The total percent equivalent linear viscous damping coefficient of the structure estimated from the response to the BDE and MCE-scaled levels of seismic action is equal to about 27% and 33%, respectively, with the DB system accounting for 25% — BDE and 30.5% — MCE, and the modal contribution of the steel skeleton accounting for 2% — BDE and 2.5% — MCE. As a consequence of these remarkably damped response conditions, the maximum base shear is limited below 20% — BDE and 24% — MCE of the total weight of the building.

The non-structural performance of the building deduced from the results of the dynamic analyses is summarized in the first column of Table 4, where the mean maximum relative displacements of the glass panes computed over the response to the seven input accelerograms scaled at the four earthquake level amplitudes — $rd_{\mathrm{FDE,max}}$, $rd_{\mathrm{SDE,max}}$, $rd_{\mathrm{BDE,max}}$, and $rd_{\mathrm{MCE,max}}$ — are compared with the admissible relative displacement thresholds formulated in section 4 for the four reference non-structural limit states — $rd_{\mathrm{NS,OP}}$, $rd_{\mathrm{NS,IO}}$, $rd_{\mathrm{NS,LS}}$, and $rd_{\mathrm{NS,CP}}$. The data in Table 4 underline a very high performance of the VDB system, which allows meeting the strict OP-related limitation of $0.005D$ up to the MCE-scaled seismic action.

Table 4. Maximum Relative Displacements of Glass Panes and Comparisons with Performance Limitations

Maximum *rd* values	Design solution		
	VDB	DBXB	EXB
$rd_{\mathrm{FDE,max}}$	0.0012 D < $rd_{\mathrm{NS,OP}}$	0.0035 D < $rd_{\mathrm{NS,OP}}$	0.0011 D < $rd_{\mathrm{NS,OP}}$
$rd_{\mathrm{SDE,max}}$	0.0016 D < $rd_{\mathrm{NS,OP}}$	0.0046 D < $rd_{\mathrm{NS,OP}}$	0.0014 D < $rd_{\mathrm{NS,OP}}$
$rd_{\mathrm{BDE,max}}$	0.0042 D < $rd_{\mathrm{NS,OP}}$	0.0148 D < $rd_{\mathrm{NS,LS}}$	0.0036 D < $rd_{\mathrm{NS,OP}}$
$rd_{\mathrm{MCE,max}}$	0.0048 D < $rd_{\mathrm{NS,OP}}$	0.0211 D < $rd_{\mathrm{NS,CP}}$	0.0044 D < $rd_{\mathrm{NS,OP}}$

The structural performance of the building is qualified by the absence of plasticization in all members, which satisfies the basic requirement for the structural OP level up to the MCE amplitude too. By combining the results of non-structural and structural dynamic assessment analyses, the formal evaluation summarized in the four leftmost columns of Table 5 is obtained for the VDB-protected building, which corresponds to the highest attainable performance objective in the seismic design of a new structure.

Table 5. Assessment of Non-structural (NS) and Structural (S) Building Performance

	Design solution											
	VDB				DBXB				EXB			
	OP	IO	LS	CP	OP	IO	LS	CP	OP	IO	LS	CP
FDE	NS				NS				NS			
	S				S				S			
SDE	NS				NS				NS			
	S				S				S			
BDE	NS						NS		NS			
	S						S		S			
MCE	NS							NS	NS			
	S							S	S			

8.2 Response and Performance of DBXB and EXB Design Solutions

The performance of the two X-bracing design solutions is summarized in Tables 4 and 5 as well. The mean maximum relative displacements of the glass panes for the EXB hypothesis are very similar to the ones of the VDB-protected structure (Table 4), and no plasticization of steel members is observed again. Therefore, the resulting non-structural and structural performances coincide for the two designs. The quality of response is very similar too, with differences in the peak displacements of control joints A and B below 10%, like in the VDB case. The maximum base shear is equal to about 61% — BDE and 73% — MCE of the total weight of the building, as a consequence of its totally elastic response at all earthquake levels. These high base shear values confirm, under a different viewpoint, the need for very massive dimensions of the structural members.

The DBXB-protected building meets the non-structural OP limitation of 0.005*D* for the FDE and SDE-scaled amplitudes. At these earthquake levels, the response is totally elastic, and characterized by very little torsional effects, again with differences lower than 10% between the displacements of joints A and B. Buckling occurs in about 45% of braces (modelled in the non-linear field as trusses with elastic buckling in compression and bilinear elastic-plastic behaviour in tension) at the BDE, and in about 85% at the MCE, whereas relatively little plasticization is observed in about 30% of beams and 20% of columns (both schematized by a Giberson-type one-component model [26], with the end-section plastic hinges modelled by inelastic springs characterized by a Bouc/Wen-type [27] moment-rotation constitutive relationship). The total drifts computed at the roof level are equal to 229 mm (BDE) and 327 mm (MCE), corresponding to 1.27% (BDE) and 1.82% (MCE) of the building height. The residual (plastic) drifts are: 31 mm (0.17%) — BDE, and 129 mm (0.72%) — MCE. The residual horizontal stiffness of the structure is equal to 75% (BDE) and 46% (MCE) of the elastic stiffness, meeting the requirements of the structural LS and CP levels, respectively. The mean peak relative displacements of glass panes reach 0.015*D* (BDE) and 0.021*D* (MCE), which are below the non-structural LS limit of 0.02*D*, and CP limit of 0.03*D*, respectively. The resulting combined evaluation (NS-OP, S-OP — FDE; NS-OP, S-OP — SDE; NS-LS, S-LS — BDE; NS-CP, S-CP — MCE, where NS, S mean non-structural and structural) identifies the basic multi-level performance objective typically underlying the ductility-based design of braced steel structures, with the only exception of a slightly better performance at the SDE, for which the non-structural and structural IO levels are normally targeted, instead of the OP levels reached in this case. The maximum base shear is equal to about 15% — BDE and 18% — MCE. Both values are 25% lower than the corresponding values computed for the VDB solution, thanks to the relatively low ordinates of the design response spectra determined by the behaviour factor of 3.2.

8.3 Comparison of costs

The total weight of the structural members mutually adopted for the three solutions, described in section 2, is equal to about 142 tons, with a cost of 312,000 Euros, structural connections and installation expenses included. The weights and costs of the profiles in Tables 1, 2 and 3 are: 119 tons and 248,000 Euros, 190 tons and 398,000 Euros, and 496 tons and 1,090,000 Euros, respectively. Therefore, the total weights and costs of the steel structure are: 261 tons and 560,000 Euros (VDB), 337 tons and 710,000 Euros (DBXB), 638 tons and 1,402,000 Euros (EXB). For the VDB hypothesis, the cost of the spring-dampers must be added. This amounts to 128,000 Euros, given by the sum of 44,000 Euros for the set of 20 BC1FN devices (2,200 Euros per device) and 84,000 Euros for the set of 28 BC1GN devices (3,000 Euros per device). Therefore, the total cost of the steel structure for the VDB solution is equal to 688,000 Euros. The cost of the foundations amounts to 90,000 Euros (VDB), 120,000 Euros (DBXB), and 145,000 Euros (EXB), which determine the following total structural costs: 778,000 Euros (VDB), 830,000 Euros (DBXB), and 1,547,000 Euros (EXB).

These data show that the cost of the VDB-based design is slightly lower than the one of the DBXB solution (about 6%), and is about 50% lower than the cost of the EXB layout capable of producing the same overall performance. In addition to these remarkable economic advantages, the comparison between the VDB and EXB solutions show a considerably lower impact on the visual perception of the building for the viscous dissipative design, as a consequence of a drastic drop in member sections. This is visually highlighted by the renderings of a mesh of a bracing alignment demonstratively displayed in Figure 20. The aesthetical benefits are evident, especially for columns, which are very massive in the case of the EXB design.

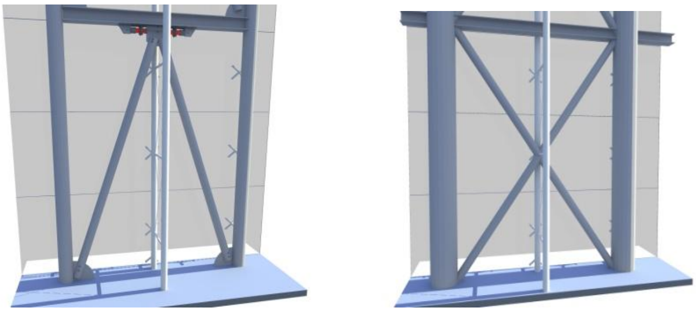

Figure 20. Rendering of a Mesh of Braces for the VDB and EXB Design Solutions

It can be noted that, should the EXB solution be designed by keeping the traditional separation between gravity load-bearing columns and bracing systems, the following tubular profiles would result: 219.1 mm × 8 mm, for the 8 standard columns (like for the VDB and DBXB hypotheses); 610 mm × 50 mm, for the 30 columns involved in the bracing system; and 323.9 mm × 25 mm, for diagonal braces. Apart from the negative effects on the building's aesthetic quality and the overall balance of structural proportions, commented in section 5, the cost of the steel structure in this case would be equal to 1,671,000 Euros, with an increase of 114,000 Euros (+7.4%) as compared to the basic EXB solution developed in this study.

The visual impact of the DBXB solution is similar to that of the VDB design, except for the layout of braces, a greater size of these members as well as of the columns involved in the bracing system, and the incorporation of two additional bracing alignments on the opposite side of the intermediate floor. The lower performance of the dissipative X-bracing hypothesis causes reparation costs of the structure and the glazed façades (all other finishes, plants and furniture excluded) estimated at 240,000 Euros for a seismic event comparable to the BDE, by amplitude and spectral composition. This amount is partly owed to the substitution of the damaged glass panes, and particularly of about 80% of their total number, which exceed the 0.01D limit of visible and irreparable cracking, with a rounded cost of 100,000 Euros, as the total cost of glass panes is equal to about 130,000 Euros. The spider fittings and supporting aluminum trusses, whose total cost amounts to 150,000 Euros, remain practically undamaged at this input earthquake level, and they need no reparations. The remaining share of reparation costs, that is about 140,000 Euros, is related to structural damages, and namely to the substitution of the braces that are subject to buckling, and by the repair works required by the beams and columns that are subject to plastic rotation. The reparation costs increase to about 200,000 Euros, for the façades, and 270,000 Euros, for the structural elements, giving rise to a total amount of 470,000 Euros, for a seismic event comparable to MCE.

9. CONCLUSIONS

A novel application of the viscous-dissipative bracing system incorporating pressurized FV spring-dampers, originally conceived for installation in multi-story frame structures, was developed in this study with regards to an open-space, single-span steel indoor sports facility wrapped by continuous floor-to-roof glazed façades, the characteristics of which are typical of the most recent architectural design trends for recreational and commercial steel buildings. The performance offered by the system, and a comparison with the ductility-based and elastic X-bracing hypotheses carried out as alternative design solutions, prompts the remarks reported below.

− The high seismic protection capabilities of the system already evaluated in previous research activities dedicated to the analysis of new steel frame buildings, as well as to the retrofit of existing ones, were confirmed in the new design context demonstratively represented by the case study building. The limitations for the non-structural OP performance level originally formulated in this paper for the spider-joined panes constituting the glazed façades, fixed by collecting and elaborating requirements and suggestions from international seismic Standards, as well as from Technical and manufacturers' Recommendations specially dedicated to the design of glass members, were met up to the highest design earthquake intensity. Also, the basic requirement for the achievement of the structural OP level, i.e. no plasticization of the steel skeleton elements, was satisfied up to MCE, determining the highest attainable combined performance objective in the seismic design of a new structure.

− These high performance capabilities were reached with structural sizes of the bracing system of the VDB-protected structure notably smaller than the ones obtained for the ductility-based X-bracing solution, with the standard columns assumed with equal sections in the two cases. This further favours an effect of slenderness in the outer look of the building. The costs — spring-dampers and foundations included — are slightly lower (6%) for the VDB design.

− The performance capacities targeted for the DBXB system, consisting in the attainment of the basic multi-level objective identified by a "diagonal" correlation between performance levels and earthquake levels (OP–FDE, IO–SDE, LS–BDE, CP–MCE), were actually met, except for the improved "extra-diagonal" correlation between non-structural and structural OP levels and SDE obtained in this case. The reparation costs for a seismic event with amplitude and spectral characteristics comparable to BDE were estimated at about 240,000 Euros for the structure and the façades, i.e. about 24% of the original construction cost of 990,000 Euros (710,000 Euros — structure, plus 280,000 Euros — façades). The reparation costs increase to about 470,000 Euros, i.e. about 47% of the construction cost, for a seismic event comparable to MCE.

− The elastic X-bracing solution designed for the same performance objectives as the VDB one shows a total weight of the structure about 2.5 times greater, and a cost 2 times greater. The outer look of the EXB-protected steel skeleton results to be inevitably massive. Nonetheless, a worsened architectural and structural look, and a further 7% increase in costs, would derive in the hypothesis of basing the EXB design on the classical separation between gravity load-bearing columns and bracing systems, which was satisfactorily adopted, instead, for the VBD and DBXB hypotheses.

The study highlights the advantages offered by the viscous-dissipative bracing system, and potentially by any other supplemental damping-based protection technology, in a different field of application, poorly explored as yet. The system, in fact, offers a convincing solution to the combined issues of enhanced seismic performance and reduced dimensional impact required by the latest generation of steel buildings.

ACKNOWLEDGEMENTS

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.

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