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Authors: Franco Mola, Owner, ECSD
Elena Mola, CEO, ECSD
Laura Pellegrini, Senior Structural Engineer, ECSD

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Franco Mola

Elena Mola

Laura Pellegrini

Authors

Franco Mola, Owner
Elena Mola, CEO
Laura Pellegrini, Senior Structural Engineer
ECSD S.r.l.
Via Goldoni, 22
20129 Milano
Italy
t: +39 02 7395 4653
f: +39 02 7000 8547
e: elena.mola@ecsd.it
www.ecsd.it

Franco Mola founded ECSD S.r.l., of which he is the owner and CEO, in 2007. He is also a full professor of Reinforced Concrete and Prestressed Concrete Structures at the Politecnico di Milano, Italy. His research and design activity focuses on the effects of the long-term behavior of concrete in complex structures of tall buildings. He has authored and presented more than 250 conference and peer-reviewed papers, and keynote lectures worldwide. He is the structural designer of Palazzo Lombardia and Allianz Tower in Milan, and the new Torre Regione Piemonte Headquarters in Turin.

Elena Mola has been a partner of ECSD since 2007. She now also serves as the CEO, human resources, project management supervisor, and earthquake engineering consultant. She has a PhD in earthquake engineering from the Institute National Polytechnique de Grenoble, France. She worked as a grantholder at the European Laboratory for Structural Assessment of the Joint Research Centre of the European Commission, where she was involved in the experimental and analytical investigation of the seismic response of buildings by means of pseudodynamic testing.

Laura Pellegrini earned a degree in civil engineering from Politecnico di Milano in 2007. Her main research topic is the analysis of the effects of long-term deformations in concrete elements in tall buildings. She was a key member of the design and construction site supervision team of Palazzo Lombardia and Allianz Tower in Milan, and the new Torre Regione Piemonte Headquarters in Turin.

The Allianz Tower, Milan, part of the CityLife complex, is notable for its slenderness and the eye-catching presence of four diagonal struts that stabilize it at the base (see Figure 1). It was a finalist for the 2016 CTBUH Best Tall Building Europe award, largely on account of its unusual structural system. This paper examines the design, testing and implementation of the primary steel and concrete structural systems in the building.

General Features of the Structural System

The Allianz Tower has a rectangular 24-by-61-meter footprint. The building has three underground floors and 50 floors above ground. The vertical structural elements consist of two lines of columns: a set of peripheral columns, which are spaced six meters apart and distributed on each of the longer sides, plus four central megacolumns, 12 meters apart on one side and 2.4 meters apart on the other side. Two shear-resisting reinforced-concrete (RC) service cores are located at the ends of the longer sides of the building, each with a 5.8-by-20.6-meter footprint (see Figure 2).

The slabs consist of continuous, 200-millimeter-thick RC slabs covering the central, four meters-long, and the lateral, eight-meter-long spans. These are supported by perimeter T-beams, spanning six meters and having a depth of 450 millimeters, and by continuous central T-beams, spanning 12 meters and having a thickness of 500 millimeters.

Figure 1. Allianz Tower, Milan. © Alessandra Chemollo

Reinforced concrete is used for all the slabs and the cores: the classes of concrete used for the different elements are shown in Figure 3. The diameters of the circular cross-sections of the columns range between 0.65 and 1.2 meters for peripheral columns and between 0.85 and 1.7 meters for the central megacolumns. The reduced dimensions of the sections required the use of high-strength concrete of C70/85 grade, as required by the Italian building code. Also, the maximum allowable steel/concrete ratio, ρsmax ≤ 4%, was used. Additionally, composite sections were required up to level 4 for internal columns and level 21 for external columns, in order to provide adequate capacity at the ultimate limit state.

In order to guarantee a better coupling of the shear-resisting cores, and to limit the displacements due to lateral loads, two special perimeter-truss systems were designed, consisting of two belt trusses each, connecting the cores at the corners. The first perimeter truss is placed mid-height, i.e., between levels 23 and 26, and consists of two-story steel truss beams, whereas the second truss is a prestressed RC wall beam, placed at the top of the building, i.e., between levels 49 and 50. The red boxes in Figure 2 show the plan location of the two belt trusses; this special configuration sets level 23 and level 49 apart from the typical floors.

The perimeter truss systems are meant to enhance the performance of the structural system for lateral loads, especially in the direction of minimum inertia, where the global geometric slenderness factor is 18.9.

Figure 2. Typical floor plan. The red boxes show the location of the belt trusses at levels 23 and 49 only. The red text shows the location of elements whose vertical displacements were compared.

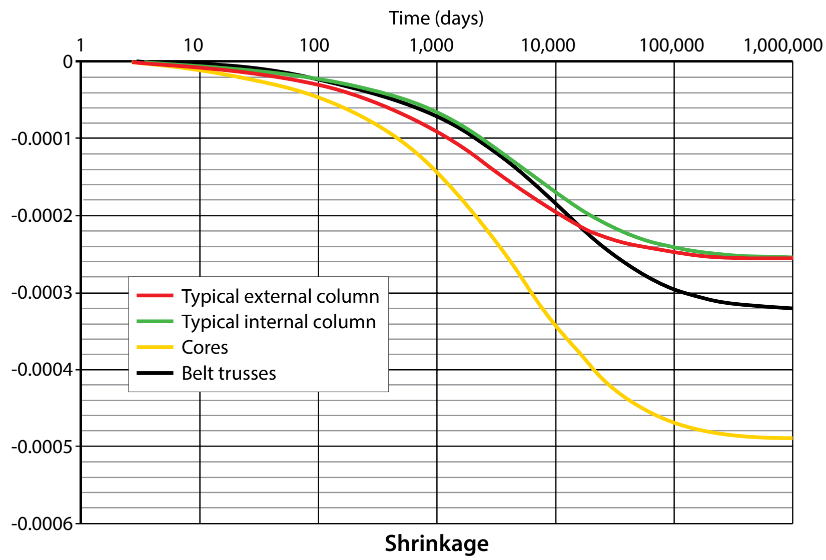

Figure 4. Evolution of shrinkage deformation over time, by element

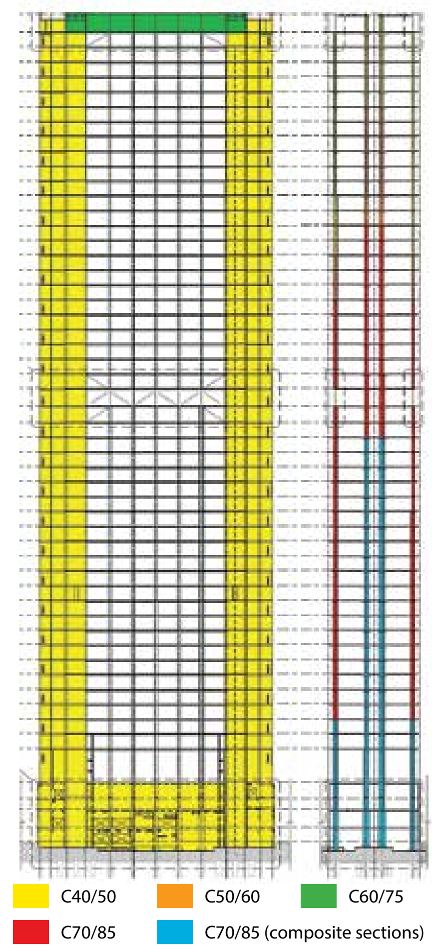

Figure 3. Classes of concrete by element.

Finally, four external steel struts, covered in gold paint, jut out of the building at mid-height, connecting it to the ground, at the top of the podium. At the base of each strut, two bidirectional viscous dampers are installed, which help mitigate the effects of the resonant component of the wind excitation, thus improving the comfort of the building.

Because of the extensive use of reinforced concrete for the structural elements, the structural system can be defined as "hybrid with localized inhomogeneity," mainly due to the steel belt trusses at levels 23–26.

Structural Analysis Approach

Structural analysis was carried out by means of local and global finite element models, implemented in the commercial software MidasGen, each with different features according to the considered limit state. In particular, to take into account the effects of the long-term behavior of concrete and its interaction with the steel elements, a construction-stage analysis was carried out. To quantify the effects of lateral loads due to wind and earthquakes, a global elastic model was used. Moreover, specific local models were implemented for the stress analysis of structural elements, for the evaluation of the cracking limits state, for inelastic analysis of the design, and for verification of the bending moment capacity of structural elements in the slabs.

The effects of the long-term behavior of concrete must be thoroughly quantified in order to effect an accurate compensation of the vertical displacements taking place during construction, thus significantly reducing global shortening effects. The *CEB-FIP Model Code 1990* was assumed to describe the evolution of the creep and shrinkage deformation of concrete (CEB 1993). Shrinkage deformation is affected by rebar content: due to higher rebar content, the effects of shrinkage are more limited in the columns than in the core (see Figure 4). Moreover, the initial axial stresses in the columns are higher than those in the core, because for the columns, the ratio between the tributary area pertaining to the element and the area of the element itself is larger.

To take into account the sectional inhomogeneity of the elements, Reduced Relaxation Functions (Mola 1993), were used to evaluate the migration of the stresses from concrete (dashed lines) to steel (solid lines) over time for different values of the geometric steel ratio (see Figure 5).

Construction-Stage Analysis

Based on these assumptions for the behaviors of the material and the sections,

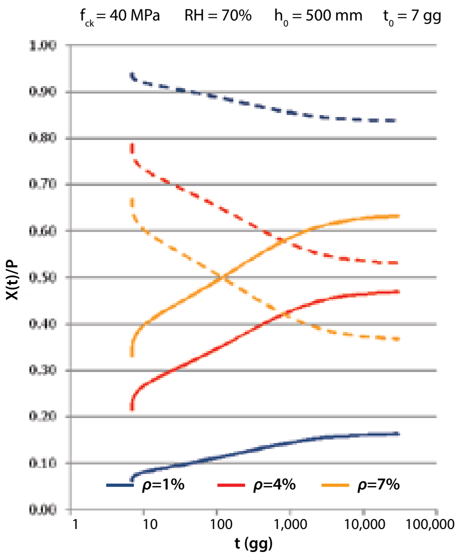

Figure 5. Time evolution of stresses in concrete and steel for different steel rebar content.

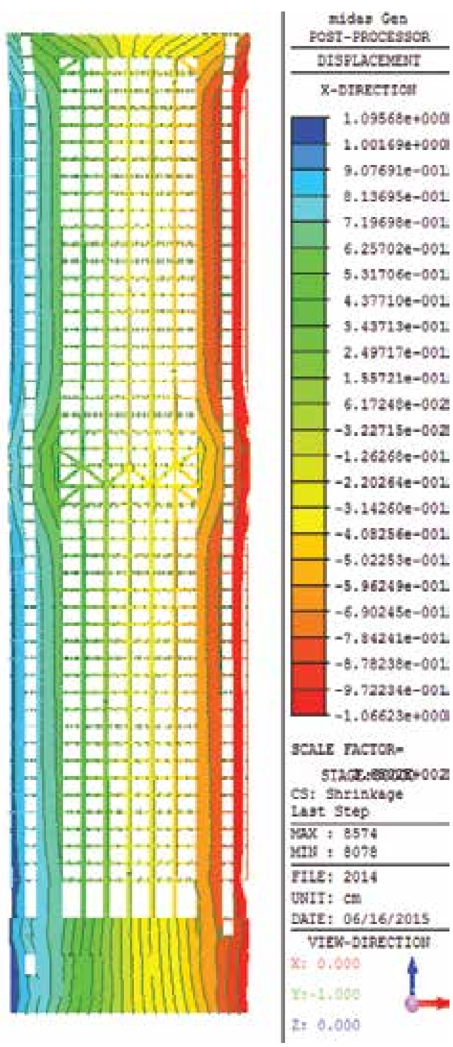

Figure 6. Discontinuity in the horizontal displacements of vertical elements due to constrained shrinkage effects produced by the steel belt truss at level 23.

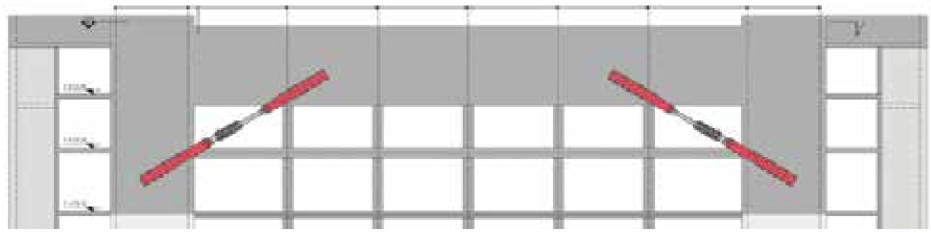

Figure 7. Prestressed concrete at top of belt truss beam, with diagonal steel reinforcements shown.

construction-stage analysis was then carried out by accurately reproducing in MidasGen the construction phases and their duration. The resulting mode incorporated complex rheological features, due to the varying age of concrete members and the sectional inhomogeneity of the composite steel-concrete vertical members. The model provided reliable results in terms of construction-stage vertical displacements, stress redistribution between concrete and steel parts, and long-term prediction of vertical displacements. The MidasGen results were validated by comparison with the theoretical solutions of the basic equations of the viscoelastic theory of non-homogeneous systems, applied to simplified structural sub-schemes extracted from the global model (Mola 1993; CEB 1993).

The preliminary results of construction-stage analysis pointed out that the presence of the two perimeter trusses, originally designed as steel belt trusses (both at the top of the building and at mid-height), introduced a significant discontinuity between their vertical displacements, as a consequence of the marked plastic inhomogeneity between RC columns and steel beams and because of the relative stiffness of the trusses. In Figure 6, the confining effect of the steel belt truss on the lateral shrinkage-induced deformations of the concrete elements is shown.

Post-Analysis Alteration: A Hybrid System

In order to reduce these effects, and to mitigate the differential vertical displacements between the core and the adjacent columns as well, the perimeter truss system located on the top floor, in a later design phase, was replaced with two prestressed RC wall beams, connected to the cores by means of diagonal steel elements encased in the cross section of the RC core walls (see Figure 7). This modification of the design also provided an improvement of the global deformation pattern and, most importantly, a significant simplification of the construction process.

The introduction of the "hybrid" solution, i.e., the upper concrete perimeter trusses, reduces the relative vertical displacements between the core and the adjacent columns (P05-P06-P07) and between the columns themselves (see Figures 2 and 8). Even in the hybrid solution, though, the differential displacements taking place between the cores and the columns cannot be neglected where they connect with the steel belt trusses. The steel elements, in fact, are not able to relax the stress patterns induced by the differential displacements of the vertical elements; thus, remarkable stress concentrations result.

Moreover, the steel elements represent a strong restraint for the shrinkage deformation of the slabs, which causes non-negligible tension stresses. In Figure 9, the axial force and bending moment plots are reported for the truss elements. The time variations of the internal actions are marked.

The behavior of the upper prestressed concrete belt truss system is different: each of the wall beams still acts as a restraint for the deformations of the connecting slabs because their shrinkage deformation is lower, due to the higher class of concrete used in the truss compared to that used in the slabs. Still, because both the belt truss and the columns are made of concrete, the differential deformations are lower than those occurring between the steel belt truss and the slabs. Moreover, the favorable dissipative behavior of concrete reduces the stresses associated with differential deformations over time.

Note that the first mode is mostly flexural along the weak axis of the building and the second mode is mostly flexural along the strong axis, whereas the third mode is mostly torsional.

For dimensioning purposes, the prescriptions of the Italian building code, ***NTC 2008,*** were followed. The code, following a performance-based approach for earthquake loads, requires design-stage consideration of the maximum base shears and the maximum bending moments in the two main directions provided by specified load combinations for the Ultimate Limit State (ULS) and for the Serviceability Limit State (SLS). For ULS in particular, the two conditions called "life safety" and "collapse prevention" must be checked. In this case, a q-factor of 1.5 was assumed to derive the design spectrum for SLS and a q-factor of 2.88 was used for ULS to take into account the global dissipative capacity of the structural system (see Figure 9).

The effects of wind in terms of induced vibrations were also carefully investigated in order to guarantee the comfort of the occupants. The tower is particularly sensitive to the across-wind effects when the wind blows along the long side of the building; also, the sensitivity to wind loads along the weak axis is enhanced by torsional effects, which are evident already in the third mode.

In addition to the high slenderness due to the architectural features of the building, the estimated hysteretic damping values for the tower, according to international guidelines, are around 1%, which further reduces its ability to dampen wind-induced vibrations. For this reason, additional damping devices were introduced in the design phase in order to guarantee high performance levels under wind loads.

> "Replacing the steel perimeter truss with prestressed concrete wall beams improved the global deformation pattern and, most importantly, significantly simplified the construction process."

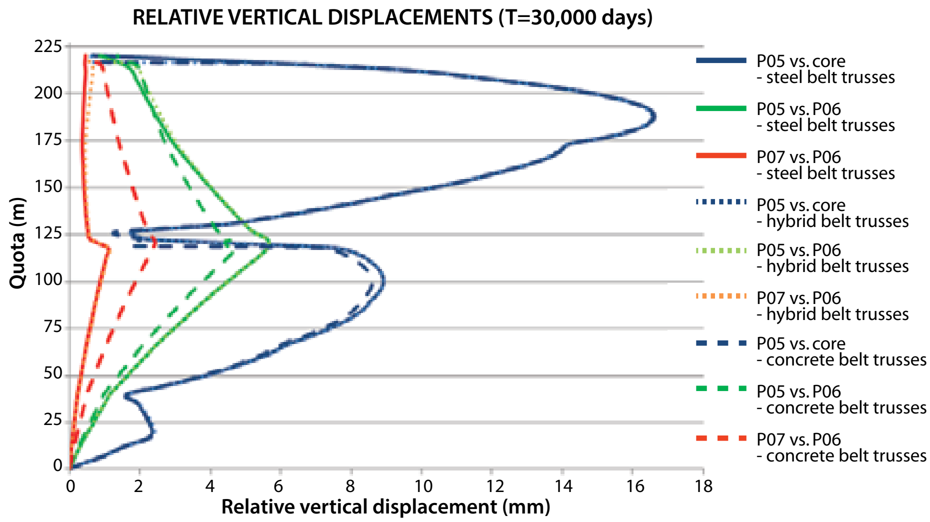

Figure 8. Long-term relative vertical displacements between selected vertical elements: comparison between the solutions with top and bottom steel perimeter trusses, top and bottom concrete perimeter trusses, and "hybrid" solution.

Code Drives Novel Damping System

The design pre-requirements in terms of occupant comfort in service life conditions were strict: the maximum allowed acceleration at the top floor due to wind for a one-year return period was set to be 5 cm/s^2. This value corresponds to 50% for the x and 30% for the y direction of the expected acceleration in the same conditions for the building without additional dampers.

The required value is well within the acceptability range set forth by Italy's Advisory Committee on Technical Recommendations for Construction (CNR) in ***CNR-DT207/2008,*** which is one of the most stringent worldwide (see Figure 10). In the image, the maximum peak accelerations, computed for one-year and 10-year return period winds according to the method codified in ***EN 1991-1-4: 2005,*** are compared with the CNR acceptable values and with

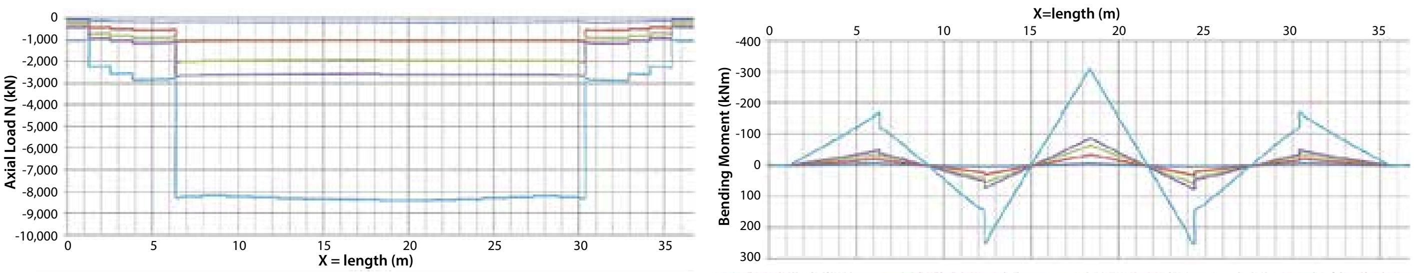

Figure 9. Stresses in the steel belt truss.

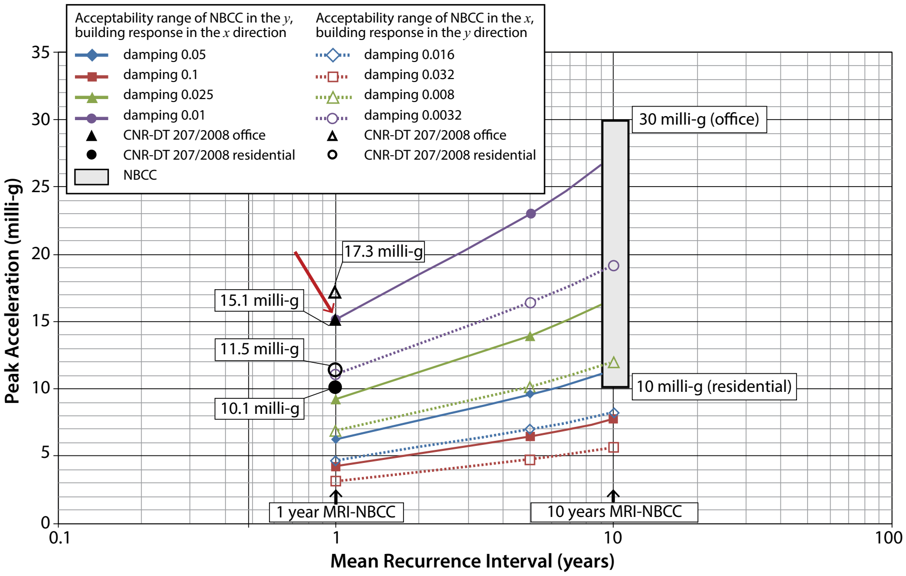

Figure 10. Peak accelerations computed for one-year and 10-year return period wind and a range of damping values, compared with the acceptability limits of CNR-DT207/2008 for residential and office buildings.

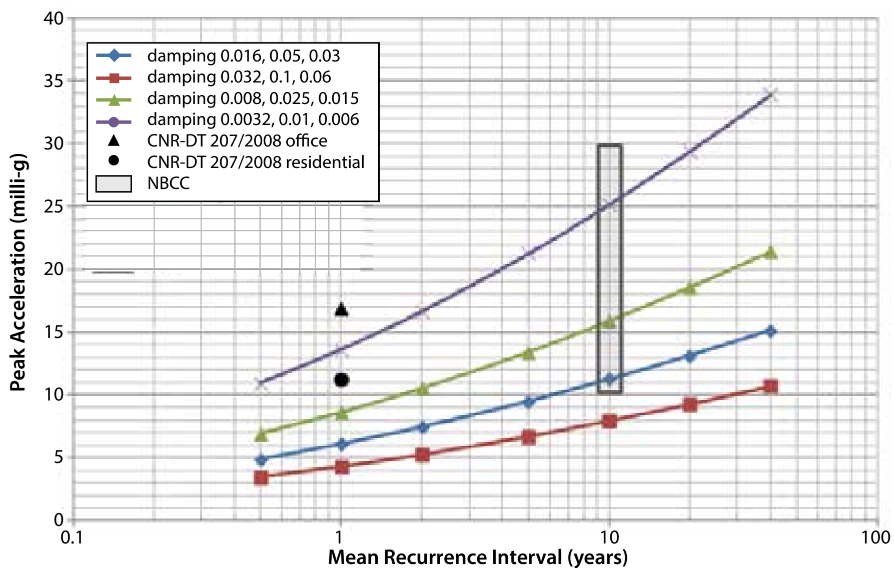

Figure 11. Maximum peak accelerations from wind tunnel tests, compared with the acceptability limits of CNR-DT207/2008 for residential and office buildings.

those codified in the National Building Code of Canada (NBCC).

The estimated accelerations (in the x direction) due to across-wind effects (in the y direction) were computed for different levels of damping and are plotted in Figure 10. For a damping value of 1% (i.e., the inherent hysteretic damping of the building), the accelerations (purple line) are at the acceptable limit for CNR and at the top of the acceptability range of NBCC. On the other hand, if a damping value of around 10% can be provided, the expected acceleration is reduced to 5 cm/s^2. This evaluation defined the starting point for the design of the additional damping devices.

For comparison with the results obtained by the Eurocode codified procedure, the peak acceleration values were also computed starting from the results of wind tunnel tests, which had been carried out in the initial design phase. Starting from the available wind tunnel data, the maximum accelerations, both in terms of Root Mean Square (RMS) and peak values, were computed for the x and y direction. The results obtained with the

Experimental Mode #	Damping (%) without dampers	Damping (%) with dampers
Mode N.1	0.50	0.58
Mode N.2	0.48	1.50
Mode N.3	0.35	1.03
Mode N.4	0.30	0.40
Mode N.5	0.52	0.77

Table 1. Experimental modal damping estimates in the configuration, with and without dampers.

Eurocode approach were confirmed. In the x direction, for a damping level of 1% and for the vibration frequencies associated with the first mode, the results are at the top of the acceptability limits of CNR, whereas in the x direction, for a damping level of 1% and for the vibration frequencies associated with the second mode, the values are above the acceptability limit of CNR for residential buildings (see Figure 10). The estimated value based on wind-tunnel data is lower than those estimated by the EC procedure, which is more conservative; even so, at around 10 cm/s^2, it is 200% of the maximum allowed acceleration. Where the peak acceleration values in the x direction derived from wind tunnel tests are plotted for different damping levels, if an additional modal damping of 0.01 is provided (green line), the maximum acceleration is reduced below 5 cm/s^2 (see Figure 11).

The dampers were thus designed so that when their whole dissipative capacity is exploited, due to prolonged exposure to the "design" wind values and also when the dampers are considered to be fully effective (i.e., the design service temperature of the devices is attained), the nominal additional damping on the second and third mode was set to be no less than 9%. If the dampers are assumed to lose 50% of their efficiency in extreme and prolonged wind conditions, the additional damping on the same modes is reduced to 6%, but is still effective enough to provide a reduction of the maximum accelerations within the range of acceptability of CNR for office buildings.

The installed dampers comply with EN 15129, with the use of a special mineral fluid to adequately lubricate the valves for service life operability. Before the devices could be installed, a number of laboratory tests on prototypes had to be carried out: the experimental tests that were required according to EN 15129 were integrated with a wind test as specified in the US Federal Emergency Management Administration (FEMA) standards, which was also used to evaluate the global expected damping values. The dampers were designed so that they would still be effective under seismic excitation, even if their contribution in terms of lateral force reduction was not taken into account for the ultimate capacity design of the resisting structural elements. In order to be effective in earthquake conditions as well, the dampers were thus equipped with a two-way hydraulic circuit, with different valves for service life and ultimate limit-state operation due to the very different velocities associated with the two loading conditions. The structural steel parts were designed with a focus on toughness and through-thickness properties, with highly reduced tolerances and gaps at each pin connection. A strict monitoring and testing protocol during the production, installation and operation of each component was enforced. The dampers are installed at the base of the steel struts (see Figure 12).

Post-Construction Validation

At the end of construction, dynamic tests were carried out to experimentally derive the global dynamic properties of the building (i.e., natural frequencies, mode shapes, and modal damping), according to the Operational Modal Analysis (OMA) method (Ewins 2000). The experimental tests were carried out in two different configurations, with and without the external viscous dampers (VD), thus allowing for better insight on the operational behavior of the devices.

The modal damping estimations provided by the tests with the dampers are reported in Table 1 and compared to those obtained in the configuration without dampers. The dampers increase the modal damping for Mode N.2 by 3.1 times and Mode N.3 by 2.9 times. These estimations proved that the design pre-requirements for the dampers were achieved. The theoretical behavior curves for the dampers were then plotted (see Figure 13).

On the vertical axis, the theoretical additional damping on Mode N.2 is plotted vs the values of the **C** constant defining the different operational conditions of the dampers. The dotted line shows the additional damping for the dampers in test conditions (i.e., much lower vibration levels than the design level, represented by the continuous line). From the latter, it can be noted that the theoretical predicted value for **C** = 160 kNs/mm/VD is very close to the actual measured performance point in test conditions, represented by the red dot.

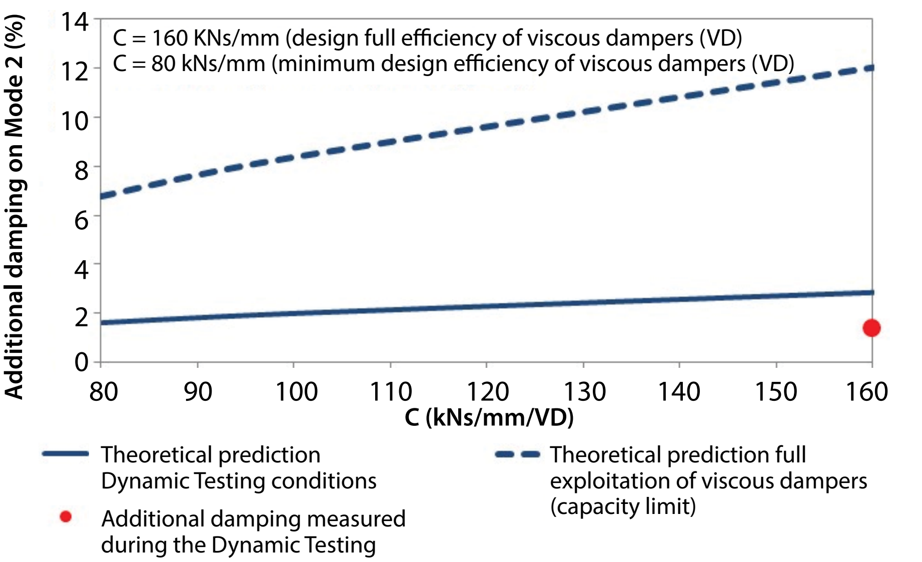

Figure 13. Plots of the theoretical expected behavior of the dampers.

Figure 12. Viscous damper installed at the base of the steel struts in the Allianz Tower.

By means of these measurements, an additional validation of the damping device was achieved. The devices had already undergone extensive laboratory tests according to the requirements of European Standards for fatigue and cyclic loading, but the OMA tests offered the unique opportunity to have an insight on their operational behavior once installed on site. ■

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