Title: **Recent Developments in Tall Buildings in Italy**

Authors: Franco Mola, CEO, ECSD
Laura Maria Pellegrini, Senior Structural Engineer, ECSD
Giuseppe Galassi Sconnocchia, Structural Engineer, ECSD
Elena Mola, CEO, ECSD

Subjects: Architectural/Design
Building Case Study
Structural Engineering
Sustainability/Green/Energy

Keywords: Structure
Sustainability
Urban Design
Vertical Urbanism

Publication Date: 2015

Original Publication: Global Interchanges: Resurgence of the Skyscraper City

Paper Type:
1. Book chapter/Part chapter
2. Journal paper
3. **Conference proceeding**
4. Unpublished conference paper
5. Magazine article
6. Unpublished

Recent Developments in Tall Buildings in Italy

Professor Franco Mola
CEO

ECSD,
Milan, Italy

Full Professor of 'Reinforced Concrete and Prestressed Concrete Structures' at the Civil Engineering Faculty of Politecnico di Milano, Italy. Licensed practice professional in Italy since 1971. In 2007 he founded ECSD S.r.l., an engineering consultancy office, of which he is the Owner and CEO. His research and design activity focus on the effects of the long-term behaviour of concrete in complex structures and on conceptual design, mostly of tall building. He is the structural designer of Palazzo Lombardia in Milan (161,3m tall), the New Torre Regione Piemonte Headquarters (207m tall) in Turin and the Isozaki Tower (202m tall) in Milan.

Laura Maria Pellegrini
Senior Structural Engineer

ECSD,
Milan, Italy

Degree in civil engineering (Major in structural engineering) from Politecnico di Milano in 2007. Registered Professional Engineer in Bergamo since 2007. Her academic career includes serving as teaching assistant for Professor Mola's course, 'Reinforced Concrete and Prestressed Concrete Structures', at the Faculty of Engineering of Politecnico di Milano. 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 team of Palazzo Lombardia in Milan and of the New Regione Piemonte Headquarters in Turin.

Giuseppe Galassi Sconocchia
Structural Engineer

ECSD,
Milan, Italy

Degree in Civil Engineering from Politecnico di Milano in 2012. Registered Professional Engineer in Milano. He first joined ECSD for an internship during his studies. He then joined ECSD as a junior engineer in 2013 and he's since been involved in the FE modeling and structural analysis of tall buildings. He was part of the design team of the New Torre Regione Piemonte in Turin, for which he served as structural analyst, with a focus on the analysis of long term effects on the global and local structural response.

Dr. Elena Mola
CEO

ECSD,
Milan, Italy

Degree in civil engineering from Politecnico di Milano in 2001. PhD in earthquake engineering from the INPG in Grenoble, with a thesis titled "Criteria for the seismic vulnerability reduction of existing irregular reinforced concrete structures", discussed in 2007. From 2002 to 2007 she worked as a grantholder at the Joint Research Centre of the European Commission, where she was involved in the analytical and experimental investigation of the seismic response of existing plan-wise irregular buildings by means of pseudodynamic testing. In 2007 she became a founding partner of ECSD, where she now serves as CEO and HR Manager.

Abstract

Major designs by internationally renowned architects, are currently under construction in Italy in the cities of Milan and Turin. In the decade 2005-2015, a dramatic change in the appearance of the cities occurred unveiling their modern European character. The resurgence of new heights is testified by important tall buildings such as Palazzo Lombardia, the CityLife and Garibaldi-Varesine developments in Milan, and the 'Intesa San Paolo' Tower and the New Regione Piemonte Headquarters in Turin. This building, designed by M. Fuksas, with its height of 207 m will be the tallest in Italy. Its structural design was carried out by the Author and it is the successful result of a series of choices and analyses for providing efficient solutions for resistance, durability, sustainability and constructability of the building. The synergy between architectural and structural design make it an outstanding example of a modern tall building exploiting state-of-the-art technology.

Keywords: Constructability; Robustness; Structure; Sustainability; Tall Buildings

Historical Overview

Northern Italy has witnessed gradual but radical changes in the urban habitat of its major cities over the past 50 years; one of the most interesting features of the new developments, especially in the last decade, are a number of tall buildings, designed by internationally acclaimed architects and currently under construction in Milan and in Turin.

Milan has a long history of towers, dating back hundreds of years. Even if the first modern tall building is the Pirelli Tower, designed by Giò Ponti in the 1960's. Starting from the Middle Ages, in the XII century, the ancient walls surrounding Milan consisted of 300 enemy sighting towers; all of them were destroyed by the Emperor Federico I during the siege that ended up with the conquest of Milan in 1163.

Thereafter, the higher spire of the Roman Catholic Cathedral of Milan, dating back to the same century, placed at a height of 111m, established the maximum building height in Milan. In the Modern Age, from the 1920s, a lot of tall buildings characterized by a significant architectural value were built in Milan: among them the so-called Torre Venetia, 50m tall, designed by G. Ponti

Figure 2. The new skyline of Milan (Source: ECSD)

Figure 1. The Pirelli and the Galfa Towers in Milan (Source: ECSD)

and built in 1930. He also designed the Park Tower, with the observatory placed at the limit height of 109m.

After World War II, some of the most famous tall buildings were built in the city, such as the Piazza Repubblica Tower, 117 m high, the Romana Tower, 89 m high, the Velasca Tower, 106 m high, the Galfa Tower, 109 m high and the Pirelli Tower, whose structural design is credited to the well-known engineer P.L.Nervi.

The Pirelli Tower, 127 m high, is worldwide renowned for its beauty and was, at that time, the tallest building in the world with a structural system totally in reinforced concrete. After the Sixties, no more buildings higher than 100m were built in Milan, so for a long time the landmarks of the city skyline were the shapes of the Pirelli Tower, Galfa Tower, Velasca Tower and the Cathedral. In Figure 1, the Pirelli and Galfa Tower are represented.

In the past decade, i.e. from 2005 to present, a dramatic change in the appearance of the city occurred, unveiling its new character of a modern European city. Responsible for this transformation are the new tall buildings, represented in Figure 2, including Palazzo Lombardia, 161.3 m high, designed by Pei Cobbs & Partners, NYC; the 'Garibaldi Varesine' development, signed by C. Pelli, S. Boeri, Khon Pederson Fox and the 'CityLife' masterplan, featuring three towers designed by architects D. Libeskind, Z. Hadid and A. Isozaki.

Some of these building gained international recognition for their architectural value: Palazzo Lombardia was awarded the "Best Tall Building Europe 2012" Award by the Council of Tall Building and Urban Habitat (CTBUH) and 'Bosco Verticale' (Vertical Forest) was awarded the "International Highrise Award" by the Museum of Architecture in Frankfurt. Such accolades not only recognize the beauty and efficiency of the architectural and structural design, but also the high performance levels in terms of energy and sustainability that these buildings are able to provide.

Almost all of the above mentioned buildings have a reinforced concrete structural system and their construction techniques have allowed the full exploitation of the material in terms of strength, durability and reliability.

In the same period, a similar trend towards new heights is clearly visible in Turin, which, even though more slowly than Milan, has been adding new landmarks to its skyline, historically characterized only by the Mole Antonelliana, 166m high, and some others interesting buildings of 1950s, 50-70 m high. The two new tall buildings currently being completed in Turin are the 'Banca Intesa' Tower, designed by R. Piano and the New Regione Piemonte Headquarters by M. Fuksas. This tall building, about to be completed, as represented in Figure 3, 4, 5 will set the new height record for Italy at 207 m. The Author was appointed with the structural design of the building, which will be presented in the following. The conceived structural system is

Figure 3. The Regione Piemonte Headquarters in Turin (Source: ECSD)

Figure 4. External view of the East Facade (Source: ECSD)

Figure 5. The interior view of the East Facade (Source: ECSD)

the successful result of a series of choices and analyses aimed at providing efficient solutions as for capacity, durability, sustainability and constructability of the building throughout its life cycle. The synergy between architectural and structural conceptual design behind this buildings are making it an outstanding example in Italy.

General Aspects Of The New Regione Piemonte Headquarters

The building has a square 45mx45m plan, is 207m tall, for a total of 43 storeys. The net interstorey height is 3m. The vertical structures consist in 4 central reinforced concrete cores and perimeter columns spacing 6m. The slabs span 6m on the North and South side and 12m on the West side. On the East Side the floor slabs consist of jump decks, with a cantilevered length of 10m. The jump decks are not connected to the East façade: a 2m wide gap in-between the slabs and the façade, extending for the whole height of the building, creates one of its main architectural features, i.e. an empty space inside the building, where stunning light effects can be enjoyed by the users.

On the West, North and South sides, the claddings are double glazed glass and aluminum frames; on the East side, the cladding is a self-supported, 45m wide, 180m tall 3D steel truss structure connected to the r.c. jump decks by means of a very limited number of pinned connections.

The design prerequisites for structural design focused on achieving full compliance with the architectural design, maximizing the net floor surfaces, increasing the robustness of the global structural system and of the connections in particular, controlling the interaction between structural and non-structural parts. Concrete was chosen for the structural system in order to maintain the rheological homogeneity in the material throughout the structure and to increase robustness of the column-to-slab joints. The global deformability of the structural system is so reduced, with a better performance in terms of comfort of users.

A peculiar feature of the structural system is the connection between the slabs and the cores, in particular the jump deck slabs, because of their cantilevered configuration. In the present case, if steel had been used for the slab beams, remarkable complications would have been faced during construction, due to the need of extensive welding, requiring specialized skills and a strict monitoring protocol, significantly increasing construction time.

For the above-mentioned reasons, concrete was chosen for all the structural elements, including the jump deck slabs. The extensive use of prestressing in slabs achieved high flexural efficiency and a reduction of slab thickness, which in turns has beneficial effects on the seismic response. Steel was used for the truss system supporting the East façade. The truss is self-standing as for vertical loads; for lateral load capacity, it is connected to the concrete slabs by means of a number of pinned connections, providing in-plane and out-of-plane restraints. The resulting global structural system is thus hybrid, exhibiting a complex behavior, caused by the rheological inhomogeneity of the materials. The interaction effects between the steel truss and the concrete main structure need to be carefully investigated in order to prevent damage to the glass cladding.

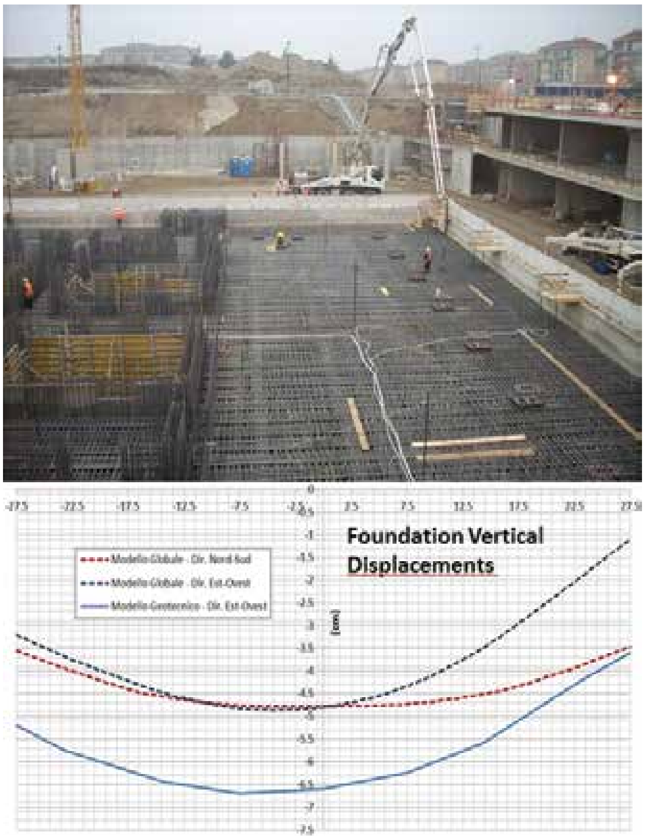


Figure 6. Foundation mat and vertical settlements (Source: ECSD)

The above-mentioned design prerequisites led to the choice of the correct mix design for concrete for the different structural elements, based on the desired performance, in terms of strength, workability and durability.

Self-compacting concrete was used for the foundation slab, columns and cores, whereas ordinary S5 concrete was used for slabs. As for strength, different classes ranging from C70/85 to C35/45 were employed. The main features of each structural element are briefly presented in the following.

Foundations

The foundation mat is a square slab measuring 55mx55m, with thickness ranging between 5m and 2.5m, resting on 64 friction piles, 50m long, each of them having diameter of 1.5m, symmetrically distributed.The soil showed a stratification consistent with type B with significant deformability. A careful investigation of the characteristics of the soil pointed out the presence of some sub-horizontal layers of clay, interspersed in the granular type soil. The possible effects the clay layers could

Figure 7. Slender columns with composite section (Source: ECSD)

generate were the magnification of the foundation settlements and the presence of local defects in the jet-grouting columns. For this reason, jet-grouting was used only as a barrier against water, and reinforced concrete bored piles were introduced in order to reduce the settlements. In this way, the non-homogeneous layers of soil were bypassed and the distribution of settlements, evaluated by means of specific analytical models, became more reliable. The final design solution was also validated by specific pile testing, conduct by means of Osterberg Cell technology. The foundation mat, consisting of about 12.000m3 of concrete, was poured continuously over 85 hours, using self-compacting concrete with a slump flow of 75/80cm (Concrete Society, 2005; De Schutter, 2008). In order to prevent cracking phenomena caused by temperature changes due to the heat of hydration of concrete, a low heat of hydration mix was used and during curing the slab was covered by a 5-6cm thick layer of water having a constant temperature of 15° (R. Springenschimid, 1998). The layer of water prevented excessive thermal gaps between the inner core of the slab and the surface, which would have induced heavy cracking. Water was removed after 15 days, when concrete had developed enough strength. The chosen mix design and the curing protocol allowed excellent results to be achieved: the cured concrete mass exhibited no visible cracks and appeared homogeneous and compact. In Figure 6, some details of the foundation mat and the plots of the monitored vertical settlements are reported and compared to those predicted by numerical analyses.

Columns

Columns have a 110cmx60cm rectangular section at all the levels, in order to comply with the architectural design and to uniformly exploit the floor surfaces. Only at the lower floors, columns have composite steel concrete sections, using ordinary steel profiles encased in reinforced concrete.

The use of composite sections at the base allowed an increase in capacity with no increase in the sectional area, a reduction of creep and shrinkage effects induced by the long term behavior of concrete and a increase in stiffness, needed in order to avoid buckling phenomena, since ground floor columns have a high slenderness ratio: the net interstorey height of the ground floor grand hall is about 17m and the columns are almost completely unrestrained against buckling, apart from the elastic restraint provided by the steel beams supporting the claddings, as shown in Figure 7. The analysis of these columns requires specific and refined procedures (see Timoshenko, 1960; Mola, 1993).

Starting from Level 06, the columns have reinforced concrete sections and starting from Level 26 lap splicing is achieved with no connectors, since the rebar diameters are smaller than 26mm. Concrete classes used for columns range from C70/85 to C35/45. The choice of the different classes was made to minimize differential shrinkage and creep effects between the columns and the cores: for the latter, lower strength concrete was used, ranging from C60/75 to C30/35.

Floor Slabs

In the key plan of a single floor, three different sectors can be distinguished, having different spans and supports.

On the west side of the building, the slabs span approximately 12m, they are supported by the cores on one side and by perimeter beams with a span of 6m, having the same thickness as the slabs. On the North and South sides of the building, the scheme is the same, but the slabs only span 6m. Finally, on the East Side of the building, the slabs are jump deck floors, jutting out of the cores for a length of 10m in a cantilevered structural arrangement.

The three different geometrical configurations required customized solutions and careful analysis investigation of the behavior of the slabs. In particular, in the corner areas, the two different structural arrangement coexist. The jump decks on the east side provide lateral restraints to the steel truss supporting the east façade, by means of steel connectors encased in the concrete slabs: for this reason, they are subjected to high in-plane stresses in the areas where the steel connectors are located. For the slabs on the West side, having 12m spans, the basic prerequisites

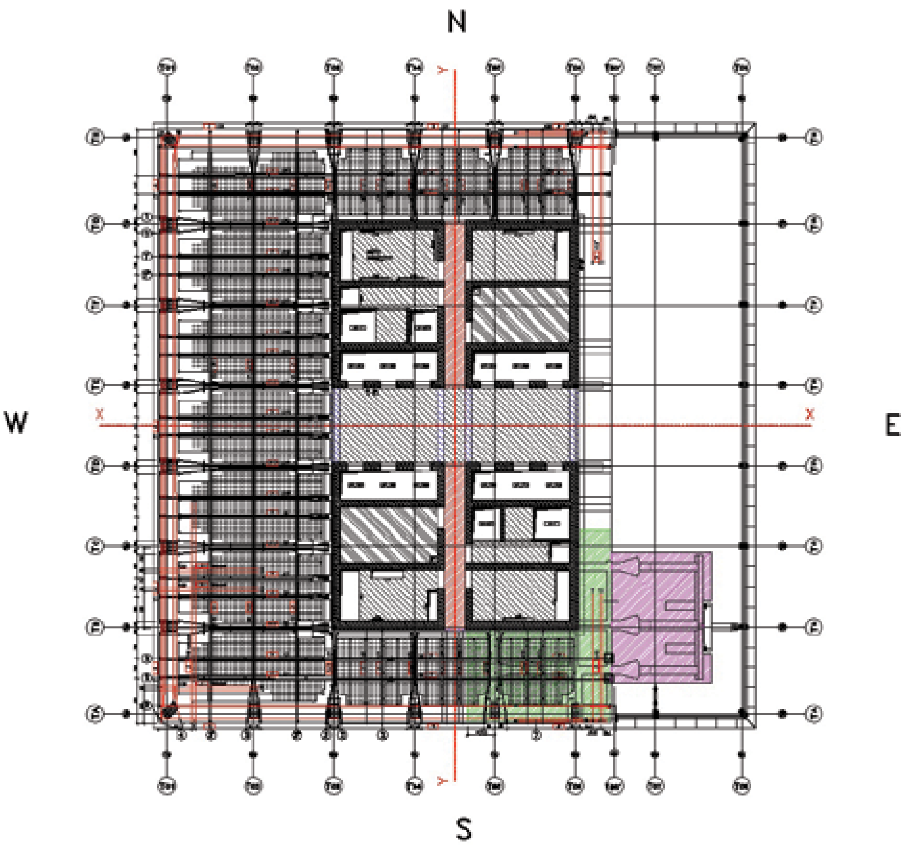


Figure 8. Structural arrangement of a typical floor (Source: ECSD)

Figure 9. The low weight post-tensioned slabs (Source: ECSD)

were reduced weight and thickness and limited deformability, whereas for the jump decks, a small deformability and an effective connection to the cores were mandatory.

The most stringent architectural feature was the limited thickness allowed for the slabs: only 43cm were allowed for the structural elements, on the North, South and West sides (see Figure 8). This limitation required a careful design of the slabs spanning 12m, due to their significant slenderness: without the use of prestressing, T-shaped beams would have been required for deformability limitations. The choice of T beams, though, is rarely advisable because it creates complications in the design of the plants layouts, in the formwork and scaffolding systems and in the pouring of concrete. In order to avoid T-beams, a prestressed reduced-weight slab was used (Figure 9). Unbonded tendons were used for prestressing in both directions and plastic spheres were used to reduce the slab weight: this technology, which is more and more used in Italy and worldwide, allowed all the design prerequisites to be complied with, reducing the thickness of the slabs to 34cm, (Lin, 1982). The same technology was used for the North and South sides, even if the spans were lower, in order to achieve uniformity and to simplify the design of formworks. The jump deck floors, having a 10m span cantilevered T-beams, directly restrained into the cores and supporting a 20cm thick concrete slab. For the beams, bonded prestressing cables were used (see Figure 10). The proposed design solutions achieved very good performance levels: the experimental tests carried out on the slabs during construction, both static and dynamic load tests, confirmed that a very efficient structural arrangement had been achieved. The use of light-weight polyethylene spheres coupled to unbonded strands for prestressing is one of the most interesting features of the slabs, due to their outstanding performance in terms of capacity and deformability. The use of the spheres, having a diameter of 22.5cm, allowed a reduction in weight of the order of 20-25% with respect to that of a reinforced concrete slab. The reduction in storey weight and mass is beneficial both because it reduces earthquake induced storey shear forces and reduces axial loads in columns and vertical settlements of the foundation mat.

This technology also has beneficial effects on the construction process, cutting construction times significantly. In fact, the bubble decks are built by means of pre-assembled, self-supporting elements, in which the longitudinal and transverse rebar and the plastic spheres are pre-arranged, which are then lifted and set in place, ready for concrete pouring. Every single element has a very reduced weight, so that limited crane lifting capacity and time are required for the installation.

The use of prestressing allowed a reduction of the slab thickness, with further reduction of slab weight and mass and achieving a high slenderness ratio. Because of the structural arrangement of the building, and the way the connection between the cores and the slabs was made, prestressing did not induce significant static effects and prestressing losses were very limited. No provisional joints were necessary in order to prestress the slabs and to prevent long-term effects, which cut construction time down to one slab per week. The east side cantilevered slabs, on the other hand, required provisional joints in the beams in order to make prestressing effective. In fact, because the beams are cantilevered, the slenderness ratio must be reduced with respect to a simply supported configuration; moreover, the cantilevered beams supporting the decks had to be located in correspondence of the inner walls of the cores, so that a more robust connection could be achieved. Because these inner walls have a spacing of approximately 4m, the cantilevered beams have the same spacing and a depth of 95cm, whereas the slab in-between is 20cm thick. The beams are post-tensioned by means of cables, the 20cm thick slabs are r.c. concrete only: the use of prestressing in the slabs is only limited to the areas where the steel connectors restraining the east façade are located, because of the high in-plane stresses induced by prestressing. Because the beams are cantilevers, the maximum effectiveness of prestressing must be guaranteed at the connection with the core walls: this is mandatory because the core walls and the beams are poured at different time, so that prestressing is the only active shear and flexural connection between the two adjacent concrete sections. The number of strands in each beam is computed in order to achieve zero tension stresses in the connecting section. Shear action is counterbalanced by the friction provided by prestressing.

Figure 10. The post-tensioned jump-deck slabs floors (Source: ECSD)

The East Façade Steel Truss System

The spatial steel truss supporting the self-standing east façade is approximately 45m wide and 180m tall. The vertical elements are hollow core rectangular profiles having thickness ranging from 15mm to 30mm, with 6m spacing. The horizontal elements are main beams, vertically spaced by 4 floors, and secondary beams, consisting of rectangular hollow core profiles. The façade is connected to the cantilevered slabs by means of three different kinds of restrains: restraints for out-of-plane deflection, and restraints for in-plane displacement-control, both along the North-South direction and the East-West direction. The restraints for out-of-plane displacement control induce high axial stresses in the 20cm thick slab of the jump decks, so that local prestressing was necessary to provide adequate capacity. The restraints limiting the North-South displacements induce high shear and flexural effects into the light weight slabs, so that an extensive use of prestressing was necessary to counterbalance the actions induced by the restraints. Finally, the relative displacements between the façade and the r.c. elements in the vertical direction needed to be accommodated by the connectors, since due to thermal effects, differential displacements in the order of 8cm are expected to take place. Such large differential displacements would induce unacceptable stresses in the structural elements in presence of a vertical restraint, so that a vertically sliding, low-friction restraint was used. Since the steel elements acting as restraints and connecting the east façade to the r.c. structures are visible through the façade, they can be considered an architecturally relevant feature. For this reason, their arrangement, their location and their geometrical dimensions were carefully designed and optimized, so that their impact on the façade would be minimized. In order to come up with the dimensions of the steel elements of the truss, different loading conditions and displacement compatibility conditions needed to be taken into account: first of all, the truss is restrained to the r.c. elements, which in fact are not fixed, since they do displace due to lateral and vertical loads. One loading condition thus consists

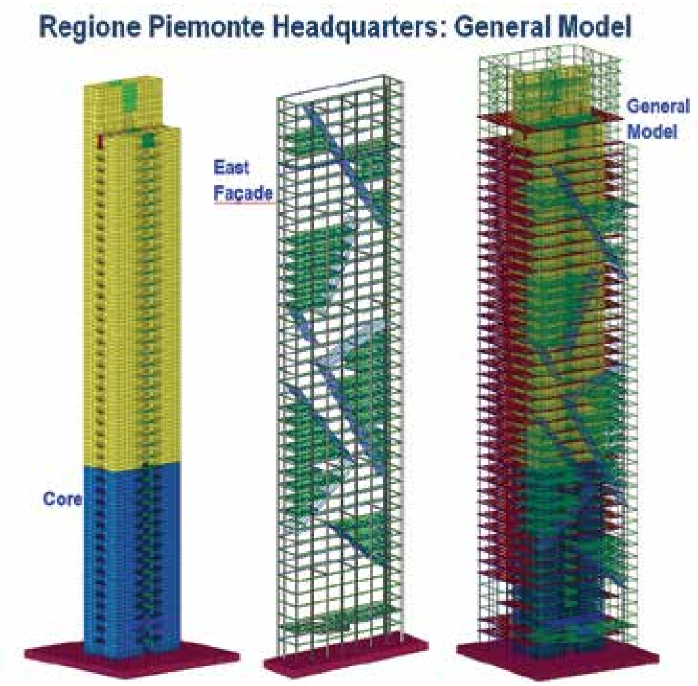


Figure 11. Model for structural analysis (Source: ECSD)

on imposed deformations at the restraints. Moreover, directly applied wind loads need to be taken into account, because the glazing elements are subjected to direct wind pressure. The hybrid system consisting of the r.c. parts and the truss elements thus becomes a complex interactive structure, requiring very careful analytical and numerical models to guarantee full displacement compatibility.

Structural Analysis

The peculiar features of the building and the complexity of its structural system, consisting of very diverse elements interacting together, required a number of different analytical and numerical models to be implemented in order to reliably predict the response under static and dynamic loads and to evaluate the performance of the buildings for different limit

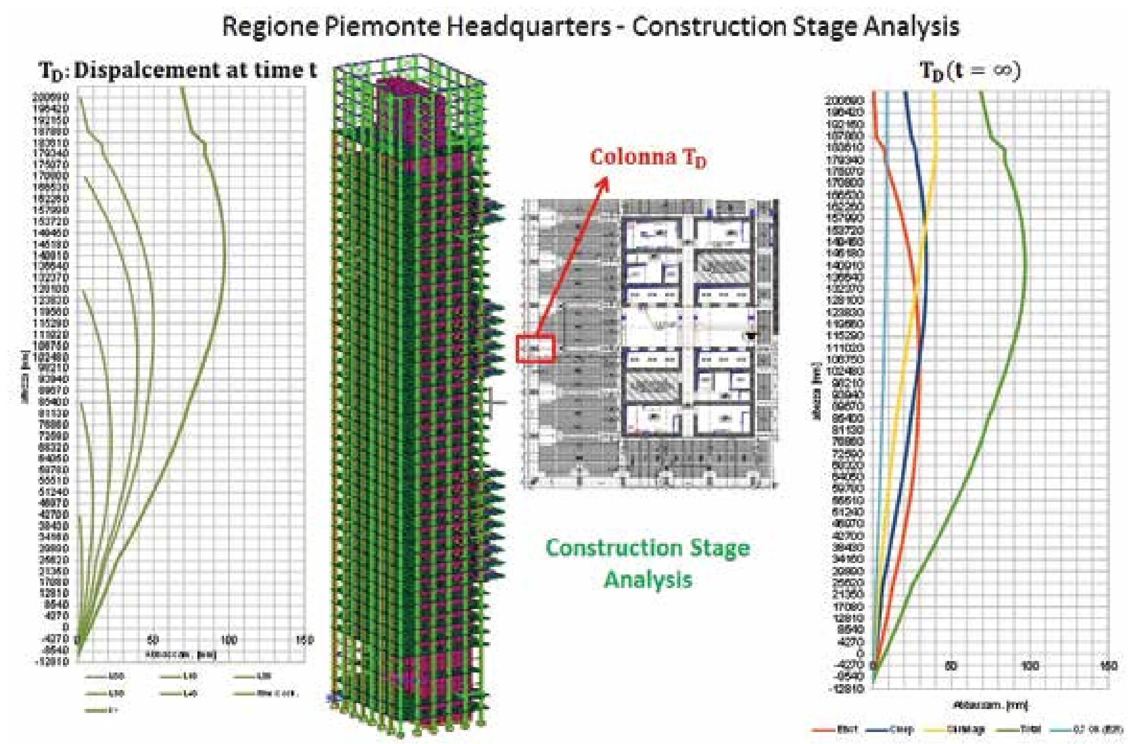


Figure 12. Construction stage analysis and column shortening (Source: ECSD)

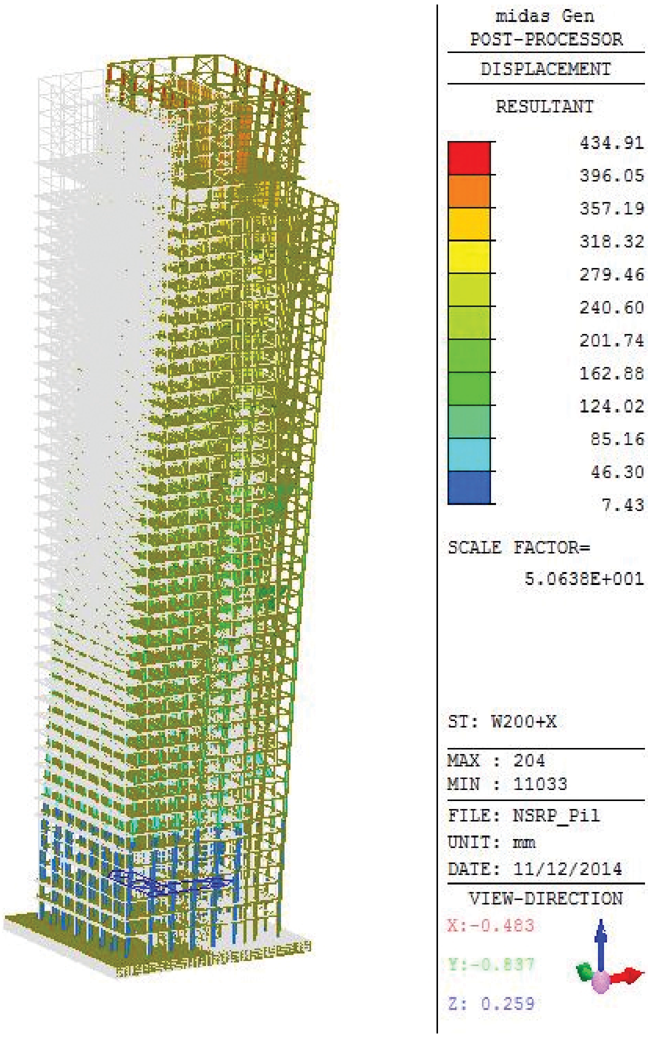


Figure 13. Model for wind analysis (Source: ECSD)

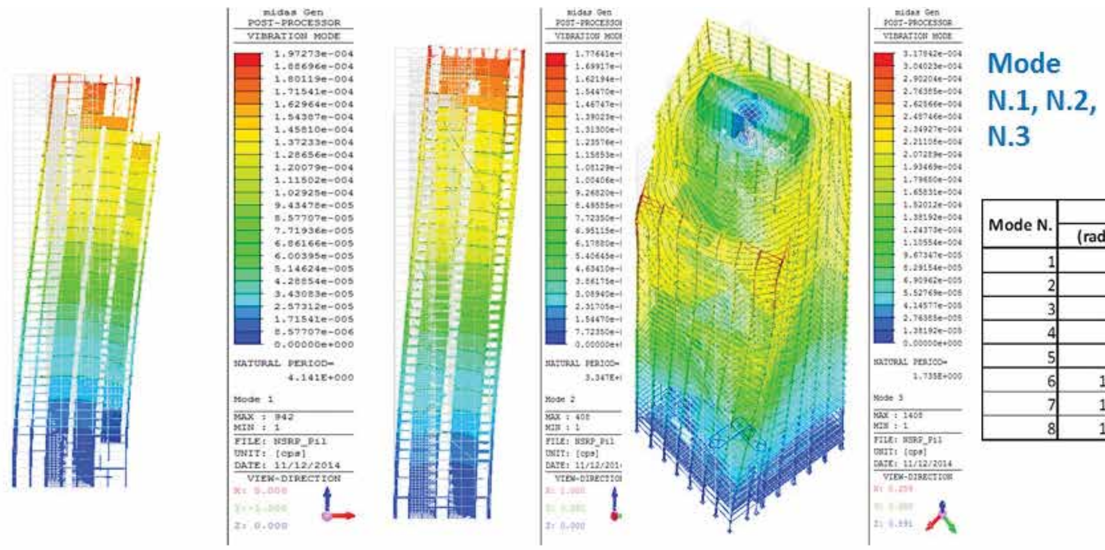
states. A general main finite element model was implemented (see Figure 11). Firstly it was used for the analysis of the effects of vertical loads only, taking into account the effects of the long-term behavior of concrete. A construction stage analysis with displacement compensation was carried out. The most significant feature of this analysis is the correct prediction of the time evolution of the stress patterns in the elements and the accurate prediction of the relative vertical displacements between cores and columns. This problem was pointed out by (Fintel, 1965; Fintel, 1971) and recently has been approached in a general way by (Mola, 2010). Local models were implemented for the analysis of the slabs, taking into account the effects of prestressing and of vertical loads. For the light-weight slabs including polyethylene spheres, specific experimental tests have been carried out in recent years in order to calibrate adequate models to predict their behavior.

The maximum differential deformation between pairs of columns and the columns and the cores is 2cm (see Figure 12). A maximum variation of 4% can be observed in the axial load in columns, which confirms that the columns mostly behave as pinned elements and that the bending action in the slabs is practically unaffected by the differential displacements between the columns and the cores, (Mola, 2014).

Wind loads were derived from wind tunnel tests, taking into account directionality effects, and used both for the dimensioning of the lateral load resisting systems and for the definition of the pressure loads on the glass façade, according to a risk-consistent approach, were carried out by Dr. Chiara Pozzuoli, whose work was highly appreciated (Pozzuoli, 2014).

In Figure 13, the results derived from a linear elastic analysis carried out on a global FE model are reporting, showing a maximum wind-induced displacement of 28 cm.

Earthquake actions were quantified according to EC8 (UNI EN 1991-1, 2005): a linear response spectrum analysis was carried out according to (Cigada, 2012). The eigenvalue analysis computed on the global model yielded the modal



Mode N.	Frequenza (rad/sec)	Frequenza (cycle/sec)	Periodo (sec)
1	1.5349	0.2443	4.0935
2	1.8908	0.3009	3.3231
3	3.7540	0.5975	1.6737
4	7.2440	1.1529	0.8674
5	7.3595	1.1713	0.8538
6	11.8669	1.8887	0.5295
7	16.3933	2.6091	0.3833
8	17.4710	2.7806	0.3596

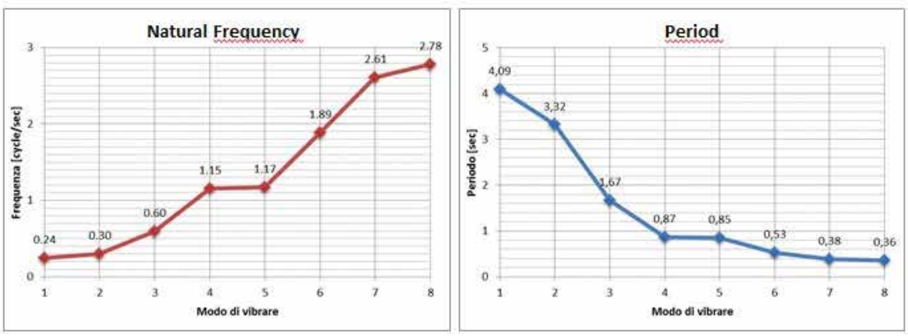


Figure 14. Modal analysis results (Source: ECSD)

frequencies and mode shapes reported in Figure 14. Due to the low seismicity of the site and the flexibility of the structure, it was found out that wind induced actions are higher than earthquake induced ones, Finally, a separate model was implemented for the steel truss supporting the East Façade, assuming fixed restraints at the location of the connectors with the r.c. structure. This model was used for linear elastic analyses: at first, the effects of imposed displacements of the restraints, corresponding to the displacements of the r.c. structure derived from the analysis, were investigated. Moreover, the effects of wind loads and vertical loads pertaining to the steel truss were evaluated. The basic analysis assumption was that the interaction between the two sub-structures would be very limited, due to the practically negligible stiffness of the truss with respect to that of the r.c. building. The two sub-structures can thus be analyzed independently, by applying imposed deformations at the restraints. In Figure 15, the deformed shape of the truss is reported.

Conclusions

Over the past decade, tall buildings have become increasingly common in Italy: the height record set by Grattacielo Pirelli in the 1960's, i.e. 127m, was unbeaten for over 4 decades, but it has recently been surpassed at first by Palazzo Lombardia in 2011, followed by the New Seat for Regione Piemonte, setting the new height record for Italy at 207m in 2015.

These recently completed tall buildings have undoubtedly changed the skyline of Milan and Turin, becoming new landmarks for both cities, in addition to the traditional heritage buildings that have made them famous all over the world, such as the Cathedral in Milan or the Mole Antonelliana in Turin.

In the New Torre Regione Piemonte Headquarters, an innovative solution for slabs was introduced: the use of prestressed, light-weight, 'bubble' slabs allowed a significant reduction of weight, mass and thickness of the slabs and a remarkable reduction in construction time. The use of bubble decks coupled with prestressing allowed a drastic increase of the static efficiency and robustness of the structural system. The complex architectural features of the building called for a customized and carefully optimized structural design of all the parts of the structural system, coupled with state-of-the-art construction techniques in order to guarantee the required performance levels and to preserve the architectural beauty of the design.

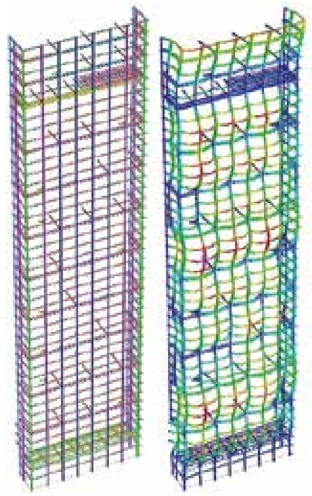


Figure 15. The model for the East Facade (Source: ECSD)

References:

Concrete Society (2005). **Self Compacting Concrete - A Review**. Tech. Rep. n°62, London, 2005.

De Schutter, G., Bartos, P.J.M., Domone, P., Gibbs, J. (2008). **Self Compacting Concrete**. Whittles Publishing, CRC Press Taylor & Francis Group, UK, 2008.

R. Springenschimid, Ed. (1998). **Prevention of Thermal Cracking in Concrete at Early Ages**. RILEM Report 5.

Fintel, M., Khan, F. R. (1965). **Effect of Column Creep and Shrinkage in Tall Structures** – Prediction of Inelastic Column Shortening. ACI J., 1965.

Fintel, M., Khan, F. R. (1971). **Effect of Column Creep and Shrinkage in Tall Structures** – Analysis for Differential Shortening of Columns and Field Observation of Structures. ACI SP 27-4, 1971.

Mola, F., Pellegrini, L.M. (2010). **Effects of Column Shortening in Tall R.C. Buildings.** Proceedings of the 35th OWICS Conference, Singapore, 2010.

Mola, F., Pellegrini, L.M., Stefopoulos, G., D'Agostino, G. (2014). **Structural Analysis and Design Issues in Tall Buildings in Italy: General Aspects**. Proceedings of the 39th OWICS Conference, Singapore, 2014.

CUR (2001). **CUR-Recommendation 86 'Bubbledeck floors'**. December 2001, CUR, Gouda, The Netherlands.

Lin, T.Y., Burns, N.H. (1982). **Design of Prestressed Concrete Structures**. John Wiley & Sons, New York, 1982.

Pozzuoli C., Mola F., Segato C. (2014). **Wind directionality effects on cladding pressures for the design of the Isozaki Tower in Milan. Proc. of Int. 6th Int. Symposium on Computational Wind Engineering**, CWE 2014, Hamburg, Germany,2014.

Pozzuoli C., Mola F., Segato C. (2014). **Risk-consistent cladding pressures on the Isozaki tower in Milan.** Proc. of the 13th Conf. of the Italian Association for Wind Engineering, IN-VENTO 2014, Genoa, Italy, 2014.

Cigada, A., Mola, E., Mola, F., Stella, G., Vanali, M., (2012). **Modal Analysis of the Palazzo Lombardia Tower in Milano**. Proc. of IMAC Int. Conference, Florida, USA, 2012.

Timoshenko, S.P., Gere, J.M. (1960). **Theory of Elastic Stability**. Mc Graw Hill, N.Y., 1960.

Mola, F. (1993). **Influence of Elastic or Visco-Elastic Restraints in Preventing Long-term Second Order Effects in Slender R.C. Columns**. Proceedings of the 18th OWICS Conference, Singapore, 1993.

UNI EN 1991-1 (2005). **Eurocode 8: Design of Structures for Earthquake Resistance. Part 1: General Rules, seismic actions and rules for buildings**. UNI EN 1998-1:2005.