STRUCTURAL PERFORMANCE OF REINFORCED CONCRETE SKYSCRAPERS OF THE 1950’s AND 1960’s

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Abstract. The preservation of reinforced concrete skyscrapers built in the Fifties and Sixties of the 20th century is currently an important consideration for building owners. Toward the end of the century, the most radical intervention of all, demolition, was viewed with growing favor in a number of European countries, particularly in France, Holland, Great Britain and Germany, as well as in the United States. This approach has over the years been supported in the name of modernity, urban beauty and common interest. Some famous contemporary monuments have only been saved from the risk of demolition thanks to the intervention of the State Authorities responsible for protecting architectural heritage. Considering the UNESCO definition of “cultural heritage”, many reinforced concrete structures, such as high-rise buildings, bridges or dams built during the 19th and 20th century, may, indeed, be considered as modern heritage structures due to their outstanding universal value from the point of view of history, art and science. Despite their importance, these buildings may lack in maintenance or require rehabilitation and adaptation to modern regulations, particularly in terms of structural safety. The work presented here analyzes the structural performance of some iconic tall buildings in reinforced concrete with respect to current design code requirements. The buildings chosen for case studies are the Pirelli building, the Velasca tower in Milano, Italy and the Marina City towers in Chicago, Illinois, USA. The Pirelli and Velasca towers have been for very long among the tallest buildings in Italy and still have a high symbolic value for their architectural and engineering points of view. The Marina City towers, at the time of their construction, were the highest residential reinforced concrete buildings in the world. The case studies presented herein show that the structural performance of these tall buildings is in agreement with current design practices.
1 INTRODUCTION

The main issues in the preservation of historical reinforced concrete structures are related to their current structural performance. These non-ductile reinforced concrete high-rise buildings were built according to different wind design criteria and without reference to any seismic design code. Considering the definition of “cultural heritage” by the World Heritage Convention (UNESCO, 1972), many reinforced concrete structures, such as residential and industrial buildings, bridges or dams built during the 19th and 20th century, may be considered as modern heritage structures for their outstanding universal value from a historical, artistic or scientific points of view. Despite their importance, these buildings may lack in maintenance or require a re-use and adaptation to the modern design codes (including the considerations for seismic loads). The aim of this paper is to determine whether iconic skyscrapers of the Fifties have a structural performance that is consistent with new reinforced concrete buildings of similar height, and in agreement with current design practices.

2 REINFORCED CONCRETE HIGH RISE BUILDINGS BETWEEN 1950’s AND 1960’s

In this research, the reference design codes of the period, the typologies and mechanical properties of the materials used in the construction process of the time as well as the possible consequences of decay on the structural response have been studied. In this context, the case of the first reinforced concrete high-rise buildings, dating back to the 1950’s and reaching a height of 30 stories (built using normal strength concrete) is of particular interest. Such buildings were simultaneously constructed in Europe, North-America and in Brazil. Their design was based on manual calculations, with an allowable stress approach without consideration for the ductility and without any requirement for potential seismic loads [1].

In regard to this argument, two reinforced concrete high-rise buildings constructed in Milano, Italy, at that time: the Pirelli building and the Velasca tower are mentioned. They present two different structural schemes, which are both now recognized as very effective with respect to seismic loads. The design was considered innovative at the time, which included using: a shear wall system for the Pirelli Building, and a tube-in-tube system for the Velasca Tower. The latter is one of the first applications of the tubular concept.

According to Schueller [2], the notion that concrete is only for fire protection changed after World War 2, when advanced construction techniques together with the development of high quality materials began to enter into new design concepts such as the flat slab and the load bearing façade grid walls. Both systems were challenging the traditional one-way slab and curtain wall, which were typical of rigid frame structures. Skyscrapers such as the 65-story Marina City Towers (Chicago, 1959 - 1967) truly express the monolithic sculptural nature of concrete and represent the first attempt in the U.S. to explore a new structural layout composed by a rigid core in concrete with reinforced concrete columns all around¹. This, introduced by the architect Bertrand Goldberg, allowed structures to reach a height of 179 meters.

The re-analysis of these high-rise buildings, performed with current computational means and with reference to present code requirements, is presented in this work, together with some insight into the performance of historical reinforced concrete buildings.

¹ Goldberg utilized the same conceptual design also of the Astor Tower Hotel (1958-63) in Chicago, reaching an height of 91 m (199 ft) with 25 story.
3 THE VELASCA TOWER

The Velasca Tower was built in 1956-58 and has been protected as a cultural heritage building since 2011. The architects were G.L. Banfi, L. Belgioioso, E. Peressutti, and N. Rogers of BBPR Studio. This architectural office, after WWII, was one of the most important design firms in Italy. The architects developed innovative projects that were nevertheless considered traditional as opposed to “international style.” The special feature of the Tower, i.e., its larger top, is due to urban as well as historical reasons, making it distinct in a landscape crowded by low-rise buildings. The structural system is made up of concrete walls and stairs and elevators block that are located in the central area. However, the concrete frames are located at the façades. This arrangement results in a lattice envelope system for the entire building. The structure is one of the first applications of the tube-in-tube scheme, which later impacted tall building design in the USA. Moreover, the Tower is one of the first reinforced concrete buildings reaching a height of 106 m. The structure was designed by Arturo Danusso, an experienced engineer and professor at Politecnico di Milano with special interests in the development of structural dynamics [3].

![Figure 1. Left: Drawing of The Velasca Tower. Right: Building plan in the expansion area.](image)

3.1 Building layout

The building has 29 stories, two of which are underground. It is a multi-use building, with shops at the base, offices to 10th, mixed use to 18th (which is the base of the enlargement), and the rest of floors are for residential use. The plan in figure 1 shows the continuous concrete walls of the inner block and the outer frames. The layout with its double symmetry is effective for seismic resistance. Traditional brick infill walls were used inside and precast panels outside. The inclined struts at the top expansion apply compression to the lower floor where they connect, and tension at the upper one. Beams with pre-tensioned cables have been used in their tension zone, with thicker slabs used in the compression zone.

Refurbishing interventions concerning façade elements are now in progress according to a maintenance plan. Per accounts by engineers in charge of the current interventions, there is no significant decay in structural elements; and the conditions of reinforcement being still of full protection by a well-conserved and efficient concrete cover. Information on design issues is available from publications of the time. Furthermore, the material properties were determined from tests conducted at the Politecnico Laboratory. The authors did have an access to design drawings.
3.2 Modeling and analysis

From the available documents, an accurate estimate of the dead load was performed. The weight of the building amounts to approximately 400 000 kN, divided as follows: concrete contributes for 65.5%, brickwork for slabs 3%, steel 2.5%, infills and cementitious decorative apparatuses 11.5%, finishing layers, floors, etc. 7.5%, and various additional overloads for 10%. Columns carry an axial load of over 10 000 kN with a cross-section of 1.1 m². Concrete works at about 8 MPa, a high stress level that was possible according to tests. In all the tests conducted, the limiting resistance was over 30 MPa. The load analysis and the detailed information on the building geometry permitted the development of a structural model suitable for modal analysis [4].

Figure 2. Model of the Tower for structural analysis and detail of the expansion zone.

Modal analysis has pointed out the symmetry of the structure in plan as well as in elevation, deriving from an accurate distribution of stiffness. As a consequence, the increase of mass in the upper part has little effect on the global response. Table 1 shows results and points out the good response agreement in the two main horizontal directions (X is the longer side), which have very similar periods and, thus, stiffness. The first three mode shapes are shown graphically in Fig. 3.

Table 1. Results of modal analysis

<table>
<thead>
<tr>
<th>Mode</th>
<th>Period (s)</th>
<th>X-direction</th>
<th>Y-direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Modal mass [%]</td>
<td>Total mass [%]</td>
<td>Modal mass [%]</td>
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</tr>
<tr>
<td>2</td>
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<td>0.00</td>
<td>67.53</td>
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<td>1.90</td>
<td>0.04</td>
<td>67.57</td>
</tr>
<tr>
<td>4</td>
<td>0.64</td>
<td>17.37</td>
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</tr>
<tr>
<td>5</td>
<td>0.60</td>
<td>0.00</td>
<td>84.94</td>
</tr>
<tr>
<td>6</td>
<td>0.51</td>
<td>0.31</td>
<td>85.25</td>
</tr>
</tbody>
</table>

The response spectrum analysis was conducted according to the present design code. The effects of 1) the wind action of the original project, 2) the wind action now required, and 3) the seismic action have been considered according to current required load combinations from the Italian National Code, corresponding to Eurocode 8. The wind action today is only slightly higher than that at the design time. For the earthquake, the area is of moderate to low seismicity: the local design spectrum with a 475-year return period has been considered with a
behavior (reduction) factor of 2. This value, quite below code assumptions for existing buildings, has been assumed because of the lack of information on the reinforcement details. The maximum spectral acceleration was just below 0.1g.

![Figure 3. Mode shapes, from left: first, X-direction, second, Y-direction, and third mode.](image)

For wind analysis, the wind was applied in the least favorable direction, with consideration for torsional effects from non uniform pressure in plan. Wind, reaching moderate maxima in the area, demands higher resistance. Table 2 presents the maximum base shears and top sways for wind in the worse direction and for earthquake in each principal direction, comprising the contribution of the orthogonal one according to the code.

<table>
<thead>
<tr>
<th></th>
<th>Base shear X-direction [kN]</th>
<th>Base shear Y-direction [kN]</th>
<th>Max. top sway [mm]</th>
</tr>
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<tr>
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<td>-</td>
<td>65430</td>
<td>67</td>
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<tr>
<td>Earthquake, X-direction (main)</td>
<td>44510</td>
<td>14490</td>
<td>44.5</td>
</tr>
<tr>
<td>Earthquake, Y-direction (main)</td>
<td>48310</td>
<td>13350</td>
<td>45.7</td>
</tr>
</tbody>
</table>

4 THE PIRELLI BUILDING

The Pirelli building, known also as “the Pirelli skyscraper”, was built between 1956 and 1961 for the Pirelli rubber industry. In some ways, it symbolizes the vital and energetic post-war industrial character of the region. Several prominent architects and engineers worked at the project. The architectural design is by Gio Ponti, the slender and elegant structure was designed by Pierluigi Nervi and Arturo Danusso [5]. The building is 33 stories high and stands close to the railway station. The plan is oval-shaped and accommodates elevators at the core. According to William J. R. Curtis², “the result was a unique prestige office building which represented the high technical aspirations of the company; it showed that not all high-rise designs in Europe had to ape the American models. The Pirelli Building was witness to a new

era of industrial design in Italy, also evident in the products and buildings of the Olivetti Company, which emerged as a major patron, as it had in 1930’s”.

4.1 Building layout

The characteristics elongated lentil-shape of the building, 70.40 m long and 18.5 m wide, created several structural analysis problems because of the height (about 127 m). At the time of construction, it was the tallest reinforced concrete building in Europe. Its plan dimensions account for its slenderness, which posed stability and structural analysis problems. The 18.5 m across for a height of 127m was an unusual ratio in high-rise buildings. Figure 4 shows the building outline and the plan, with the characteristic tapered extremes, at the tips. The structural analysis challenges posed by geometry resulted in a mixed structure, where the area of the tips presents stiff boxed vertical structures that contained also stairs and elevators. Additional load bearing members included 4 concrete walls positioned two by two transversally at a distance of 24 m from one another. The walls actually had openings that permitted the use of the floor as open space.

The design of the structure is a combination of two different systems: an elastic one, in which the stability was assured by the bearing capacity of multiple frames to absorb horizontal actions, and a “gravity” one in which the vertical elements do not collaborate with the horizontal ones and behave as isolated walls.

Figure 4. Left: Pirelli building plan. Right: “Tip” structural detail.

Fig. 5. Left: photographic image of the Pirelli building by night. Center: Elevation of the Pirelli building. Right: mathematical model of the structure.
Contrary to the Velasca case, reinforcement and details were available, together with an adequate information on the materials properties at the time of construction. The Politecnico Laboratory performed the testing of concrete and steel. The two types of concrete used had an average limit resistance of 26.9 and 38.1 MPa, respectively. Tests on steel specimens also provided information on the ductility of materials. More recently, additional tests were carried out, in order to control the state of the structure and possibly plan a remedial maintenance action. The meticulous design and care of the structural engineer may be appreciated by the fact that rebars were not overlapped, but joined with threading and a cuff. First forms of deformed bars were also used to improve adherence.

The decay of the concrete cover, generally modest, was measured at various locations. The building underwent some rehabilitation work in 2002, after a small airplane crashed into it in October 2001.

4.2 Modeling and analysis

A numerical model of the building has been developed in order to analyze it according to current design codes. Modal analysis indicated a fundamental period of 2.84 s corresponding to a bending mode in its short direction. Similar to the case of the Velasca tower, modes are either translational or torsional, with longer periods in the lateral direction as expected. The period is in full agreement with approximate formulas in Eurocode 8 (2.95 s) and UBC (2.81), but shorter than the one obtained from the scaled model. The higher deformability of the physical model is likely due to a more accurate modeling of soil effects.

<table>
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<th>Mode</th>
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<th>Mx [%]</th>
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<td>0.00</td>
<td>6.33</td>
</tr>
<tr>
<td>10</td>
<td>0.275</td>
<td>4.63</td>
<td>0.02</td>
</tr>
</tbody>
</table>

A response spectrum analysis was carried out with a code-suggested value of 3.6 for the behavior factor. All the main elements survived the actions resulting from the seismic load combinations. Subsequently, nonlinear push-over analyses were carried out, indicating as appropriate the lower value of the factor close to 2. Spectrum analyses were repeated with the updated reduction factors. Satisfactory results were confirmed.

At the time of structural design (1955-56), as traditional computational methods were perceived as insufficient to guarantee the safety of a tall structure with the slenderness characteristics of the Pirelli, a scaled model (1:15) was developed and tested at ISMETS Laboratory (a facility in Bergamo specialized in modeling and testing large structures). The aim was to understand the behavior of the building subjected to different wind loads.
The nonlinear push-over analyses, carried out for different load distributions, permitted to investigate the seismic behavior for increasing seismic action, which could not be modeled in the linear spectrum case. The coupling beams in the shear walls, which were not designed according to modern criteria, appear to enter the post-elastic field fairly early, for a PGA just below 0.6 g. The mode of response changes with the vertical elements connected only by stiff floor slabs; yet the very high capacity of the system permits to counteract high acceleration values. The ultimate value according to the analysis was 0.77 g.

5 THE MARINA CITY TOWERS

Marina City’s apartment towers, when completed, were the tallest concrete and residential structures in the world, with clearly expressed curvilinear concrete balconies. Goldberg described each as a “tremendous tree trunk”.

The curves of the balcony floors, flowing smoothly into vertical columns, represent a seamless transition between an integration of architectural and engineering decisions.

Goldberg thought that rectilinear housing projects were depressing. He said: “I wanted to get people out of boxes, which were psychological slums”, adding “those long hallways with scores of doors opening anonymously are inhuman. Each person should retain his own relation to the core. It should be the relation of the branch to the tree, rather than that of cell to the honeycomb”.

Goldberg’s rupture with orthogonal steel architecture (which was inherited from the Bauhaus and was then making a big hit in the United States before spreading across Europe and the whole world as a kind of symbol of modernity) finds its place in a group of experiments that were to metamorphose the esthetics of reinforced concrete, which had remained dependent for too long on the post and beam, while also blowing a breath of lyricism and a completely new poetic into contemporary architecture.

From the circular central core, Marina City apartment plans radiate in the form of sixteen “petals”. An efficiency apartment occupies one petal, a one-bedroom unit occupies a petal...

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Figure 6. Main Pirelli modal shapes.

4 Goldberg’s experiments with circular form paralleled the work of other architects. A relevant precedent is constituted by Buckminster Fuller’s four dimensional tower (1928, unbuilt), featuring a prominent central circular core with cantilevered floor plates; it anticipated the form and logic of Marina City. In his diagrams he showed the aerodynamics advantages of circular form.
and a half, while a two-bedroom unit occupies two and a half petals. The bathroom and the kitchen are close to the core, while living and sleeping areas extend to the balcony. This reduces utility distribution lines and locates darker areas inward, while opening living and sleeping quarters to the sun [6].

Figure 7. Left: image of Marina City (courtesy of the archive of the Art Institute of Chicago). Center: Marina City elevation (courtesy of the archive of the Art Institute of Chicago). Right: Marina City apartment layout (courtesy of the archive of the Art Institute of Chicago).

5.1 Building layout

Each tower’s structure is made up of a rigid frame and core. The rigid frame responds to lateral loads primarily through the behavior of the beams and columns. This type of behavior results in large lateral drift for the buildings. However, introducing a core structure, the lateral resistance of the building increased as a result of the core and frame interaction.

The central core is structural and it has been designed to resist mainly the wind. According to Goldberg: “The shape of the core means that the buildings have only 30% of the wind resistance that they would otherwise have with the same dimension, but in rectilinear form”. Each core is 180 m (588 ft) high and has an inside diameter of 9.7 m (32 ft). Its walls vary in thickness from 76.2 cm (30 in) at the base to 30.5 cm (12 in) at the top and take the main transverse load of the building.

Outside the core there are two rings of 16 columns each which carry most of the vertical load. The inner ring of rectangular columns has a diameter of 14 m (47 ft); an outer ring of diamond-shaped columns has a diameter of 33 m (109 ft). Sixteen radial beams span from the interior core across the inner columns to the outer ones.

Ramps or balconies extend 3 m (10 ft) beyond the exterior columns for a total outside diameter of 39 m (128 ft). The foundation system goes down to 33.5 m (110 ft) below the adjacent Chicago River through dump debris, soft Chicago clays, an abandoned railroad tunnel and boulders to rock. Structural engineers in the Goldberg office were Bertold E. Weinberg and Frank Kornacker. To them, Goldberg added Moran-Proctor-Mueser-Rutledge, with Mueser and Rutledge working with them directly. Added to them on foundations were Ralph Peck and Sidney Berman (Case Foundation Company was the general foundations contractor).

Engineering consultants were Severud, Elstad & Krueger, with Severud and Bandel as principals. Dr. Andrew Fejer was their adviser on aerodynamics. To this group also joined John Banker, Frank Randall, and R. H. Olson of Portland Cement Association, who gave
them their seal of approval. The general contract was a joint venture of James McHugh Construction Co. and Brighton Construction Co., of Chicago [7].

In a construction spectacle still recalled by Chicagoans today, the towers were built at the amazing speed of one floor per day on alternating towers – or one floor every two days on each tower. As the cores were poured, cranes moved up one floor and a construction elevator rose within each shaft to supply building materials.

Figure 8. Columns details and core structural plan (courtesy of the archive of the Art Institute of Chicago).

5.2 Modeling and analysis

It is interesting to note that it has been the first structure designed with lightweight concrete. For all vertical members – columns and walls – conventional aggregate was used to produce concrete in strengths of 34 MPa (5000 psi), 23 MPa (3750 psi) and 21 MPa (3000 psi), decreasing in strength from bottom to top. For the horizontal members – slabs and beams – 21 MPa and 23 MPa – have been used, made from lightweight aggregates. The concrete core was modeled with shell elements while the columns and beams with beam elements. The modal analysis shows a complete symmetric behavior due to the geometry of the building and a proper distribution of participating mass at different periods.

<table>
<thead>
<tr>
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Since the Marina City is taller than the Pirelli building and Velasca tower, the first period of the structure is longer.

Moreover, as Chicago is not considered a seismic zone, the wind remains the main issue for tall buildings. The main difference between seismic and wind response is related to the section design. The wind analyses were performed using three different codes: the Chicago Building Code of the 1950s, the current Chicago Building Code and the ASCE7-05.

The results showed that the core is working as expected for wind resistance as it takes 94% of the wind load while the columns just 6%. Moreover, the maximum deflection obtained is about 27 cm (10.636 in) with the more restrictive code (ASCE7-05).
The wind analysis with the Chicago Building Code of the Fifties showed an interesting result: 22.4 cm (8.83 in) of maximum displacement at the top. If compared with the Chicago Building Code as it is used today, the maximum deflection is 25.7 cm (10.1 in). Accordingly, the ASCE7-05 imposes a more challenging requirement for the structure.

An interesting point to mention is that when the structure was originally designed, the maximum deflection allowable was calculated by a very simple formula (the height of the building divided by 600) and was around 28.2 cm (11.1 in) – a value that is not too far off from modern analyses.

6 CONCLUSIONS

The three buildings were conceived with an essential synergy between architects and structural engineers. The research and thoughtfulness of details in the design process and during the execution have been fundamental for these significant and durable projects. Although these conditions may be exceptional, they are more likely to occur for important buildings perceived from the beginning by their designers as possible masterpieces.

As a conclusion, the seismic safety of existing high-rise buildings in reinforced concrete necessarily implies their re-analysis and the consideration of many different aspects related to the time of construction and to the effect of the period elapsed. Yet, from the structural point of view, these buildings, albeit designed with analysis tools not comparable to the present ones, are often found to reflect great wisdom and intuition in the structural concept.

The following are therefore the main conclusions from this study:

1) For the three towers analyzed for current seismic and wind load effects, adequate design is still observed in these buildings.

2) The methods used in 50’s and 60’s for analyses produced results that are not too far off from those by modern computations.

REFERENCES


