

Cite this article

Carpinteri A, Lacidogna G and Mamone A (2026)
The Michelucci Tower in Livorno, Italy.
Proceedings of the Institution of Civil Engineers – Civil Engineering 179(1): 68–83,
<https://doi.org/10.1680/jcien.25.00196>

Research Article

Paper 2500196
Received 16/06/2025
Accepted 30/09/2025

Emerald Publishing Limited: All rights reserved

The Michelucci Tower in Livorno, Italy

1 Alberto Carpinteri

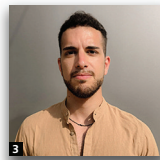
Department of Civil Engineering and Intelligent Construction, Shantou University, Shantou, China

2 Giuseppe Lacidogna

Department of Structural, Geotechnical and Building Engineering, Politecnico di Torino, Turin, Italy (corresponding author: giuseppe.lacidogna@polito.it)

3 Andrea Mamone

Department of Structural, Geotechnical and Building Engineering, Politecnico di Torino, Turin, Italy



In the contemporary era, civil engineering design is predominantly carried out using software tools that offer significantly reduced operational time due to automatic calculations. However, these tools often require a substantial amount of data input and extensive post-processing of results. In the context of high-rise buildings, the General Algorithm and its software implementation represent a promising solution to this challenge, ensuring a significant improvement in efficiency during both data input and result analysis. The Tall Building Structural System (TaBu) computational code, which implements analytical models within the Matlab environment, despite inherent modelling limitations, has demonstrated its ability to estimate results comparable to those obtained through finite-element (FE) analysis, with deviations often less than 10%. However, in cases involving complex geometries like the Michelucci Tower in Livorno, Italy, the differences can reach 20–30%, due to the simplifications inherent in the analytical approach. Nevertheless, TaBu can be considered a valuable tool for preliminary building analyses, facilitating the subsequent transition to more detailed modelling. The purpose of this study was to compare the horizontal displacements of the high-rise Michelucci Tower estimated using TaBu software (derived from an analytical model) and Dolmen FE software.

Keywords: built environment; civil engineering software; conservation; displacement estimation; efficiency in design processes; high-rise building analysis; Michelucci Tower; municipal & public service engineering; structural modelling

1. Introduction

The twentieth century witnessed significant innovations in civil engineering constructions, marked by the emergence of novel structural solutions, including high-rise buildings. The Michelucci Tower, in Livorno, Italy, features an unconventional volumetric configuration that distinguishes it from contemporary structures. Designed by Giovanni Michelucci, a renowned Italian architect of the twentieth century, the skyscraper was constructed between 1961 and 1966. Standing approximately 90 m tall, the tower comprises 26 storeys above ground, culminating in a terrace at its highest level. At the time of its completion, it was one of the tallest structures in Italy (Figure 1).

The architectural composition of the building is characterised by two distinct structural parts – a polygonal base plate, which encompasses the first six storeys and serves as a pedestal, and a

tower that rises above it. The tower is further subdivided into five sections, with façades aligned with those of the base plate.

The tower exhibits remarkable originality in its design, adhering to an entirely innovative and, at times, seemingly chaotic scheme. This is evidenced by a series of additions and subtractions dictated by the presence of projections and bow windows (protruding structures akin to enclosed balconies), which contribute to the building's distinctive architectural identity. Such complex and unconventional architectural features often pose significant challenges for structural analysis and assessment. Therefore, understanding and accurately predicting the building's response to environmental loads is crucial for ensuring safety and performance.

The main contribution of this paper is demonstrating the effectiveness of a simplified, analytical modelling approach (using Tall Building Structural System (TaBu) software based on the General Algorithm (GA)) for estimating horizontal displacements in high-rise



Figure 1. (a) View of Michelucci Tower. (b) Perspective view taken from the on-line documents of the Giovanni Michelucci Foundation (GMF, 2025)

buildings. The method was validated through comparison with detailed results from finite-element (FE) analyses. By applying this approach to a structure characterised by intricate and seemingly chaotic design (the Michelucci Tower in Livorno), the study highlights the potential of simplified models to provide reliable initial insights. New insights into the accuracy (maximum deviations of 20–30%) of this rapid assessment method for wind and seismic loads are provided, emphasising its significant time-saving advantages. The results show that, despite the complexity and uniqueness of such architectural schemes, simplified analytical tools can serve as practical and efficient means for early-stage structural evaluation, facilitating informed decision making when detailed data is limited. The findings support the practical application of analytical tools in preliminary structural analyses, offering a balance between computational efficiency and accuracy of results.

2. Preliminary investigations

Considering the temporal context of the construction of the Michelucci Tower, it was immediately apparent that it was not possible to easily find the entire design documentation. For structures made of plain or reinforced concrete, before commencing construction, Article 4 of Royal Decree No. 2229 of 16 November 1939

required private entities to file a notice of works with the prefecture of the relevant province, accompanied by a preliminary design. The same article mandated the submission of a Final Acceptance Certificate to the prefecture upon completion of the works (Figure 2). However, while the design documentation, including all executive details, had to be kept on site and made available for potential inspections, there was no legal obligation to submit a copy for official record keeping. In contrast, structures built on behalf of the state and under the supervision of state technical authorities were exempt from the requirement to file a notice with the prefecture.

As a result, a comprehensive and meticulous document investigation was necessary. An initial search at the Giovanni Michelucci Foundation in Fiesole (Florence) led to the discovery of 17 drawings, including various elevations and several floor plans (Figure 3). However, subsequent investigations conducted at the Civil Engineering Department of the Municipality of Livorno and the building administrator were less successful, yielding no additional design documentation.

Given the historical and architectural significance of the tower, an extensive bibliographic investigation was undertaken. This research identified specialised texts containing further detailed floor plans for several key levels of the building, including those by Fabbri (2016) and Lusconi (2010). Although these

Art. 4.	Article 4.
<p>Ai costruttori, prima di iniziare la costruzione delle opere, di cui all'art. 1, è fatto obbligo di presentarne alla prefettura della provincia denuncia, corredata di una copia del progetto di massima.</p>	<p>Prior to commencing the construction of the works referred to in Article 1, builders are required to submit a notification to the prefecture of the province, accompanied by a copy of the preliminary project.</p>
<p>Nei cantieri, dal giorno dell'inizio a quello di ultimazione dei lavori, deve essere conservata una copia dei particolari esecutivi di tutte le parti delle opere in costruzione, datati e firmati dal progettista, dal direttore dei lavori e dal costruttore. Il direttore dei lavori deve riportare nei disegni, con inchiostro di colore diverso, tutte le modifiche introdotte nelle opere all'atto esecutivo, datandole e firmandole.</p>	<p>At construction sites, from the start date until the completion of the works, a copy of the detailed execution plans for all parts of the works under construction must be kept, dated and signed by the designer, the site manager, and the builder. The site manager must record in the drawings, using ink of a different color, all modifications made during the execution phase, dating and signing them.</p>
<p>... Al termine dei lavori il committente, per ottenere la licenza di uso della costruzione, deve presentare alla prefettura il certificato di collaudo delle opere, rilasciato da un ingegnere di riconosciuta competenza, iscritto all'albo.</p>	<p>... Upon completion of the works, the client, in order to obtain the occupancy permit, must present to the prefecture the testing certificate of the works, issued by a recognized and registered engineer with proven competence.</p>
(a)	(b)

Figure 2. Excerpt from Royal Decree No. 2229 of 16 November 1939, 'Regulations for the execution of structures in plain or reinforced concrete': (a) original version in Italian; (b) English translation

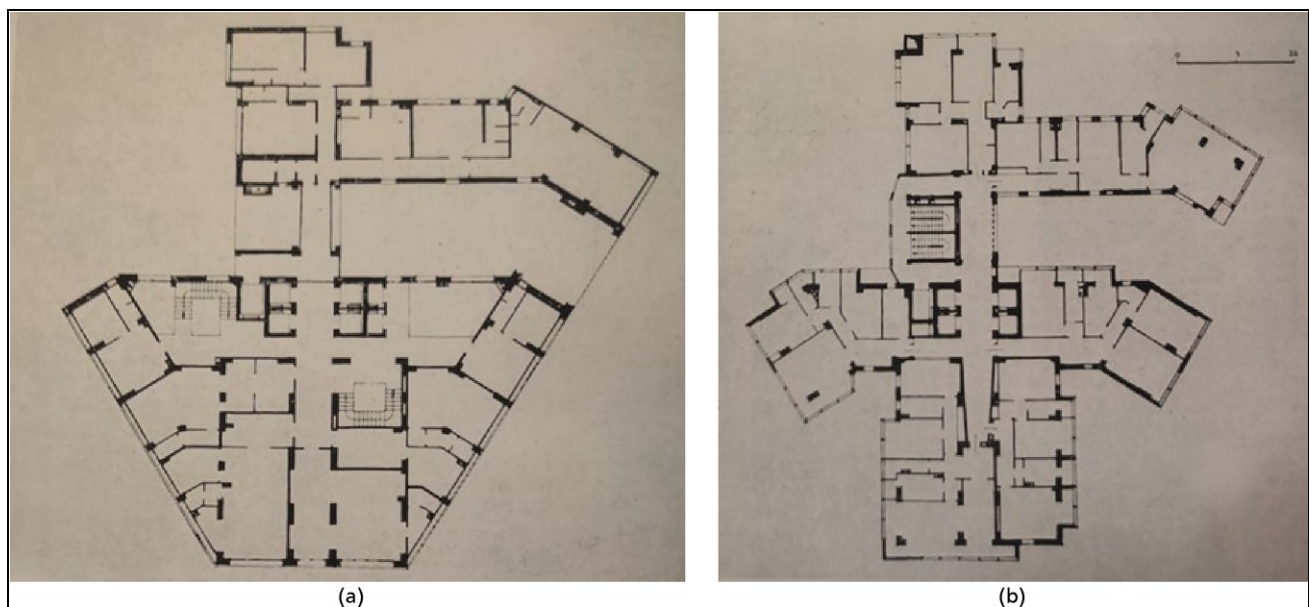


Figure 3. (a) First floor plan. (b) Floor plan for levels 8–11. Extracted from the online documents of the Giovanni Michelucci Foundation (GMF, 2025)

documents primarily focus on the architectural aspects of the building, they provide a reasonably accurate representation of the main geometric characteristics of the typical floors.

At the time of writing, no structural documentation had been found.

3. Structural features and construction phase challenges

The first challenges in erecting the structure emerged during the foundation works. The soil conditions in the city of Livorno are

characterised by high clay contents and significant moisture levels, necessitating extensive and complex preparatory measures. The foundation system required the construction of a sheet pile, essentially a large metal caisson filled with tonnes of concrete and reinforced with 500t of steel. Additionally, 600 piles, each of dia. 40 cm and length 20 m, were driven into the ground. These measures resulted in the formation of a stable platform capable of supporting the entire structural complex.

Once the foundations were completed, construction progressed at a remarkable pace. By September 1963, the 13th floor had already been reached, with the 27th floor completed by April 1964. By late summer of the same year, finishing works commenced, requiring meticulous attention due to the integration of specific decorative and aesthetic elements. However, in early 1965, construction of the tower came to an end. Workers were progressively laid off and, by February of 1965, only two remained on site.

According to Fillea-CGIL, the Italian trade union representing workers in construction and part of the larger CGIL (Italian General Confederation of Labour), the province of Livorno – like the rest of the country – was experiencing a severe crisis in the construction sector. Numerous construction sites either reduced their activities or limited working hours. Conversely, the construction company attributed the delay to seasonal factors, arguing that the freezing temperatures of January and February rendered construction at considerable heights unfeasible. Due to these adverse weather conditions, work resumed in April 1965. By May, construction activities were underway on both the façade and the interior, including the installation of hydraulic and electrical systems. By the end of 1966,

following a construction period spanning a decade, the building designed by Michelucci was finally completed.

3.1 Volumetric and structural description

The arrangement of structural elements within the building exhibits considerable variability across different floors. The ground floor presents the most distinct configuration compared with the upper levels. At ground level, the structure is divided into two separate blocks by a vehicular passage that connects the two streets flanking the building. Access to the upper floors is provided by two staircases and seven elevators; however, the staircases extend only up to the sixth floor.

Most columns are integrated within the cladding or infill walls, thereby minimising their visual and spatial impact. In contrast, the shear walls follow a non-symmetrical configuration, except for those enclosing the elevator shafts (Figure 4). The horizontal slabs are supported by load-bearing dropped beams, which reveal their structural layout as well as the orthogonal orientation of the secondary joists.

From the third floor onwards, cantilevered perimeter structures emerge, extending up to the sixth floor. Beyond this level, the seventh floor primarily serves as a roof for the underlying block. In this configuration, the external projections transform into open-air spaces, leading to a significant structural variation due to the absence of two large sections of the structure.

A notable structural change occurs on the fifteenth floor, where the right rear wing of the building is entirely removed (Figure 5).

With regard to the shear walls, it is important to note that those associated with the first two elevator shafts are interrupted, as these elevators only serve the floors below the seventh level.

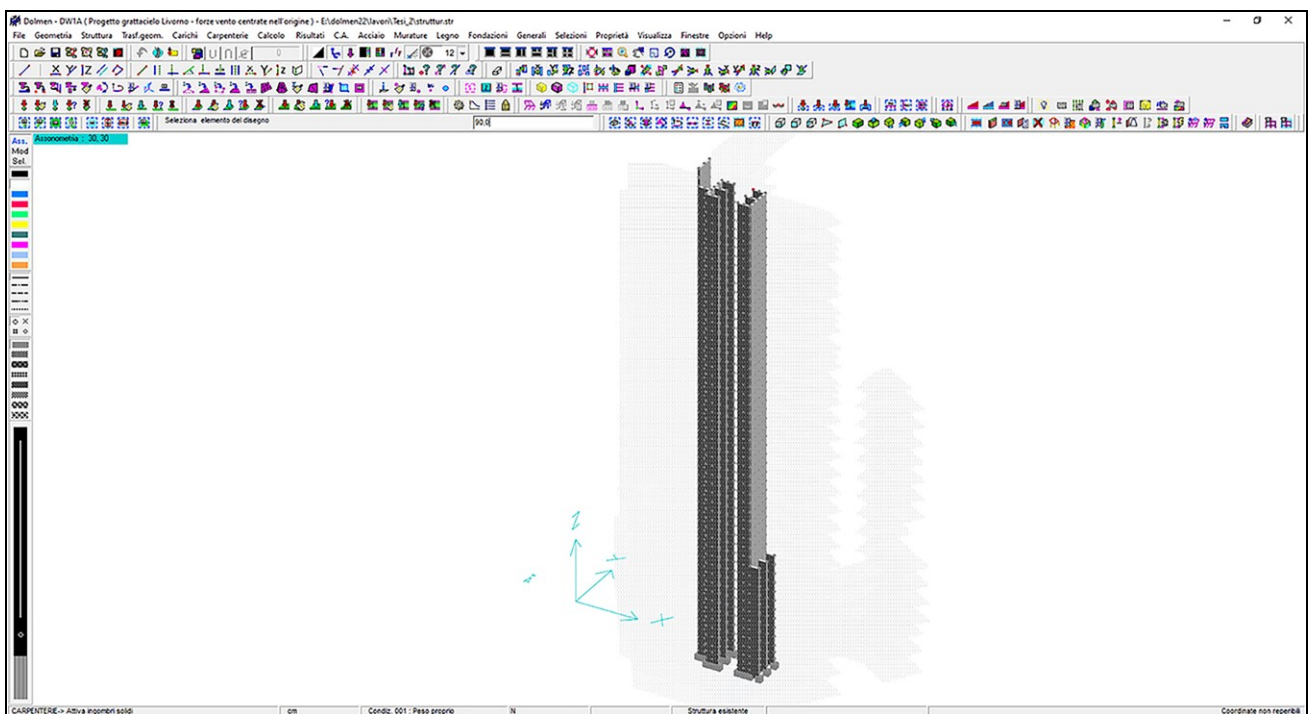


Figure 4. Three-dimensional FEM: view of the central reinforced concrete shear walls

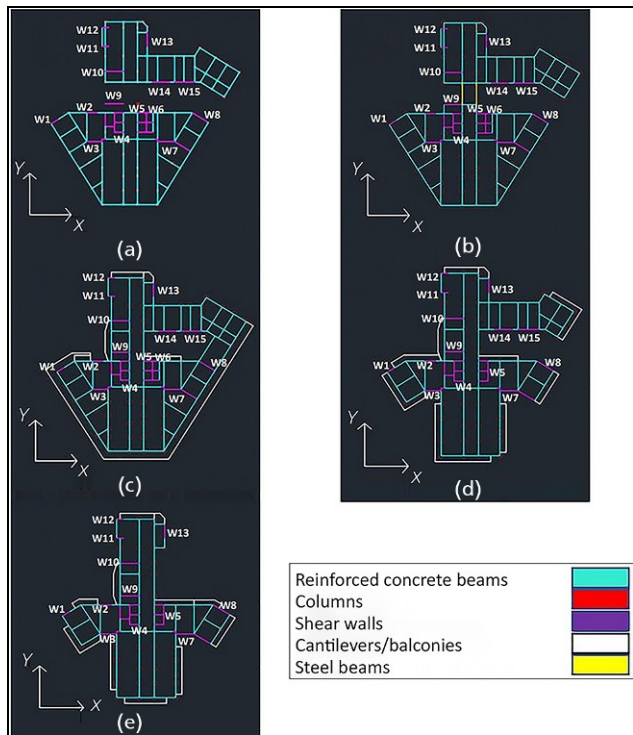


Figure 5. Five distinct typical floor plans of the Livorno skyscraper, illustrating the lateral load-resisting systems and their continuity across different levels. The plans include wall numbering (W1, W2, W3 etc.) to facilitate tracking of the structural elements: (a) ground floor; (b) first floor; (c) floors 2 to 6; (d) floors 7 to 15; (e) floors 16 to 26

The final identifiable structural block comprises the floors between the 16th and 26th levels. The primary structural modifications compared with the preceding block include the removal of the emergency elevator and the two elevators that previously served floors 7–15. Despite these changes, the overall floor layouts remain largely consistent, with only minor geometric variations in the balcony configurations and the introduction of additional architectural details, such as the two ramps of the emergency staircase.

The top floor consists of an accessible roof, featuring open-air peripheral areas and a central volume resembling an attic space with a roof covering. The emergency staircase, which serves as the sole access to this level, leads directly into this box-shaped structure, which is equipped with exits providing direct access to the terrace.

In conclusion, five distinct typical floor configurations were identified with reasonable accuracy (Figure 5), allowing for a comprehensive definition of the volumetric composition of the tower.

3.2 Construction details

Due to the absence of structural documentation and considering the objectives of this study, the structural elements were defined through a meticulous analysis of the available architectural drawings and reasonable design assumptions. Since the primary goal

was to evaluate the feasibility of a simplified modelling approach for estimating horizontal displacements, significant effort was made to accurately identify and approximate the dimensions of the structural elements.

The investigations clearly established the positions of the main structural components and the spans of the load-bearing beams. However, detailed data regarding the exact thickness of the shear walls and the precise dimensions of beam and column sections were unavailable. Where necessary, the definition of the building's load-bearing structure was based on reasonable assumptions regarding expected structural behaviour and common construction practices of the time.

3.3 Materials

This study, being limited to a comparative analysis of results obtained through different calculation methods, did not include any in situ material testing. As a result, the compressive strength of the concrete, the type of materials and the exact arrangement of the reinforcement bars were unknown. In the absence of direct test results, the mechanical properties of the concrete and reinforcing steel were defined based on previous studies that examined the evolution of material characteristics over time.

3.3.1 Concrete

Estimation of the compressive strength of concrete in an existing reinforced concrete structure can be performed using the model proposed by Fantilli and Ferraro (2020) from Politecnico di Torino. This approach derives the strength parameter as a function of the construction year, using strength–age curves.

Figure 6, based on 100 years of structural concrete certification data, illustrates the distribution of average concrete cubic strength values, including the 5%, 25%, 75% and 95% percentiles. Given that the building under study was constructed beginning in 1961, with its design dating back to approximately 1956, it was assumed – based on the available data – that the in situ concrete likely possesses mechanical characteristics comparable to those of modern C20/25 concrete.

3.3.2 Steel

Similarly, to estimate the mechanical properties of the existing steel reinforcement, reference was made to a study conducted by Verderame *et al.* (2011) at the Department of Structural

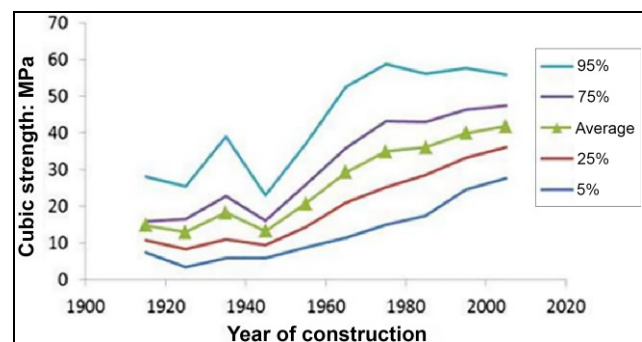


Figure 6. Cubic strength of structural concrete as a function of year of construction (Fantilli and Ferraro, 2020)

Engineering of the University of Naples Federico II. Figure 7 shows a statistical analysis of the mechanical properties of reinforcement steel used in Italy between 1950 and 1980 (Verderame *et al.*, 2011).

4. Objective of the analysis and working assumptions

The objective was to assess the effectiveness of a simplified schematic representation of the structure, as implemented in TaBu software, for obtaining horizontal displacements induced by wind and/or seismic actions. The extensive computational capabilities of TaBu are described in Section 5. This evaluation was conducted by comparing the results of the simplified TaBu model with those derived from a detailed finite-element model (FEM) developed using the commercial software Dolmen (CDM Dolmen, 2025).

Since the exact dimensions of certain structural elements (e.g. beams, columns and shear walls) were partially unknown, plausible structural assumptions were made to define these geometrical parameters. The structural analyses were not intended for building purposes but rather for evaluating the structural response under horizontal loading. Given the limited availability of detailed structural data, the assessment of stresses was performed only on wall W4, shown in Figure 5. Details of this assessment are provided in the Appendix. On the other hand, the study focused on comparing the displacement obtained from the two different computational methodologies.

5. Modelling with TaBu: General Algorithm and analytical formulation

The TaBu computational code, developed within the Matlab environment, is based on a robust analytical model known as the GA. This code is designed to facilitate rapid data input and enable prompt evaluation of design modifications for high-rise structures. The core of the methodology is derived from the foundational

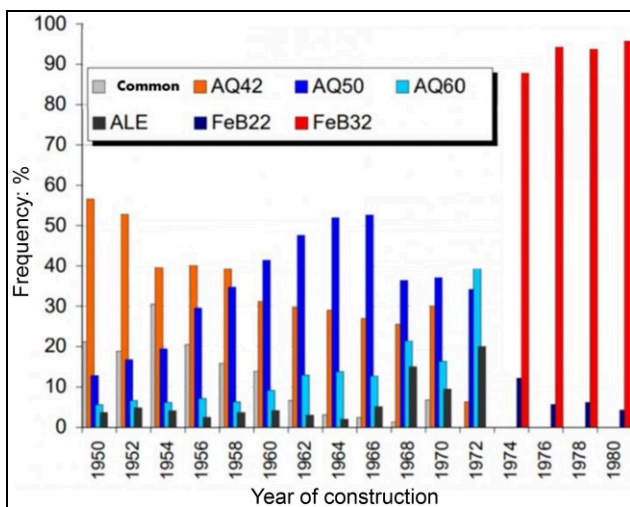


Figure 7. Frequency distribution for various steel types as a function of year of construction (Verderame *et al.*, 2011)

work of Capurso (1981) and Carpinteri and Carpinteri (1985), which has been further developed in recent years (Carpinteri *et al.*, 2010, 2013, 2014, 2016; Lacidogna, 2017; Nitti *et al.*, 2021) to analyse a wide range of building typologies.

5.1 General Algorithm and structural discretisation

The GA formulation models the entire building by connecting all vertical bracings (frames, cores, walls) at each floor level through infinitely rigid slabs in their own plane but infinitely deformable out-of-plane. Each vertical bracing is localised in its own local reference system (origin at the shear centre, axes parallel to the principal inertia directions).

The model's computational efficiency stems from a drastic reduction in the number of degrees of freedom (DoF). For an N -storey building, the model considers only three DoF per floor, counted from top to bottom. The unknown displacement vector for each floor is $\{\delta\} = \{\{\xi\}; \{\eta\}; \{\theta\}\}$, representing the two translations along the global x and y axes and the rigid rotation around the z -axis.

5.2 Static analysis formulation

The external load vector applied to the structure is $\{F\} = \{\{p_x\}; \{p_y\}; \{m_z\}\}$, consisting of lateral forces and torsional moments at each floor.

The local stiffness matrix $[\bar{K}_i]$ of each i th bracing, defined in its local reference system, relates the internal loads it carries to the floor displacements:

$$1. \quad \{F_i\} = [\bar{K}_i] \{\delta\}$$

Global equilibrium is enforced by summing the contributions of all M bracings, leading to the global stiffness matrix $[K]$ of the entire building:

$$2. \quad \{F\} = \sum_{i=1}^M \{F_i\} = \left(\sum_{i=1}^M [\bar{K}_i] \right) \{\delta\} = [K] \{\delta\}$$

The distribution of the external load $\{F\}$ to each individual bracing is calculated using the distribution matrix $[R_i]$:

$$3. \quad \{F_i\} = [\bar{K}_i] [K]^{-1} \{F\} = [R_i] \{F\}$$

Once the forces on a bracing $\{F_i\}$ are known, Equation 1 can be inverted to determine the displacements $\{\delta\}$ and, consequently, the internal stresses acting on that element.

5.3 Dynamic analysis formulation

For dynamic analysis, the building is modelled as an equivalent cantilever beam with floor masses concentrated at their centres of gravity. The undamped equation of motion is:

$$4. \quad [M] \{\ddot{\delta}\} + [K] \{\delta\} = \{0\}$$

where $[\mathbf{M}]$ is the mass matrix and $[\mathbf{K}]$ is the global stiffness matrix. Assuming the solution can be separated into a spatial component $\{\zeta(z)\}$ and a temporal component $\psi(t)$, such that $\{\delta\} = \{\zeta(z)\} \psi(t)$, Equation 4 leads to the eigenvalue problem:

$$5. \quad \det([\mathbf{K}] - \omega_n^2[\mathbf{M}]) = 0$$

Solving this yields $3N$ eigenvalues (ω_n), from which the natural frequencies ($f_n = \omega_n/2\pi$) and periods of vibration ($T_n = 1/f_n$) are derived. The corresponding eigenvectors $\{\zeta(z)\}$ represent the mode shapes of the building (Carpinteri *et al.*, 2016; Nitti *et al.*, 2021; Zalka, 2001).

5.4 Model advantages and limitations

This analytical approach provides detailed insights into stress distributions and dynamic characteristics while offering the following significant advantages.

- Drastically reduced computation times compared with commercial FE software due to the low number of DoF.
- No meshing is required.
- Simplified and rapid input process.
- Results generally align well with FE analyses, typically exhibiting deviations of less than 10% (Carpinteri *et al.*, 2013, 2014), which is acceptable for preliminary design phases. Figure 8 illustrates the simplified model of the tower discretised using TaBu.

The model's efficiency imposes simplifications on the modelling process.

- Floors are assumed to be rectangular.
- Frames are modelled as independent planar elements with constant spans and constant member cross-sections.

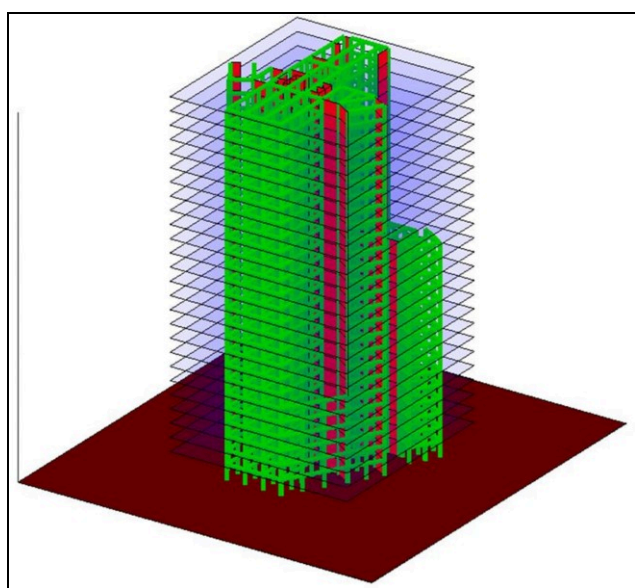


Figure 8. Simplified model of the Michelucci Tower discretised with TaBu

- All elements are clamped at the base.
- Elements modelled with Vlasov's theory (Vlasov, 1961) for thin-walled sections are of constant thickness.

This methodology provides a versatile and efficient conceptual tool for the initial three-dimensional (3D) design of tall buildings, allowing for accurate modelling of lateral load distributions, secondary effects and dynamic characteristics.

6. Modelling with Dolmen

Based on the floor plans identified in the literature and supported by several in situ measurements, a geometric representation of the building was obtained with a satisfactory degree of approximation. Given the general geometry of the typical floors, the vertical load-bearing elements (columns and shear walls) were identified, and the arrangement of the main beams and slabs was assumed accordingly.

Using the five identified typical floor plans (Figure 5), structural schemes of the corresponding building levels were developed. The cross-sections of the individual structural elements (beams and columns), which were not entirely known, were defined based on plausible design assumptions. For the floors without any graphical documentation, structural layouts were generated by analogy with the documented ones, considering the overall structure of the building.

Using the extensive structural modelling capabilities of Dolmen, the complete structural model was successfully generated. Obviously, the generation of the structural floor plans required certain approximations due to the lack of detailed information regarding the cross-sectional dimensions of structural elements. The floor plans were vertically connected by inserting the corresponding column and wall elements maintaining a constant inter-storey height of 3.20 m, except for the ground floor, which is separated from the first floor by a height of 3.60 m.

The FEM generated in Dolmen, assuming the use of concrete comparable to class C20/25, can be considered sufficiently representative of the real structure in terms of global geometry.

It is important to emphasise that the FEM scheme was not intended for a detailed assessment of structural elements but rather at evaluating the global behaviour of a tall building typology. Consequently, the detailed assumptions regarding the dimensions of the structural elements introduced in the calculations, being consistently applied in both the TaBu and FEM, retained their validity and did not compromise the results of the comparison despite potential deviations from the actual dimensions.

The FEM (Figure 9(a)) consisted of

- 5693 one-dimensional beam elements
- 3203 two-dimensional shell elements
- 400 slabs
- 5533 nodes.

In analogy with the simplified modelling approach adopted in TaBu, the assumption of infinitely rigid slabs within their own plane was also adopted in the FEM.

The seismic forces were defined based on the parameters shown in Figure 10. The seismic analysis parameters (according to NTC 2018 (MIT, 2018)) were

- building location = Livorno (longitude 10°32', latitude 43°55')
- ground type = C



Figure 9. (a) FEM constructed using Dolmen. (b) View of the real structure

- stratigraphic amplification coefficient $S_S = 1.500$
- topographic amplification coefficient $S_T = 1.000$
- S factor = 1.500
- design life of structure $V_N = 50$ years
- importance factor $C_U = 1.0$
- reference period $V_R = 50.0$
- behaviour factor $q = 1.500$.

7. Analyses performed

The analyses were conducted based on the adopted working hypotheses, with the primary objective of comparing the horizontal displacements estimated using the analytical model through TaBu software and the FEM created with Dolmen software. These comparisons were carried out under four loading conditions

- wind action in the X direction
- wind action in the Y direction
- seismic action in the X direction
- seismic action in the Y direction.

The detailed discretisation of the structure enabled by Dolmen necessitated appropriate refinements to facilitate a meaningful comparison with the results obtained using TaBu.

As a preliminary step, an operational calibration phase was undertaken to ensure adequate consistency between the two models. This phase involved testing simplified models to properly represent the assumptions inherent to TaBu, particularly regarding floors, which were considered infinitely rigid within their plane and infinitely deformable outside it. Additionally, this phase aimed to optimally define the constraints between structural elements.

The comparison of the results obtained from both models under the four loading conditions was intended to assess the degree of correspondence between the two approaches.

8. Comparison of TaBu analytical model and FEM

The FEM developed with Dolmen was used to estimate horizontal displacements (in both the X and Y directions) and the rotation at a point near the barycentre of each floor (Figure 11).

After identification of the nodes of each floor corresponding to the desired geometric location, the data export functionalities provided by Dolmen (Figure 12) were used to generate simple tables containing the required displacements and rotations. The export of data in a format compatible with Excel enabled an efficient and rapid graphical interpretation.

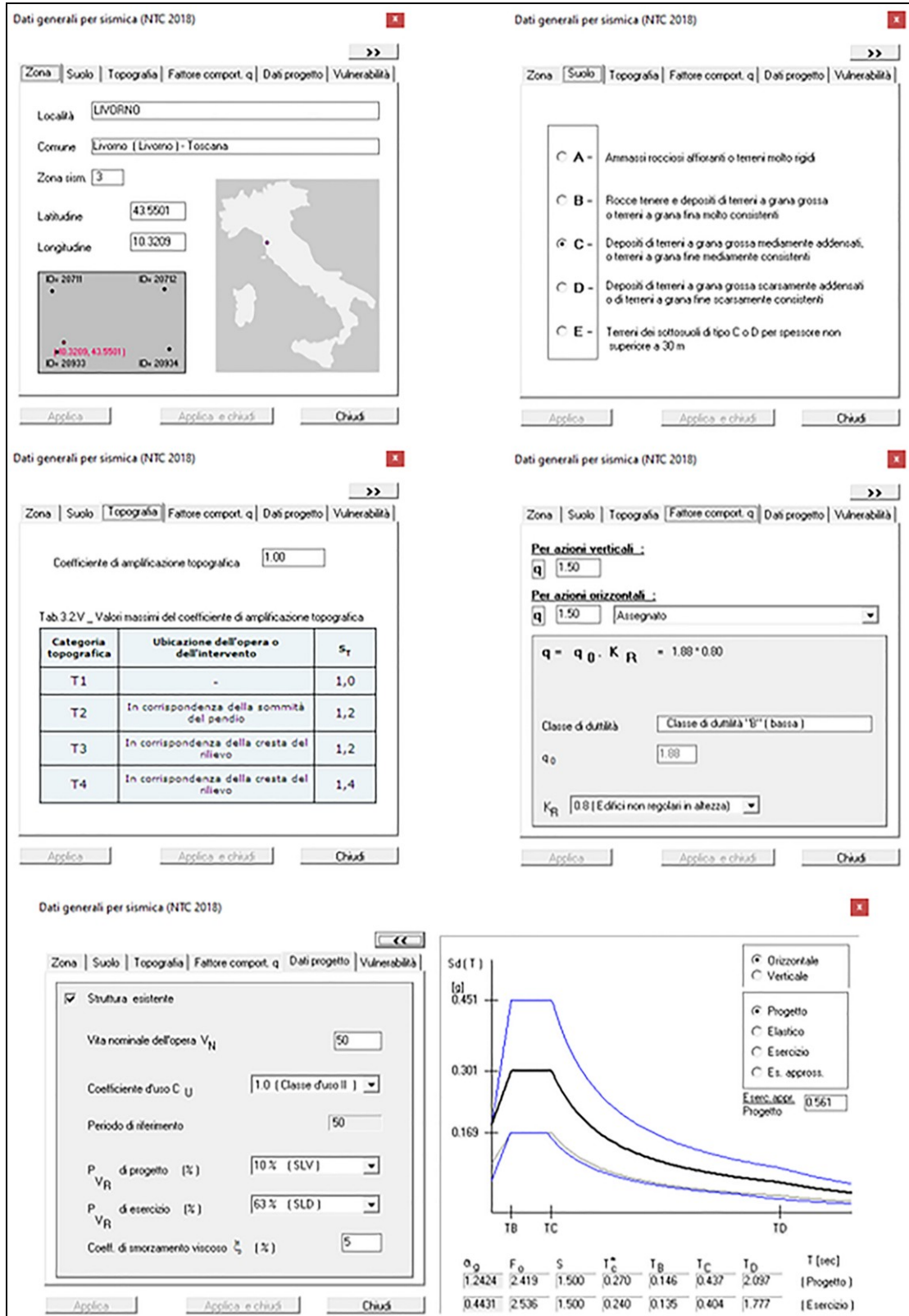


Figure 10. Parameters adopted for seismic actions on the Michelucci Tower

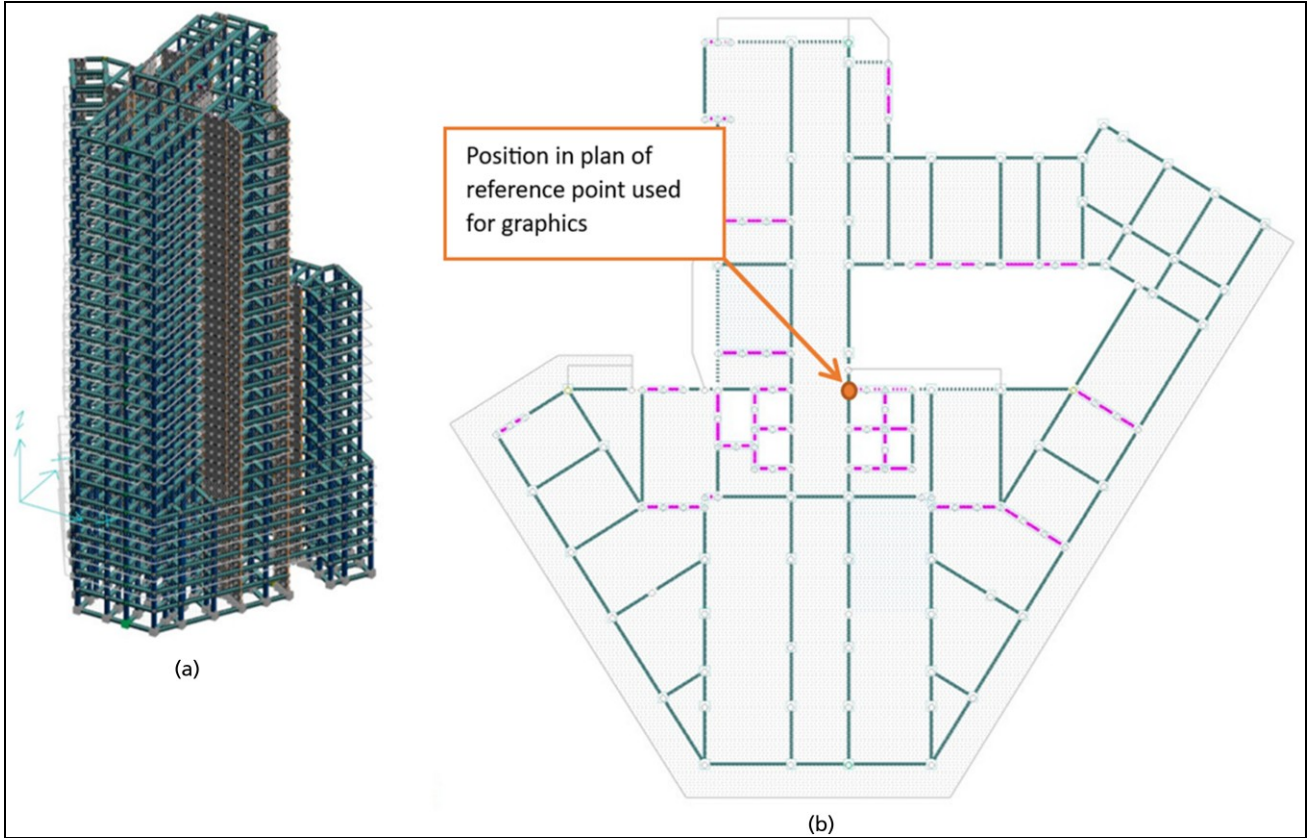


Figure 11. (a) FEM developed using Dolmen. (b) Chosen position for displacement measurements

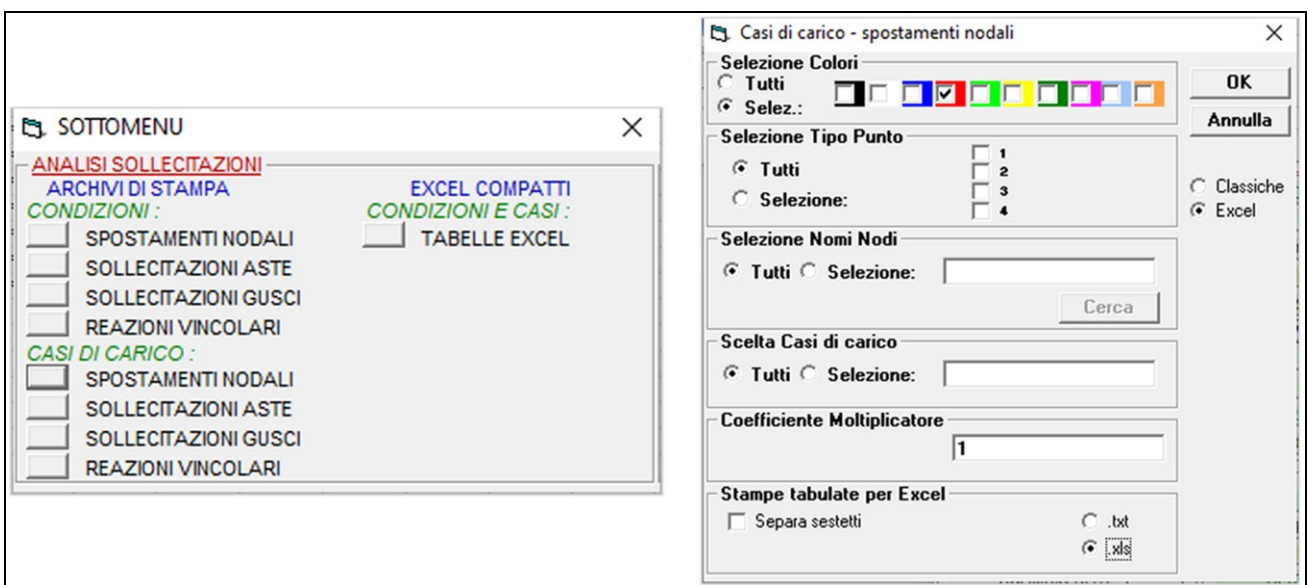


Figure 12. Dolmen menu for reading and saving node displacement data (selected by colour)

The same displacements and rotations were estimated using TaBu and subsequently compared with those obtained from the FEM. Below, a comparative analysis of the results provided by the two software tools is presented. The graphs illustrate horizontal displacements (X and Y) and rotations, where the results from Dolmen are represented by the blue curve and those from TaBu by the red curve. In each graph, the horizontal axis represents the displacements or rotations, whereas the vertical axis corresponds to the floor levels.

A strong quantitative agreement was observed for both displacements and rotations due to wind action in the X and Y directions (Figures 13–15), as well as for displacements and rotations related to seismic action (Figures 16 and 17).

For wind action in the X direction (Figure 13), the overall trends of the diagrams exhibited a high degree of similarity. The deviations between the values obtained for each floor using the two models were as follows: in the direction of the applied load, the maximum deviation occurred on the top floor, reaching 22.61%; whereas in the direction orthogonal to the load and in the case of rotation, the greatest discrepancy was observed at the 15th floor, with maximum deviations of 31.68% and 30.99%, respectively.

For wind action in the Y direction (Figure 15), the smallest deviation between the diagrams was again recorded in the direction of the applied load, with a maximum deviation of 19.59% at the top floor. The maximum deviation in the orthogonal direction to the applied load was also at the top floor, reaching 32.42%. For rotation, the maximum discrepancy was again observed at the top floor, amounting to 29.27%.

Finally, considering seismic action in the Y and X directions (Figures 16 and 17), the following differences between the FEM

and the TaBu code were observed. In the Y direction (direction of the applied load), the maximum deviation in displacements occurred at the top floor, reaching 23.51%. In the orthogonal X direction, a maximum deviation of 17.56% was observed at the same level. For rotations, the diagrams were nearly superimposed, with a maximum deviation of 18.97%, again at the top floor.

It is worth noting that the discrepancies of 20–30% between the analytical model and the FEM are primarily due to the complex and unconventional architecture of the Michelucci Tower, which challenges the simplifications inherent in the analytical approach. While some data assumptions were involved in both models, the idealised analytical model assumes regular, rectangular floors and uniform structural elements (Carpinteri *et al.*, 2013, 2014). These simplifications led to less accurate results, especially given the tower's intricate geometry, featuring projections, bow windows and irregular load paths.

When structural continuity, especially along the vertical development, is interrupted (as in this case), local effects captured by the detailed FEM may not be perfectly simulated by the analytical approach. Nonetheless, despite these differences, the analytical model effectively captured the irregular and non-trivial behaviours of displacements and torsional rotations, as shown in Figures 13, 15 and 16.

9. Conclusions

The Michelucci Tower in Livorno, a remarkable example of Italian architectural and construction capabilities from the early 1960s, was conceived by the inventive genius of Giovanni Michelucci, one of the most renowned Italian architects of the

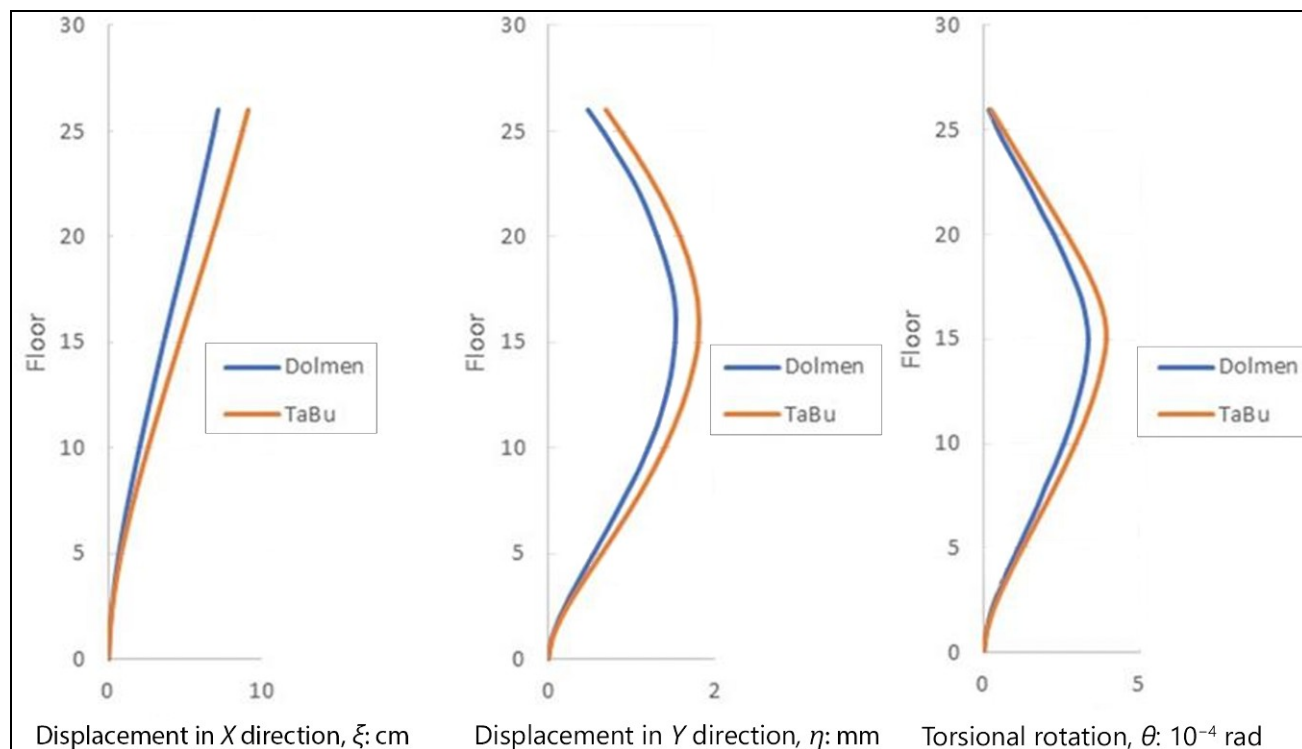


Figure 13. Comparison of displacements and rotations due to wind action acting in the X direction obtained with the FEM developed with Dolmen and the TaBu code

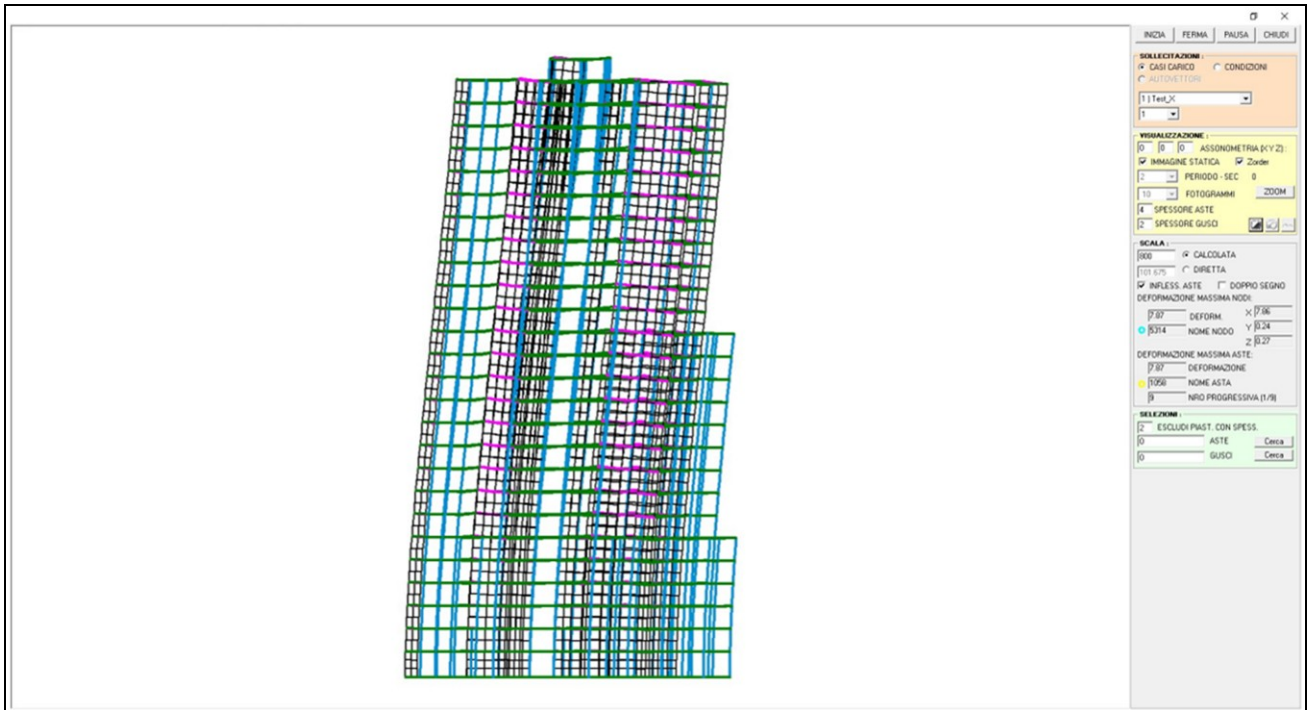


Figure 14. Deformed shape considering displacements along the X axis due to wind action in the X direction obtained from the FEM

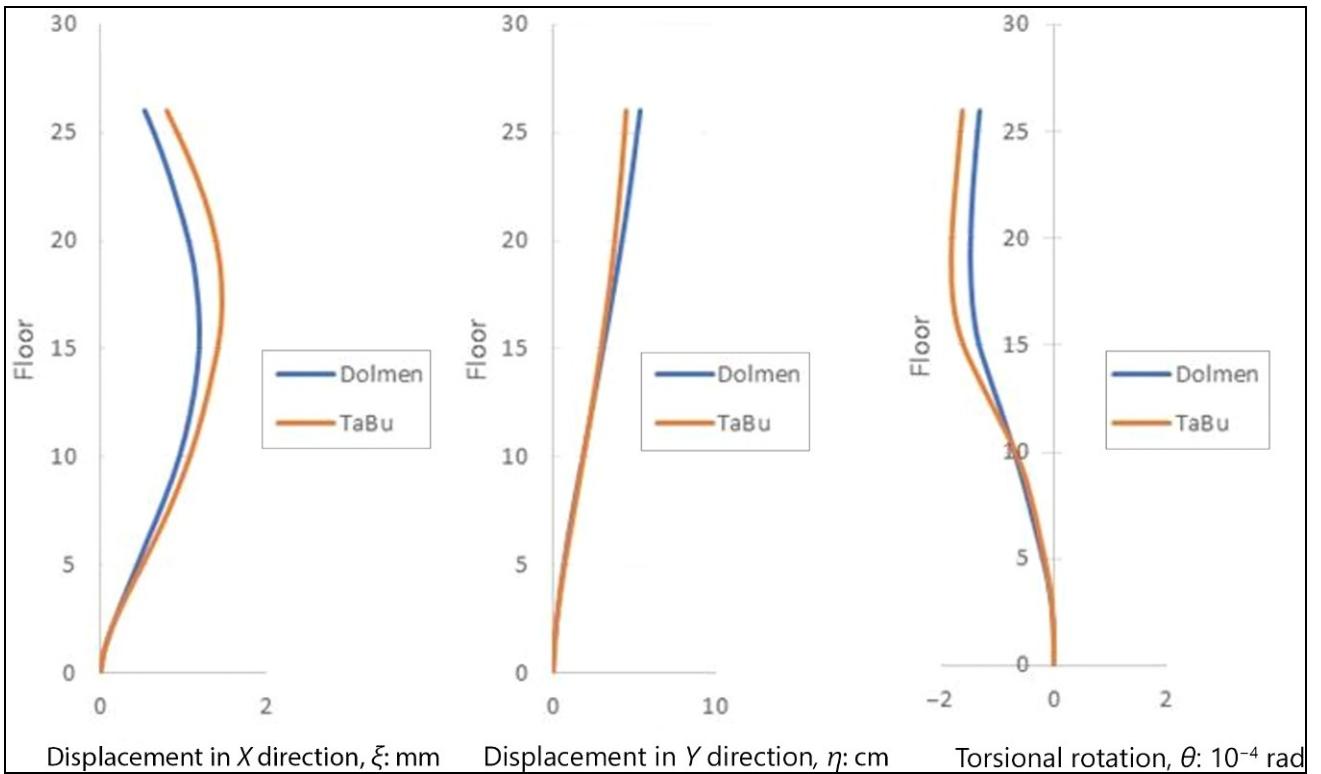


Figure 15. Comparison of displacements and rotations induced by wind action in the Y direction, obtained using the Dolmen FEM and the TaBu code

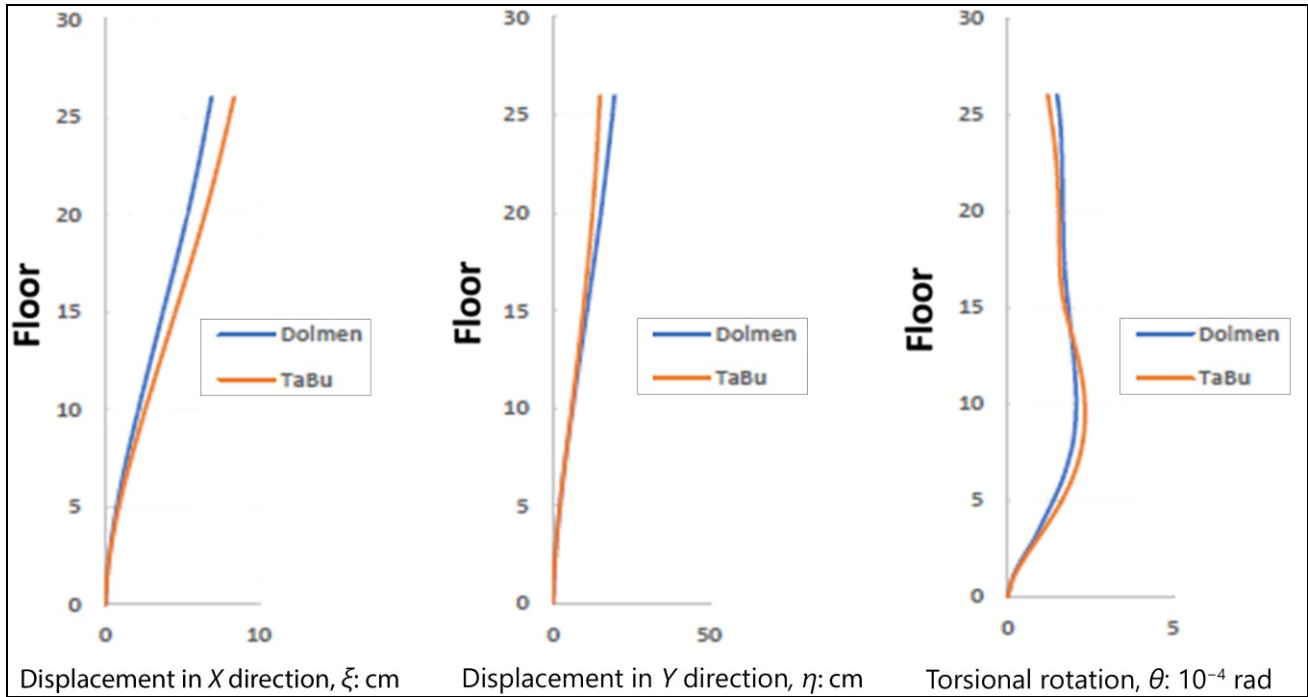


Figure 16. Comparison of displacements and rotations induced by seismic action in the Y direction, obtained using the Dolmen FEM and the TaBu code

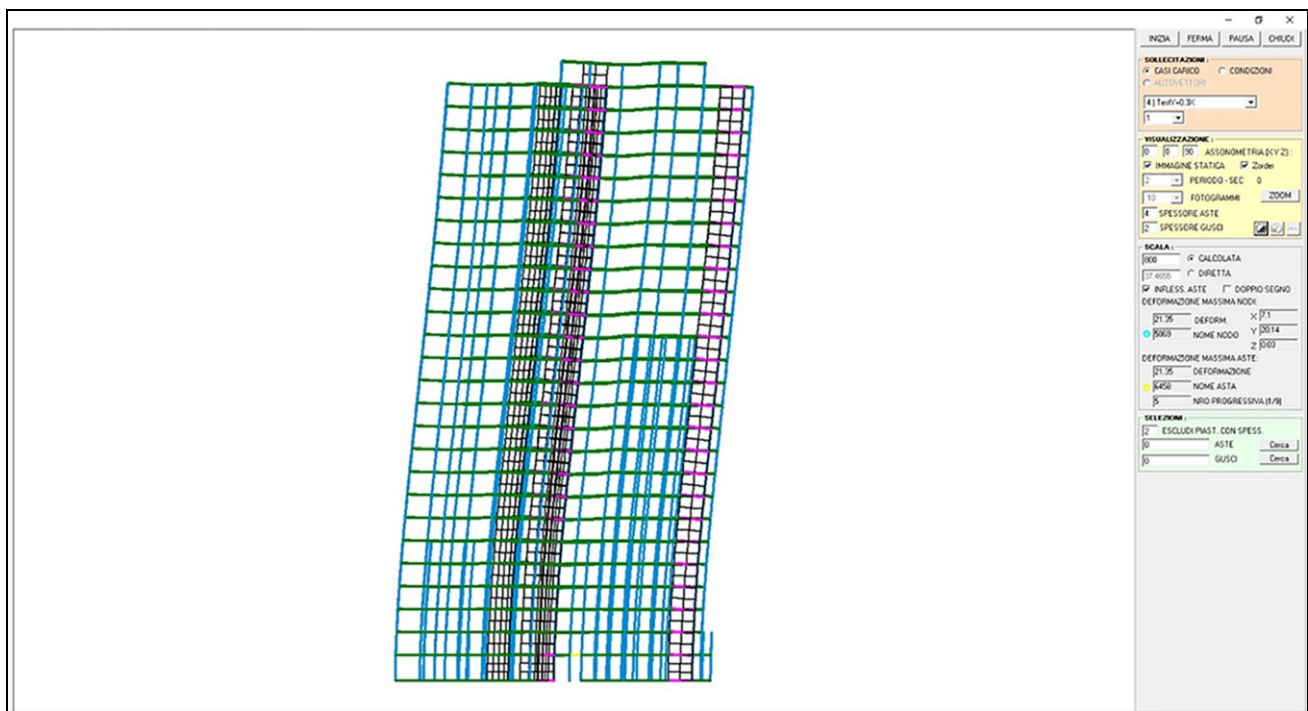


Figure 17. Deformed shape considering displacements along the Y axis due to seismic action in the X direction obtained from the FEM

twentieth century. This tower was chosen as a reference structure for comparing the results of two distinct computational approaches on tall buildings. The first approach, characterised by a more schematic and expedite analytic computation of the structure, was implemented through TaBu software, which offers a simplified modelling process and greater computational efficiency in estimating horizontal displacements due to wind and seismic actions. The second approach, involving a more detailed representation of the geometry, loading conditions and mechanical behaviour of the structure, was conducted through the development of a 3D FEM using Dolmen software.

Comparison of the results from the two software programs, focusing on the horizontal displacements induced by wind and seismic loads, demonstrated a reasonable level of agreement, with maximum deviations of 20–30%. However, it is essential to emphasise that one of the main advantages of using TaBu software lies in its extraordinary computational speed. It is believed that the ratio between the processing times required by the FEM, which had six DoF per node, and those needed for TaBu, which had three DoF per floor, is proportional to the square of the ratio between the number of DoF of the two systems. This means that, as the complexity of the structure increases, TaBu can ensure a significant saving in time, making it a particularly efficient tool for preliminary structural analysis.

Given the relatively modest discrepancies in horizontal displacement estimates between the two models, it is recommended that rapid and simplified preliminary analyses using TaBu be conducted during the early design stages of tall buildings. These initial assessments can provide valuable guidance for early design decisions, which should subsequently be validated through more refined FEMs to ensure greater accuracy and reliability.

Funding

The authors would like to express their sincere gratitude to the Giovanni Michelucci Foundation for granting permission to utilise the archive sources of Grattacielo Roma, Livorno 1956-1966. They also extend their heartfelt thanks to CDM Dolmen - Software di Calcolo Strutturale for their valuable contribution to the structural calculations presented in this study. Finally, the authors are deeply grateful for the sponsorship and financial support provided by Politecnico di Torino, Italy, which made this work possible through basic research funds.

Appendix. Comparison of stresses in wall W4

Following the displacement analysis, another important application in structural calculations involves the stresses acting on the bracing cores. It is important to clarify that Dolmen software cannot provide point-specific values at the vertices of 2D shell elements. Instead, it gives values at the centroids of the triangles that make up each shell element.

Therefore, for a given load case, a comparison was made between the TaBu model and the FEM using the average values calculated at the base of wall W4 (see Figure 5), which is the most stressed wall. The term ‘average’ refers to the mean of the values

calculated analytically at two vertices and at the midpoint of a section segment in the TaBu model.

As an example, the plan of a representative bracing element subjected to wind loads in the X direction is shown in Figure 18. The type of shear wall section is a Vlasov thin open section. In building studies, Vlasov-type sections are generally of greater interest because, due to their more varied configurations, they can sometimes be the only types present. The Michelucci Tower includes several vertical elements with Vlasov sections, but the most notable one is W4 (see Figure 5).

Stress calculation

Starting from the applied loadings and geometric properties entered into the TaBu software, the stresses acting on various sections of the bracing wall were calculated. Unlike the displacement results, which were provided directly by the software, these stresses required detailed analysis. The section shown in Figure 18 was used to extract the necessary information for this calculation. All data related to the section, including the coordinates of the centroid, the centre of shear, moments of inertia and so on are presented in Table 1.

Geometrical coordinate system and stress calculation

As indicated in Section 5.1, each vertical bracing element is located in its own reference system (origin at the shear centre, axes parallel to the principal axes of inertia). For this reason, all mixed moments of inertia were null. The stresses were then calculated at nine points in the vertices of the section using the following equations.

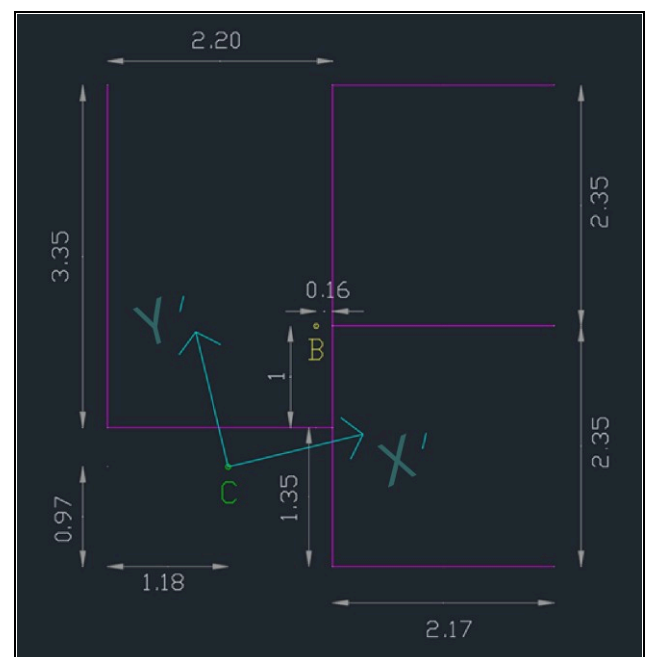


Figure 18. Schematic diagram of wall W4 section (dimensions in m)

Table 1. Data relating to the section

Parameter	Value
Thickness	
B	0.4 m
Centroid coordinates	
X_g	-6.64 m
Y_g	-2.42 m
Shear centre coordinates	
X_c	-7.5 m
Y_c	-3.8 m
Moments of inertia	
J_{xx}	16.04 m ⁴
J_{yy}	11.57 m ⁴
$J_{\omega\omega}$	15.56 m ⁶
J_{xy}	0
$J_{x\omega}$	0
$J_{y\omega}$	0
Torsional stiffness factor	
J_t	0.36 m ⁴
Principal axes rotation angle	
θ	13.56°

Normal stresses

$$6. \quad \sigma_z = \frac{M_x}{J_{xx}}y + \frac{M_y}{J_{yy}}x + \frac{B}{J_{\omega\omega}}\omega$$

where the terms M_x and M_y , and the bimoment (B) are known from the software.

Shear stresses

Following Vlasov theory, the shear stresses acting along the cross-section are:

$$7. \quad \tau_{(s)} = \frac{T_x S_y}{J_{yy} b} + \frac{T_y S_x}{J_{xx} b} + \frac{M_z^{VL} S_\omega}{J_{\omega\omega} b}$$

where the terms T_x and T_y , and the Vlasov moment (M_z^{VL}) are known from the software.

Following Saint-Venant theory, the shear stresses acting along the cross-section are:

$$8. \quad \tau_{(s,T)} = \frac{M_z^{SV}}{J_t} b$$

The static moments S_y , S_x and S_ω in Equation 7 must also be calculated analytically. For the sake of simplicity the calculations of their values are omitted here, but can be found elsewhere (Carpinteri *et al.*, 2010).

Table 2. Stress characteristics at the base of wall W4

Quantity	Value
M_x : kN.m	-11037.17
M_y : kN.m	28612.86
B : kN.m ²	-753.97
T_x : kN	1392.74
T_y : kN	-563.32
M_z^{VL} : kN.m	93.41
M_z^{SV} : kN.m	0

The stress characteristics at the base of wall W4, due to wind loads acting in the X direction, are reported in Table 2.

Referring to Figure 19, which shows the numbering of the section edges, the normal stresses at the base of wall W4 were then determined, as shown in Table 3. The values shown in Table 4 were obtained for the Vlasov shear stresses.

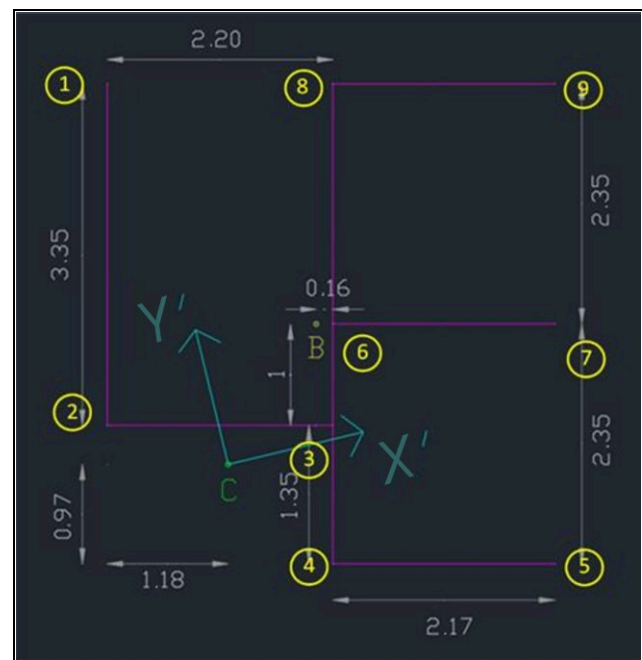


Figure 19. Numbering of the edges of the shear wall W4 (dimensions in m)

Table 3. Normal stresses at the base of wall W4

Point	σ_{M_x} : kN/m ²	σ_{M_y} : kN/m ²	σ_B : kN/m ²	σ : kN/m ²
1	-1617.04	-5044.96	147.79	-6514.22
2	688.10	-5044.96	-43.61	-4400.47
3	688.10	395.68	-3.39	1080.39
4	1617.04	395.68	63.96	2076.69
5	1617.04	5762.14	-39.25	7339.93
6	0	395.68	-52.33	343.35
7	0	5762.14	91.58	5853.72
8	-1617.04	395.68	-168.62	-1389.98
9	-1617.04	5762.14	222.41	4367.51

Table 4. Vlasov shear stresses

Point	τ_T : kN/m ²	τ_{T_x} : kN/m ²	τ_{M_x} : kN/m ²	τ : kN/m ²
1	0	0	0	0
2	821.56	79.02	21.61	922.19
3 → 2	1068.33	1.76	15.01	1085.09
3 → 4	-352.10	-258.13	8.25	-601.97
3 → 6	-716.23	256.37	-23.26	-483.12
4	-325.01	-179.11	3.30	-500.82
5	0	0	0	0
6 → 3	701.19	-274.81	19.96	446.34
6 → 7	-325.01	0	5.10	-319.91
6 → 8	-376.17	274.81	-25.06	-126.43
7	0	0	0	0
8	-325.01	179.11	7.20	-138.70
9	0	0	0	0

Regarding the shear stresses according to Saint-Venant’s theory, since the torsional rotation of the wall is zero at the base, the torsional moment is also zero. Therefore, at each point at the base, we have:

$$9. \quad \tau_{(s,T)} = 0 \text{ kN/m}^2$$

Considering the normal stresses and noting that the wall analysed was the one between vertices 1 and 2 (see Figure 19), the results obtained from the FEM graphical output, generated using Dolmen, are shown in Figure 20. Considering the shear stresses, the results obtained between vertices 2 and 3 from the FEM are shown in Figure 21.



Figure 20. Section at the base of the wall depicted in the Dolmen FEM, bounded by vertices 1 and 2, subjected to wind load acting along the X direction, with normal stresses in daN/cm²



Figure 21. Section at the base of the wall, depicted in the Dolmen FEM, bounded by vertices 2 and 3, subjected to wind load acting along the X direction, with shear stresses in daN/cm²

Comparison of average values obtained from the FEM and the analytical model

By converting all the values obtained in the FEM to kN/m², it was possible to proceed with the analytical method to calculate the average stresses between the endpoints of side 1–2 and the midpoint, as well as between the endpoints of side 2–3 and the midpoint. These values were based on the data listed in Tables 1–4. From the comparison, it is evident that the overall differences between the average stresses computed from the two models at the base of wall W4 were approximately 9.66% for the longitudinal stresses and 4.78% for the shear stresses.

References

- Capurso M (1981) Sul calcolo dei sistemi spaziali di controventamento, parte 1. *Giornale Del Genio Civile* **1**–2–3: 27–42 (in Italian).
- Carpinteri A and Carpinteri AN (1985) Lateral loading distribution between the elements of a three-dimensional civil structure. *Computers & Structures* **21**(3): 563–580.
- Carpinteri A, Lacidogna G and Puzzi S (2010) A global approach for three-dimensional analysis of tall buildings. *The Structural Design of Tall and Special Buildings* **19**(5): 518–536.
- Carpinteri A, Lacidogna G and Cammarano S (2013) Structural analysis of high-rise buildings under horizontal loads: a study on the Intesa Sanpaolo tower in Turin. *Engineering Structures* **56**: 1362–1371.
- Carpinteri A, Lacidogna G and Cammarano S (2014) Conceptual design of tall and unconventionally shaped structures: a handy analytical method. *Advances in Structural Engineering* **17**(5): 767–783.
- Carpinteri A, Lacidogna G and Nitti G (2016) Open and closed shear-walls in high-rise structural systems: Static and dynamic analysis. *Curved and Layered Structures* **3**(1): 154–171.
- CDM Dolmen (2025) See <https://cdmdolmen.it/> (accessed 20/10/2025).
- Fabbrizi F (2016) The variable city and its icon. The Livorno skyscraper by Giovanni Michelucci. *Firenze Architettura*: 88–93.
- Fantilli A and Ferraro E (2020) The strength of concrete cast in the first half of the 20th century (in Italian). *Ingenio*. See <https://www.ingenio-web.it/> articoli/la-resistenza-dei-calcestruzzi-confezionati-nella-prima-meta-del-900/ (accessed 09/12/2025).
- GMF (Giovanni Michelucci Foundation) (2025) Grattacielo Roma, Livorno 1956-1966. See <http://db.michelucci.it/archivi/progetti/?opera=125> (accessed 20/10/2025).
- Lacidogna G (2017) Tall buildings: secondary effects on the structural behaviour. *Proceedings of the Institution of Civil Engineers – Structures and Buildings* **170**(6): 391–405.
- Luseroni F (2010) *Giovanni Michelucci and the Vertical City: The Livorno Skyscraper*. ETS Editions, Pisa, Italy.
- MIT (Ministero delle Infrastrutture e dei Trasporti) (2018) NTC 2018: Nuove norme sismiche per il calcolo strutturale, DM 17 January 2018. MIT, Rome, Italy (in Italian).
- Nitti G, Lacidogna G and Carpinteri A (2021) An analytical formulation to evaluate natural frequencies and mode shapes of high-rise buildings. *Curved and Layered Structures* **8**(1): 307–318.
- Verderame GM, Ricci P, Esposito M and Sansiviero FC (2011) Mechanical properties of steels used in reinforced concrete structures built from 1950 to 1980. In *Proceedings of the 26th National AICAP Conference, Padova, Italy*.
- Vlasov VZ (1961) *Thin Walled Elastic Beams*. National Science Foundation, Washington, DC, USA.
- Zalka KA (2001) A simplified method for calculation of the natural frequencies of wall-frame buildings. *Engineering Structures* **23**(12): 1544–1555.

How can you contribute?

If you would like to comment on this paper, please email up to 200 words to the editor at support@emerald.com.

If you would like to write a paper of 5000 words about your own experience in this or any related area of civil engineering, the editor will be happy to provide any help or advice you need.