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DESIGN OF A TALL MASS TIMBER TOWER FOR OCCUPANT COMFORT UNDER WIND-INDUCED ACCELERATIONS

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ABSTRACT: The current trend of increasing the height limits of timber buildings makes wind-induced accelerations for occupant comfort an important design parameter. This article presents the two-step process used for the design of a 51m tall mass timber tower located in Sweden. The lateral stiffness of the tower is provided through a CLT inner core and a braced glulam post-and-beam system along its outer periphery. The structural scheme, the dynamics of high-rise timber structures, the 3D modelling assumptions to capture the global stiffness with suitable accuracy as well as the analytical state-of-the-art methods used to determine the wind-induced accelerations in the along-wind, across-wind and torsional direction at the top of the tower are presented. Through a parameter study, the sensitivity of the structure to changes in connection stiffnesses and mass distribution is assessed. Given the uncertainty inherent to the analytical methods, the calculations are validated in a second step against the acceleration response obtained from wind tunnel testing. The differences between the analytical and experimental approaches are compared and the key parameters are discussed.

KEYWORDS: Tall Mass Timber Building, Wind-induced Accelerations, Occupant Comfort, Parameter Study, Design process, 3D Modelling, Wind Tunnel Testing

1 INTRODUCTION

For the *Embassy of Sharing* project in Malmö, Sweden, a total of seven buildings are in planning. The *Fyrtornet* is the first of these buildings and with a height of 51m, will be Malmö's first and Sweden's tallest office building in mass timber. This 11-story building, a landmark for progressive and sustainable construction built entirely in timber above ground, consists of offices with public functions on the two lower floors. The wooden structure is visible from the outside through the glazed facades. The structure is expected to be completed in 2023.

Figure 1: *Architectural rendering of the Fyrtornet Tower*

2 STRUCTURAL SYSTEM

2.1 OVERVIEW

The gravity system consists of CLT slabs that span from the timber core across an intermediary and an outer post-and-beam system in glued laminated timber. The only exception is at the 11th floor, where a concrete slab is used to add mass to control the wind-induced vibration levels. The lateral bracing of the building is achieved via the combined action of an inner CLT core and the outermost braced post-and-beam structure. To accommodate the sloped geometry of the two glazed facades facing south and east, a horizontal transfer is introduced at the 6th level. Vertical transfers occur on the 2nd and 8th floor.

2.2 STRUCTURAL LAYOUT

The following sections and plan views from the building show the structural layout of the CLT slabs, the CLT core and the post-and-beam structure in glued laminated timber. The columns are spaced on a regular 4.8m grid in both directions. The main structural elements are located between gridlines A1-A6 and B1-B6. Within these gridlines, all columns and diagonals are directly supported on the reinforced concrete foundations. Between gridlines

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A6-A7 and B6-B7, the vertical loads from the upper levels are brought to the foundation through cantilevering beams on the first floor, supported by inclined columns. This design feature allows for a protected pedestrian area. Due to the sloped geometry of the south and east facades, the floors decrease in surface as higher levels are reached. On every third floor starting from level 2, there is an accessible terrace just behind the glass facade. The next 2 levels are then identical but without any terrace.

Figure 2: *Axonometric overview of timber structure*

The CLT slabs span over two bays, except around the CLT core where they are simply supported. The top faces of the slabs are either flush or laid on top of the glued laminated timber beams. This provides the required space for the main ventilation ducts coming from and running around the CLT core. On the first floor, due to the library usage, 7-ply CLT slabs with a thickness of 240mm are used. The slabs on all the upper floors are 5-ply with a thickness of 180mm. The inner core is built-up from 125cm wide and 280mm to 320mm thick CLT panels.

To accommodate the geometry of the sloped facades, the diagonal bracing, which contributes to the overall lateral stability of the building and provides the necessary torsional stability around the CLT core, is shifted from axes A6/B6 to A5/B5 respectively. To ensure structural continuity of the horizontal load transfer across the height of the building, the horizontal loads are transferred from the bracing on axes A5/B5 above level 6 to the shifted bracing on axes A6/B6 and the CLT core on axes A4/B4 on the lower levels through the CLT slabs, which form a diaphragm at each level. The vertical loads are directly transferred through the columns located on gridlines A2/B5, A4/B5, A5/B2, and A5/B4.

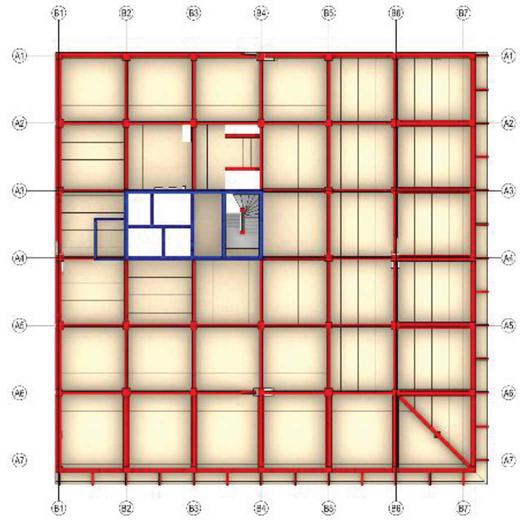

Figure 3: *Level 1 plan view (View from below - Core in blue, beams and columns in red)*

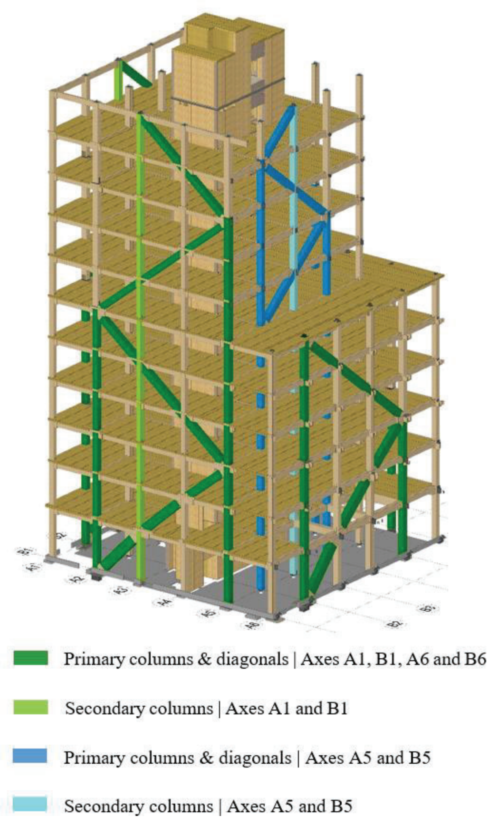

Figure 4: Axonometric view of the glued laminated bracing system

On axes A1 and B1, the glazed facade is vertical so that the bracing system is continuous across the height of the building. To reduce the amount of steel connections, the primary columns are, in general, continuous over 2 to 3

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levels. Secondary columns span, in general, from one level to the next.

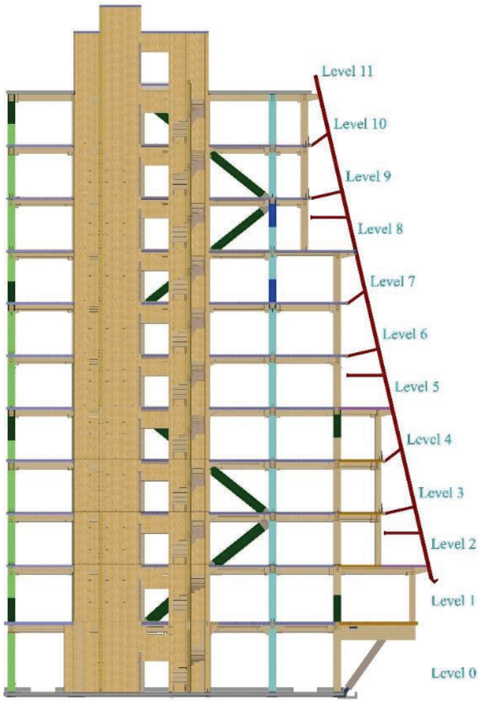

Figure 5: *East-West section of the Fyrtornet Tower*

The cross sections of the bracing diagonals (GL28h) are 560x920mm on the first floor and reduce to 560x600mm on the top floor. The main columns of the bracing system have a constant cross-section of 560x560mm and 440x560mm for the shifted bracing beams from level 6 onwards while the secondary columns are 440x560mm wide. These columns and diagonals are defined as “key elements” according to the Swedish national annex to the Eurocode 1-7 and are therefore designed with an additional safety coefficient of 1.3.

The sloped glazed facades start from the 2nd floor and extend through the entire height of the building. The design of the facade was outside the design scope but a close exchange with the facade designers was necessary to assess the weight distribution on the building and to coordinate the geometry for the assessment of the wind induced loads.

2.3 CONNECTION DESIGN

The bracing diagonals located on the ground floor must sustain a characteristic load of approximately 4000kN. The joints are realized with a dowel-type connection with multiple shear planes. The design of these connections is based on Eurocode 5 [1] and the related Swedish national annex as well as the recommendations given in reference [2]. The connection between the timber elements and the foundation follows the same principle and is achieved through a multiple shear plane connection (4 slotted-in steel plates) with dowel-type fasteners. The large cross-section sizes were achieved by block gluing three beams together.

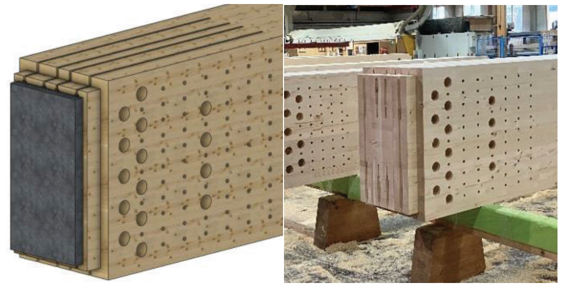

Figure 6: *Connection design (Cadwork 3D) and bracing glulam beam in the factory.*

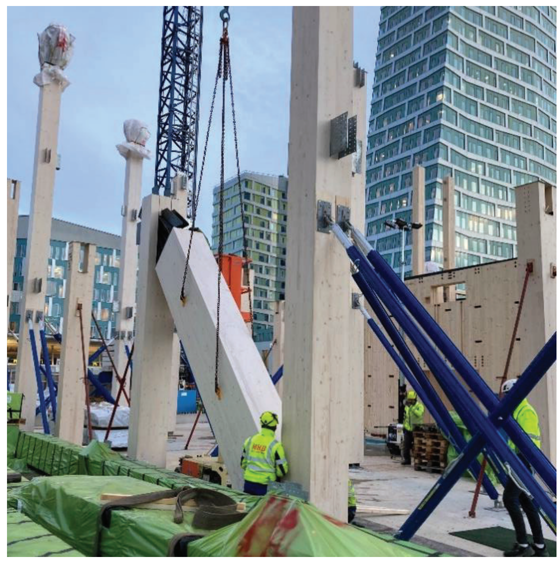

Figure 7: *Bracing beam during installation*

The slab-to-slab connections are realized with a rabbet and a screwed 27mm, 3-ply coverboard. On level 6, these connections are replaced by nailed perforated steel plates to transfer the horizontal loads coming from the shifted bracing diagonals. The 125cm wide CLT core-wall panels are joined with a steel slot connector fixed with inclined auxiliary screws that tighten the panels against each other. This design allows for a very stiff connection and therefore generates a near-monolithic behaviour.

3 DESIGN FOR VIBRATION

3.1 APPROACH

At an early design stage, wind-induced comfort at the top of the tower was recognised as an important design driver. To assess the influence of various design decisions early in the process, a set of analytical tools based on the relevant construction codes and state-of-the-art literature were developed to allow for a quick assessment of the wind-induced accelerations at the top of the tower and to provide feedback to the design team. Once the calculations from these initial calculations showed satisfactory results, the accelerations were subsequently checked in a second step with the input from the data generated from a wind tunnel test. Sufficient time was allowed to make final adjustments to the structure if needed and to steer the dynamic properties in the right direction in order to meet the required comfort criteria.

3.2 COMFORT CRITERIA

For high-rise buildings, and in particular for tall mass timber buildings such as the Fyrtornet tower, which have a low mass compared to their traditional concrete or steel counterparts, it is important to assess if the horizontal accelerations on the top occupied floors under (turbulent) wind action might be disagreeable or even unacceptable. The perception of acceleration levels is dependent on many factors and is to some extent subjective. Furthermore, the planned usage of the building must be considered. For residential buildings, more restrictive acceleration criteria apply than for office buildings.

Of interest are, in general, the peak accelerations expected to occur only a few times over the lifetime of the building, (for instance during a windstorm) as opposed to the root-mean-square values which are also sometimes used. The common criteria for the perception of acceleration relates the acceleration levels to the frequency of occurrence. Following the generally accepted criteria for high-rise buildings given in [3], the wind-induced horizontal peak accelerations should not exceed the values given in table 1 below.

Table 1: *Generally acceptable 10-year peak accelerations depending on the usage of the building [8]*

Planned usage	Generally acceptable accelerations
Office	2.0 – 2.5 %-g
Hotel	1.5 – 2.0 %-g
Residential	1.0 – 1.5 (to 1.8) %-g

g denotes gravitational acceleration

Extensive examinations indicate that the people's sensitivity to motion becomes less as the frequency of motion (eigenfrequency of the building) decreases. This dependency is not fully reflected in the criteria mentioned above. The criteria in [3] were developed primarily for tall buildings which tend to have natural frequencies in the range of about 0.15 Hz to 0.30 Hz. Following the criterion of the International Organisation Standardization (ISO 10137:2007) [4], the dependence of the frequency is considered in the range of interest between 0.06 Hz and 1 Hz. The published criteria are based on a one-year wind and the peak accelerations of the considered building should not exceed the evaluation curves given in ISO 10137: 2007 – Annex D [4], reproduced below for convenience.

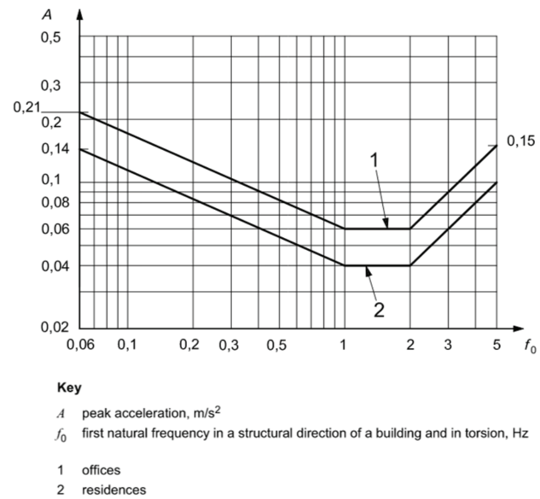

Figure 8: *ISO 10137: 2007 – Annex D: Evaluation curves for wind-induced vibrations in buildings in the horizontal direction for a 1-year return period.*

The acceleration limits in [4] need to be satisfied for peak accelerations resulting from a 1-year return period wind in the along-wind, across-wind, and torsional direction. In many cases, however, the fundamental translational and torsional modes are not independent, and the superposition of these modes will result in higher accelerations than from each mode alone. Although there is little information in the technical literature on how to address the combined acceleration from different modes of vibration, this was considered in both the analytical and wind tunnel assessment. This is described in more detail in subsequent sections.

Finally, it is important to note that the threshold of vibration perception is around 0.5%g. In other words, even if the limiting acceleration criteria above are met, wind induced accelerations will be felt by the occupants under strong winds, but the levels are limited such that with a high likelihood, they do not cause discomfort.

3.3 3D FE MODEL AND KEY PARAMETERS

The creation of a reliable global FE model was critical to properly assess the wind-induced accelerations, which strongly depend on the natural frequencies and mass distribution of the building.

3.3.1 Workflow

Two important criteria for the modelling were the geometrical accuracy and a precise representation of the structural system. For the former, the procedure consisted of extracting the geometry as well as the relevant structural attributes from the 3D Cadwork construction model provided by Binderholz. Centerlines of the structural elements, material grades and cross-sections were retrieved and referenced in the modelling Software Rhinoceros 3D from Robert McNeel & Associates and its visual programming interface Grasshopper. The finite-element model was then created using the Grasshopper

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interface to the finite-element software SOFiSTiK AG This workflow allowed for a precise and seamless translation from a "Solid" 3D geometry to the needed line and surface geometry. Furthermore, relying on a semi-automated process greatly improved the speed and flexibility of the information transfer while reducing the risk of errors.

3.3.2 Connection stiffness

To capture the natural frequencies and associated vibration modes of the building with sufficient accuracy, it is important to consider the loss of stiffness generated by the mechanical connections (a) between the 1.25m wide CLT panels in core walls (b) between the CLT panels that make up the floor diaphragms and (c) the diagonals in the peripheral bracing elements. The connections between the post-and-beam elements were considered ideally articulated.

The approach chosen was to generate equivalent material properties with reduced shear stiffnesses (G-modulus) rather than representing the detailed plate discretization in the global model. To estimate the equivalent material properties, sub-models were generated, and the key behavioural parameters extensively studied. For the walls (
Figure 9), the actual panel partition was considered together with the connector stiffness. In addition, the stiffness of the hold-down bracket and the bottom shear connector were also integrated in the model to accurately capture the wall behaviour. An iterative procedure was then undertaken to obtain the same lateral and rotational displacements between the detailed wall model and the simplified one based on equivalent material properties. For the floors (
Figure 10), the full discretization of the constitutive panels and the associated spline connector stiffness was modelled and evaluated to determine the equivalent material properties. A similar strategy was implemented to capture the stiffness behaviour from floor to floor where the hold-down and shear brackets were modelled as continuous springs rather than discrete elements while maintaining the overall wall in-plane rotational behaviour.

This approach had the advantage to greatly simplify the global modelling while maintaining the overall behaviour sufficiently accurate. This proved to be an effective strategy to perform fast and reliable parameter studies.

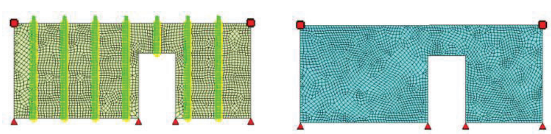

Figure 9: *Walls sub FE-models used to tune the equivalent shear stiffness (G-Modulus)*

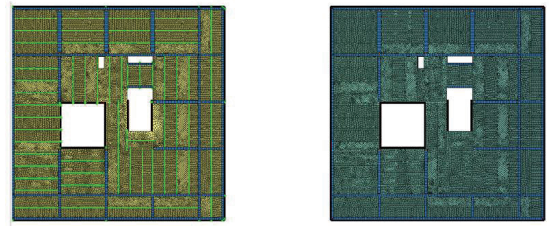

Figure 10: *Floor sub FE-models used to tune the equivalent shear stiffness (G-Modulus)*

To assess the effect of the basement and the foundation on the overall stiffness, the soil and anchorage stiffness provided by the local engineers responsible for the foundation design were considered at an early stage in the model. Considering that the connections had a high stiffness coupled with the fact that the soil stiffness is, in general, significantly higher under dynamic loads (increased by factor of around 4 to 6 compared to its stiffness under static loads [5]), the differences in natural frequencies compared to an idealized rigid support proved to be insignificant.

3.3.3 Analytical assessment of vibrations

The methods given in the Eurocodes and its national annexes are restricted to the calculation of the wind-induced accelerations in the along-wind direction. Furthermore, these methods are approximations that cover only simple geometries, and the effect of the surrounding buildings is not considered leading to uncertainties in, for instance, the force coefficients and peak factors. Moreover, as pointed out in the comparative study in [6], the methods according to the EKS 11, BFS 2019:1 [7] yield a significantly higher accelerations than those presented in EN 1991-1-4 [8]. The Eurocode framework does not provide methods to calculate the across-wind accelerations nor those arising from torsional modes. Furthermore, no guidance is given to account for the superposition of these components. To be able to steer the design into the right direction and assess the effect on the wind-induced vibrations early in the design, the approach described below was adopted.

To calculate the accelerations in the along-wind direction, the method in EN 1991-1-4 [8] was used according to the framework set-out in Annexe B. This method has also been employed in recent studies [6][9][10][11] to assess the wind-induced acceleration of tall mass timber buildings to meet the ISO 10137:2007 [4] comfort standards. In particular, based on the comparative studies in [6], the input parameters according to the EKS 11, BFS 2019:1 [7] were used for the reasons already mentioned above. The wind-induced accelerations in the across-wind direction were calculated according to the Canadian National Code NBCC [12], with the basic wind input data brought back into the Eurocode framework. The accelerations from the torsional modes were calculated according to the principles given in [13]. Also, the superposition of all three components as a square-root-of-sum-of-squares, with an overall reduction factor of 0.8 according to [13] was assessed to account for the fact that the peaks of all three components are unlikely to occur simultaneously.

3.3.4 Parameter study

To assess the effect of the connection stiffness and mass distribution over the height of the building on the wind-induced vibrations of the tower and use this as an input to inform the structural design, a parameter study was undertaken. For each set of parameters, the wind induced accelerations at the last occupied floor of the tower were computed following the method outlined in the analytical assessment given in the previous section i.e., the accelerations from the first three modes of vibration (along-wind, across-wind and torsion) as well as their superposition. The sensitivity of the vibration response to the various parameters could be determined and changes made where they are the most effective.

The wind-induced accelerations based on the modal parameters obtained from the global finite element model were computed by considering the variation of the following parameters: a) Inter-panel shear connection between wall elements b) Inter-panel shear connection between floor elements c) Tie-down anchors level to level d) Horizontal shear connection level to level e) Tie down anchors level to basement f) Horizontal shear connection level to basement g) Damping and h) Mass distribution over the height of the building.

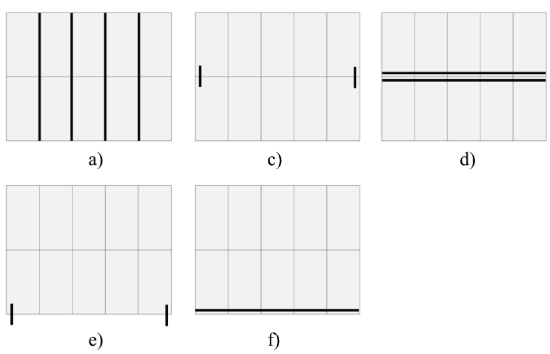

Figure 11: Schematic illustration of the different parameters at the walls between two levels.

For a constant damping value, it was found that the mass distribution and the shear connection between the CLT-panels in the core walls (Figure 11 - a) were the driving parameters. The stiffness of the tie-downs (Figure 11 - d, f) and the horizontal shear connection (Figure 11 - e, g) had only a marginal effect on the calculated accelerations. The reason was that under the 1-year and 10-year return period winds, the CLT core walls remained largely in compression. In general, it was found that a variation of the connection stiffnesses of around 20% from the base design values would not lead to a significant increase in acceleration levels. This was also the case for the connection stiffness of the bracing elements. To satisfy the ULS requirements, these connections inherently exhibited a very high stiffness.

While increasing the mass in the top third of the building decreases the natural frequencies, wind-induced peak accelerations could efficiently be decreased by this measure. This effect has been used to influence the floor build-ups in the top floors and to set out the thickness of the concrete slab on the last level to meet the required comfort criteria.

Figure 12 below shows the robustness of the calculated acceleration levels with the changes in stiffness and mass parameters towards the end of the design stage. While the acceleration levels from each individual mode were met with sufficient margin, the superposition tended to exceed the criteria given in ISO 10137:2007 [4]. However, given the uncertainties inherent in this computation and the lack of guidance available, it was chosen to give the modal parameters of the structure (see Figure 13) to the wind engineers and assess the accelerations via wind tunnel testing on this basis.

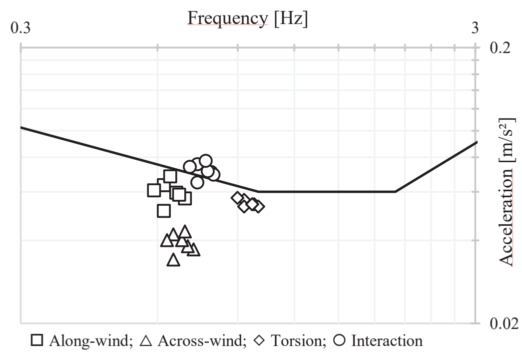

Figure 12: *Summary of the parameter study near design completion showing the first three natural frequencies (translational and torsional modes) and the resulting acceleration as well as the acceleration limit for office buildings according to [4]*

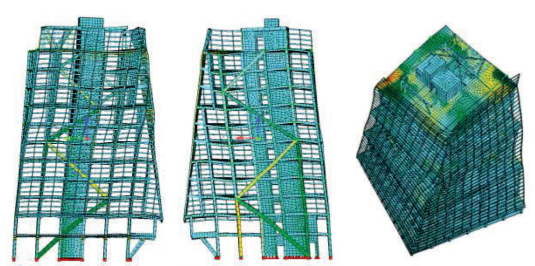

Figure 13: *First lateral mode-shape (0.72Hz), second lateral mode-shape (0.73Hz) and third torsional mode-shape (0.97Hz) from the finite-element model.*

3.3.5 Damping

The damping of a structure, expressed either as the logarithmic decrement δ or as the damping ratio ζ (% of critical damping), is a key parameter in determining the dynamic response of a structure subjected to dynamic loads. Yet, as opposed to stiffness and mass, it cannot be directly calculated, and the designer must rely on values that are code-based or given in state-of-the-art literature for similar types of structures.

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EN 1991-1-4 [8] provides a table in appendix F with damping values for various types of structures depending on type, material and connections used. However, the damping values given cover primarily different types of structures made from steel and reinforced concrete. For timber structures, only a single reference is given in this table, namely that for timber bridges with the damping expected to be in the range from δ =0.06 to δ =0.12. The ISO 10137 [4], from the which the acceleration limits are taken for the one-year return period wind, gives damping values of ζ =0.01% for steel frame buildings and ζ =0.02% for concrete frame buildings. Timber buildings are not covered.

The major contributors to overall damping are (i) material damping (ii) structural damping and (iii) damping due to friction from relative movement of non-structural elements (e.g., screed and cladding). In addition, aerodynamic damping i.e., energy dissipation as the building moves through the air may also be considered according to EN 1991-1-4 [8] but was omitted here on the conservative side.

Due to the fact that (a) the Fyrtornet tower consists of a CLT core and slab diaphragms made up from 1.25m wide panels connected with a significant number of dissipative connectors (b) on the periphery, bracing is provided by a braced post-and-beam system, joined with dowel-type connections across multiple shear planes (c) each floor has at least 4cm of floating screed and (d) the building is closed with a timber façade system, it is expected that the overall damping of the tower will be significant. Most studies that address the issue of wind-induced vibrations tend to take damping ratios in the vicinity of 2% of critical, see for instance [6][9][10][11]. Damping ratios from measured examples show a significant scatter: minimum values tend to be around 1.3% while high damping ratios around 3%-4% and values in excess of 5% have been reported [14][15][16]. The large scatter can be attributed to the type of lateral resistance system and its connections, the inclusion of façades and other non-structural elements during measurements as well as the amplitude of the vibration levels. Measurements in [17] showed that for large vibration amplitudes, defined as > $0.05 m/s^2$ i.e., in the perceptible range but still below the ISO 10137 limits, the damping ranged from 2.5%-3.5%, depending on the mode. In [15][16], damping values across different structures with different lateral bracing systems are listed, also with measurements after non-structural elements have been included. For post-and-beam systems with a CLT core, the measured damping ranges between 2-3% [16]. In the conclusion of [16], the following recommendations based on the pool of measurements taken are given: for a post-and-beam structure a damping ratio of 1.9% may be used while for a CLT structure, a damping ratio of 2-2.5% may be used. For hybrid structures, damping values are in the range between 2-3% of critical.

Based on the above literature research, while staying within the limits given in EN 1991-1-4, a damping of

1.9% of critical or δ=0.12

was chosen for the 1-year and 10-year wind criterion given that the lateral bracing system of the Fyrtornet consist of both a braced post- and-beam system and a CLT core and diaphragm system.

4 WIND TUNNEL TESTING

To eliminate the uncertainties and unknowns outlined in the analytical assessment, the wind-induced accelerations at the last occupied level of the tower were computed with the wind tunnel generated data.

4.1 WIND TUNNEL SET-UP

For the pressure measurements, a rigid model of the tower in a scale of 1:200 was constructed. The tests were carried out in two configurations: (1) the Fyrtornet tower as a stand-alone solitary building without any additional surrounding buildings and (2) in a configuration including all significant existing and proposed future buildings in the near vicinity within a radius of approximately 200 metres. Surroundings beyond the immediate neighbourhood of the building are simulated by appropriate roughness elements on the tunnel floor in order to provide an accurate simulation of the wind speed profile and turbulence of the approaching wind. The facades of the tower were equipped with about 150 pressure taps and the time-series were recorded simultaneously by means of high-response differential pressure transducers.

Figure 14: *Wind tunnel test set-up with local environment*

4.2 CALCUATED ACCELERATIONS

As a basis for the wind-induced vibration calculation based on the wind-tunnel measurements on the rigid model, a simplified linear analytical model was established that correctly represents the mass distribution and modal properties of the tower as established in the more complex 3D finite-element model.

The main steps for the computation of the dynamic response of the tower can be summarized as follows: (1) calculation of the generalized force-time series based on the measured pressure time-series and the mode shape information, (2) Fourier transformation of the generalized force-time series and multiplication with the mechanical admittance function for the respective mode shapes. This results in the complex generalized deformation spectrum. On this basis, the deformation and acceleration time-series are computed. (3) Extreme value analysis of the acceleration time-series.

For the vibration response calculation, a logarithmic damping decrement of δ = 0.12 was assumed to be representative for 10-year and 1-year winds. The predicted peak accelerations in the upper floors of the tower are:

$$a_{10a,peak}\,(\delta=0.12) = 2.40\%\text{- g}$$

for the 10-years-wind and

$$a_{1a,peak}\,(\delta=0.12) = 0.70\%\text{-g}$$

for the 1-year-wind.

These accelerations occur on the outer corner range of the last occupied floor level (Level 20; 39.6 m above ground) and include the contribution of the along wind, crosswind, and radial accelerations. The values represent the maximum peak accelerations of all investigated wind directions and constellations. Considering the natural frequencies of the building, both the 10-year and the 1-year accelerations are within the acceptable limits for an office building according to the comfort criteria described above.

It is important to note that, although the wind tunnel analysis seems to validate the code-based analytical calculations, the reasons for the reasonably good match are not obvious. In fact, the general wind climate on site proved to be less onerous than the wind speeds given in the code. On the other hand, the peak acceleration values given above arise predominantly from one direction in which the presence of one of the tall buildings next to the Fyrtornet plays a significant role.

5 SITE ASSMEBLY

The assembly sequence starts with the CLT core. Walls are pre-assembled on the floor and are then installed as one monolithic element. Once the core over the height of one storey is assembled, the glulam structural elements are installed: first all the columns, which are temporarily propped, then the bracing and finally the horizontal beams.

In the next step, the CLT slabs are installed and act as a temporary water protection. A moisture protective, diffusion open, membrane is already applied to the slabs in the factory and only some minor rework on-site is required. Keeping timber as dry as possible during assembly is one of the key elements to ensure a good quality to the building. This sequence of steps is then repeated for each storey.

The timber structure assembly started in November 2022 and is planned be completed by June 2023.

Figure 15: *On site situation the 6th of December 2022. Picture taken from south-east.*

Figure 16: *On site situation the 21st of February. Picture taken from south-west. Timber structure levels 1 to 4 is completed.*

6 CONCLUSIONS

The two-step approach taken to assess the wind-induced accelerations for comfort on the Fyrtornet tower proved to be a reliable and necessary strategy.

In the first step, the analytical assessment of the acceleration at the last occupied floor of the tower based on methods given in codes and state-of-the-art literature, coupled with a parameter study on the influence of the connections stiffnesses and mass distribution on the accelerations allowed to steer the structural design in the right direction. However, the code-based methods that are available to date do not provide comprehensive guidance on how to compute the wind-induced vibrations for the across-wind and torsional modes and no information is given on how the response from individual modes is superposed. Also, recommendations for damping values applicable for tall mass timber buildings are not existent in the current European codes so that an extensive literature research of the current state-of-the-art must be undertaken. Additional research in this field is needed.

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Given that analytical methods ultimately remain idealisations for simple geometries and cannot fully account for the effects of the wind effects from adjacent buildings, it is recommended to assess the wind-induced vibrations in second step through wind-tunnel based methods in order to yield a reliable and yet efficient structural design.

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