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The Structural Engineering Design And Construction Of The Tallest Building In Europe Lakhta Center, St. Petersburg. Russia

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Abstract

The Lakhta Center is a Multifunction Complex Development (MFCD) consisting of 1) an 86 story office tower rising 462 m above the ground to provide high-end offices for Gazprom Neft and Gazprom Group affiliates 2) a Multi-Function Building (MFB) that includes, a scientific/educational center, a sport center, a children's technopark, a planetarium, a multi-transformable hall, an exhibition center, shops, restaurants, and other public facilities 3) a Stylobate 4) "The Arch, which forms the main entrance to the tower, restaurants, and cafes 5) underground parking and 6) a wide range of large public plazas. While each of the MFCD buildings is technically challenging in its own right, the focus of the paper is to present the development and integration of the structural and foundation systems of the bowed, tapered, and twisted shape of the tower into the fabric of the tallest Tower in Europe.

Keywords: Super Tall Tower, Composite Structural system, Composite floor framing design, floating raft on reinforced soil structure, Composite Outrigger Core wall Connection with externally stiffened floor diaphragm

1. Introduction

The Lakhta Center (LC), situated in the Primorsky district at the outskirts of Saint Petersburg, is developed and implemented as a business center and a pilot of integrated sustainable development, with the public and city's interest at heart. LC is strategically located to have the connectivity and access to all public infrastructure and to serve as a catalyst to the development of a 21st-century new landmark and iconic business center that is well suited to expand and complement the existing historic and world heritage Central Business District (CBD). This in turn will bring significant economic benefits to the city such as attracting global corporations, adding revenue and creating new jobs. With careful planning, the new CBD has the potential to become one of the most sustainable and iconic cities in the Northern Hemisphere, as seen in Figure 1.

The Lakhta Center is a Multifunction Complex Development (MFCD) consisting of: 1) an 86 story office tower rising 462 m above the ground to provide high-end offices for Gazprom Neft and Gazprom Group affiliates 2) a Multi-Function Building (MFB) that includes, a scientific/educational center, a sport center, a children's technopark, a planetarium, a multi-transformable hall, an exhibition center, shops, restaurants, and other public facilities 3) a Stylobate 4) "The Arch that forms the main Tower entrance and houses restaurants and cafes 5) underground parking and 6) a wide range of large public plazas. While each of the MFCD buildings is geometrically and technically challenging in its own right, the focus of this paper is to present the development and integration of the structural and foundation systems into the architectural fabric of the tallest building in Russia and Europe.

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2. The Complex Geometry of the Tower

The geometry of the Tower was influenced by the Swedish town of Niyen and Niyenscans Fortress as shown in Figure 2. The exterior geometry of the tower is sculpted around a central circular core wall with 5 equal extrusions/petals that rotate 90 degrees from the base to the top of the spire and bows/tapers relative to the tower geometric center. The overall dimension of the tower is approximately 65 m at the base, 67.3 m at level 17, 27.8 m at level 86, and diminishes at the tower pinnacle at

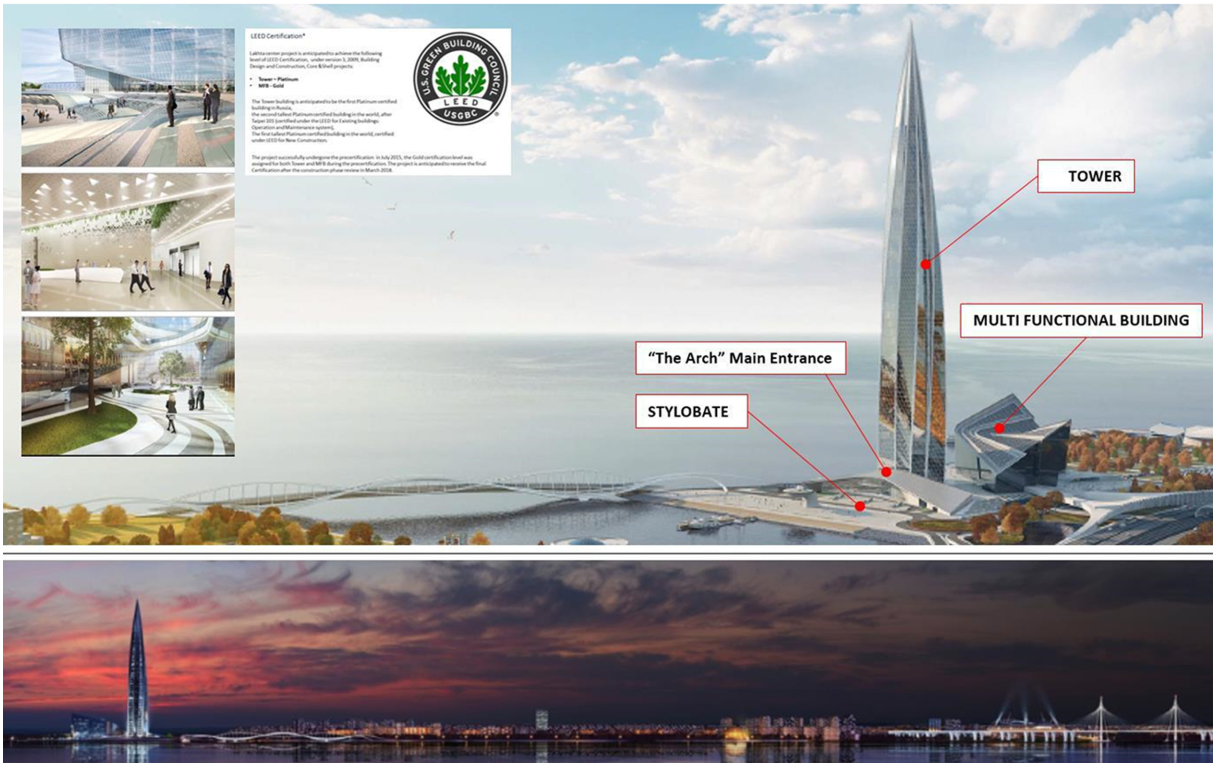


Figure 1. Rendering of Lakhta Center, city context, and of completed Complex.

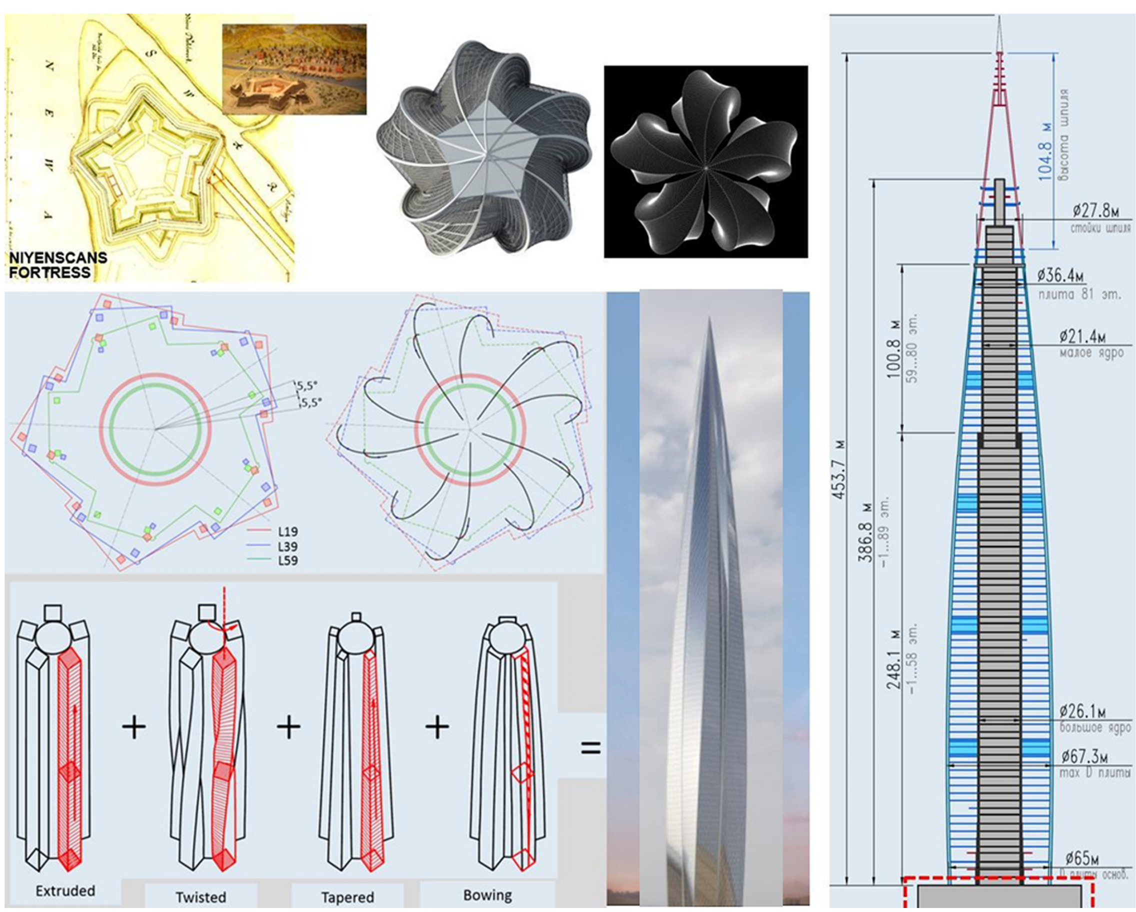


Figure 2. Niyenscans Fortress, Tower Geometry (extruded, twisted, tapered, and bowed).

462 m. The exterior structural skeleton is developed to weave into the ever-changing crystalline form of the architectural fabric as shown in Figure 2.

The center core wall dimensions are optimized to house the vertical transportation systems, stairs, mechanical shafts, elevator lobbies, and other technical facilities. The core wall is 26.1 m in diameter between levels 1 to 58, 21.4 m in diameter from level 59-80, and 16 m in diameter from level 81 to 88. The exterior composite steel columns follow the exterior geometry of the tower by twisting at 0.89 per floor, tapering and bowing from level 1 to the tip of the pinnacle as depicted in Figure 2. The structural steel braced frame provides stability and lateral load resistance to the spire. The main steel pipe columns of the spire are founded at the exterior composite columns at level 83 which then taper to a single central ring (as shown in Figure 2) that supports the single pipe pinnacle at 462 m.

3. Design Description of the Structural System of the Tower and Super-structure System Selection

The crystalline and complex geometry of the tower resulted in significant changes to the floor plate shape and floor area. This resulted in significant structural challenges that required simplifications and innovations in selecting structural systems that:

1. integrates the exterior structural frame into the crystalline architecture fabric
2. maximizes the exterior views and minimizes the number of exterior columns
3. continuous throughout the building height without transfers
4. integrates all vertical elements into the stability and lateral resisting system to maximize its resistance to wind, seismic, stability, etc. and maximizes the overall bending stiffness of the building
5. considers constructability and avoids complex detailing when designing for construction
6. provides a robust and redundant structural system in case of accidental events of loads.
7. minimizes effects of differential column shortening between center reinforced concrete core wall and the exterior columns
8. resolves biaxial design forces that are generated at the exterior columns due to change in building geometry (rotation, tapering) without additional cost or complex detailing
9. addresses wind effects and provides for wind engineering treatment to reduce wind loads and wind excitation due to the dynamic wind effects. Variation in building geometry at every floor, because of floor rotation, tapering, variation at edge conditions, and open/meshed/vented façade at the spire (above level 88), has resulted in favorable wind responses that are well within the internationally accepted

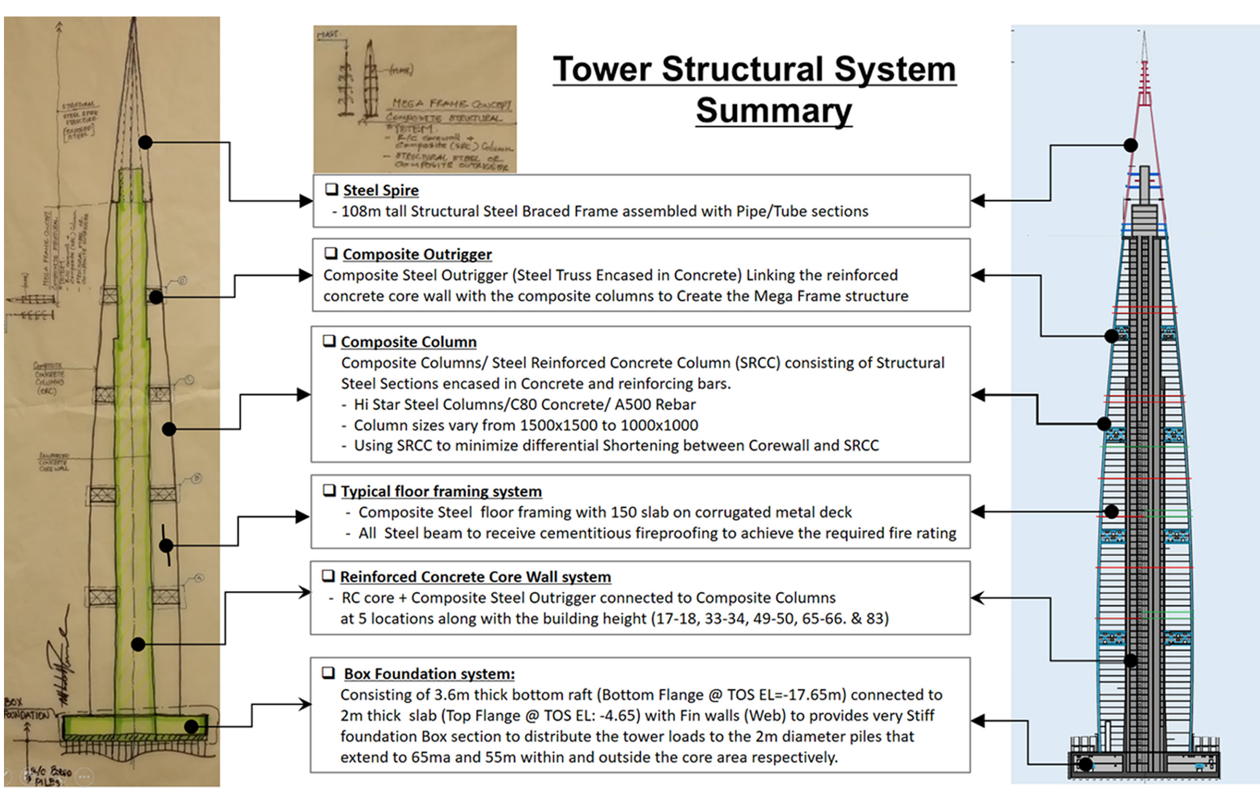


Figure 3. Tower superstructure and summary and description of the foundation system.

criteria thus eliminating the need for auxiliary supplemental damping system

10. addresses potential ice formation/loading at both the façade and the open meshed façade of the spire structure.

The above consideration has resulted in selecting a robust and redundant Mega Frame system as the primary lateral load resisting and stability frame for the tower superstructure. This comprises of centrally located and torsionally stiff reinforced concrete (R/C) core wall that is connected to ten (10) composite Steel Reinforced Concrete Columns (SRCC), at the perimeter of the building, through a series of two-story equally spaced composite outrigger trusses at five (5) levels as depicted in Figure 3. A series of options were considered to optimize the overall bending stiffness of the Mega-Frame structure by balancing the bending stiffness of the reinforced concrete core wall and the bending stiffness of the equally spaced composite outriggers, to that of the axial stiffness of the exterior SRCC (stays). This maximized the use of the vertical gravity load support structure to increase the bending stiffness of the Mega-frame and its resistance to the lateral loads (wind, seismic, notional) at minimal cost.

The center reinforced concrete core wall is rigidly connected to the composite outriggers through an externally stiffened reinforced concrete floor ring plate that transfers the top and bottom outrigger horizontal forces to the reinforced concrete core wall externally. This new and innovative core wall-outrigger connection is not only very efficient but has significantly improved the constructability aspects of the tower without the complex detailing associated with traditional outrigger structural systems.

SRCC, used in lieu of solid structural steel columns (proposed in the original design) are utilized to not only maximize the lateral stiffness of the Mega-frame structure, but also to optimize the design (cost & time benefit), improve the constructability of the exterior frame, and to minimize the differential shortening between the SRCC and R/C core wall and its impact on the composite outrigger design and other nonstructural components of the tower. Figure 3 provides an overall summary of the structural and foundation systems selected for the tower.

3.1. Floor Framing System at Typical Odd and Even Floors

Surrounding the core wall, the tower geometry is comprised of five (5) similar individual tower forms

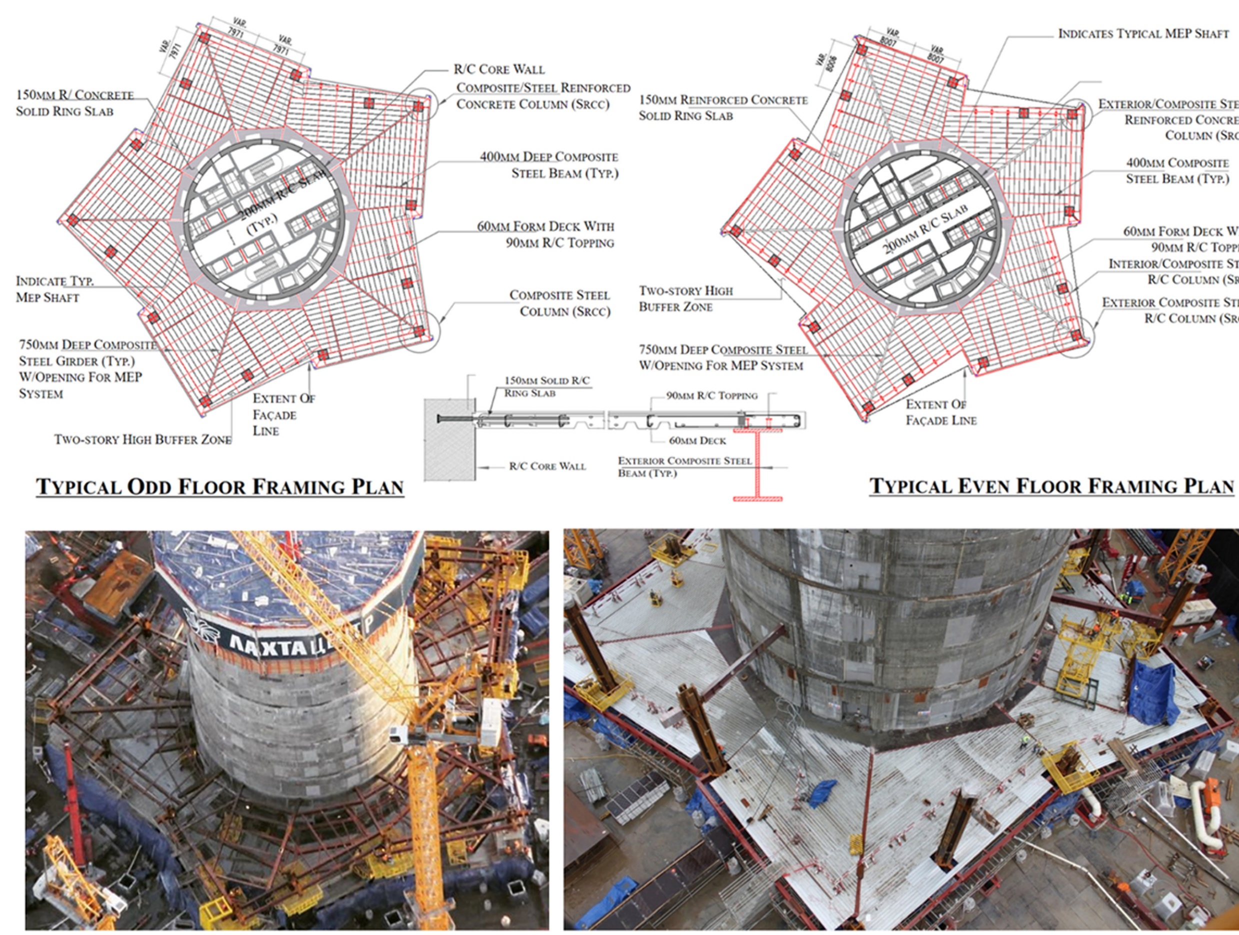


Figure 4. Typical floor framing at odd and even plans and Typical floor construction photos.

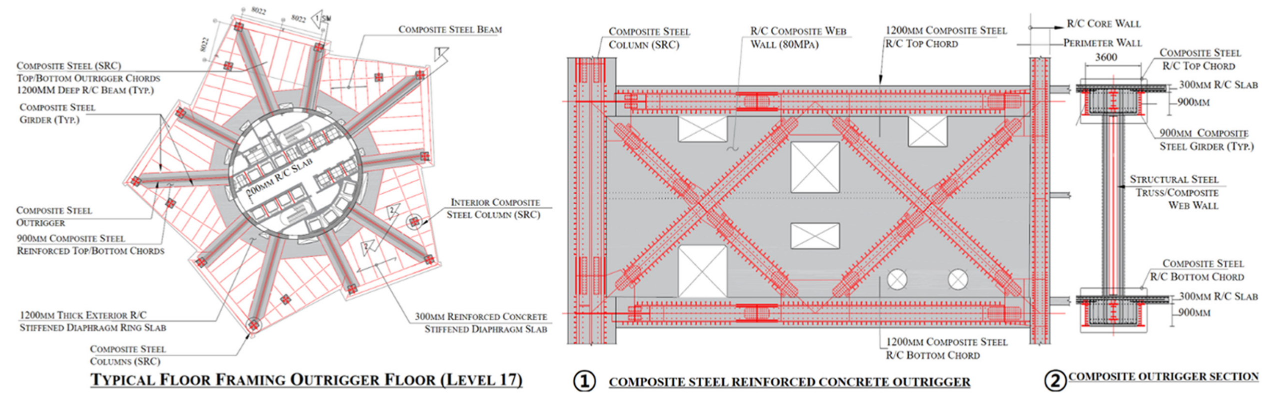


Figure 5. Typical outrigger floor framing plan and composite outrigger section and detail.

("leaves/petals) that rotate, taper, and bow relative to the geometric center of the tower thus resulting in complex geometry and significant changes to the floor plate shape and floor area at every floor. To simplify the framing concept, the framing arrangement for a typical odd and even floor is shown in Figure 4. Composite steel floor framing was selected as the primary structural material not only for its flexibility and speed in construction but also to account for 1) significant changes in building geometry at every floor 2) long spans (up to 17 m) between the core and exterior columns, and 3) long spans between exterior columns.

The typical floor framing shown in Figure 4 comprises of 60 mm deep deck with 90 mm reinforced concrete topping, spanning between the 400 mm composite steel beams, that frame into 750 mm deep composite steel girders. Composite steel girders are designed with 400 mm diameter openings to allow for MEPF (Mechanical, Electrical, Plumbing & Fire Protection) system integration. Note that variation in the floor framing layout at even and odd floors are due to the presence of two-story double-wall system buffer zones, that are used to optimize the tower energy requirements.

Because the SRCC geometry changes for every floor, biaxial horizontal forces are generated at each of these floors in proportion to the additional column loads received from every floor. These forces are transferred to the core wall through 1) the composite girder, which is strategeically located to brace the exterior inclined columns to allow for direct force transfer to the core wall, 2) 150 mm composite deck/reinforced concrete slab, and 3) the solid concrete ring slab at the perimeter of the core, as seen in Figure 4. Extensive studies and detailed 3-dimensional finite element analysis was developed to verify the magnitude

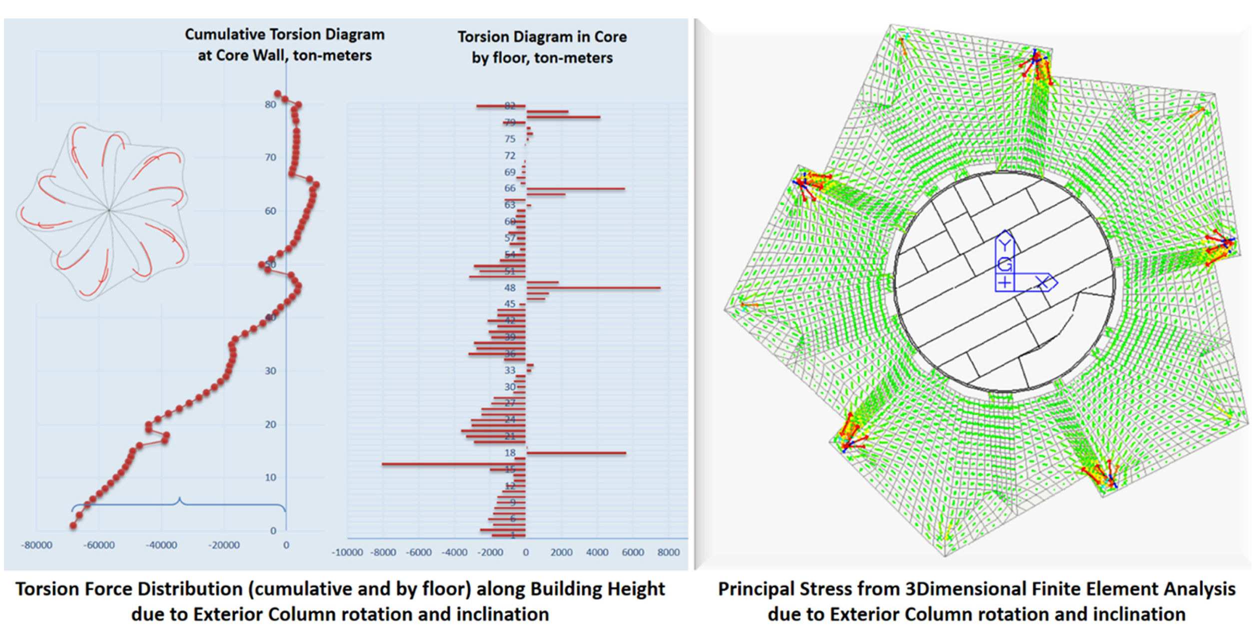


Figure 6. Summary of cumulative and floor-by-floor Torsion Force Distribution and principal stresses at typical floor due to exterior column rotation and inclination.

of biaxial forces generated at every floor (typical 100-200 kN at the location of each column) and the mechanism of dissipating them into the slab and the core wall, as shown in Figure 6. The biaxial horizontal forces from the composite columns are transferred to the structural steel girder connections and reinforcing bar at the SRCC, which in turn transfers to the slabs through shear studs.

3.2. Typical Floor Framing at Mechanical and Outrigger Levels

The composite floor framing system at typical mechanical/outrigger levels is more complex and consists of composite steel framing, spanning between the exterior columns and the core wall as shown in Figure 5. The floor framing system is arranged to allow for ten (10) composite outriggers to pass between the composite structural steel girders, thus allowing the steel construction to proceed normally without delay or hindrance to the composite outrigger construction. Refer to Figure 10 for a typical outrigger section and construction photos. The biaxial horizontal forces from the exterior columns and the horizontal top and bottom outrigger forces are dissipated to the reinforced concrete core wall through 300 mm reinforced concrete slab, 1200 mm deep com-posite top and bottom chords of the outrigger and 1200 mm thick reinforced concrete stiffened diaphragm ring plate at the core wall.

Detailed 3-dimensional finite element analysis accounting for all design forces and their combination was developed to confirm 1) the mechanism of dissipating the horizontal design force into the core wall 2) the magnitude of cracking in the slab and its potential impact on the overall stiffness of the slab diaphragm, and 3) the reinforcement required to anchor the solid stiffened ring plate into the core wall. Extensive sensitivity studies were also performed to confirm the variability of the bending stiffness of the composite outrigger and its overall impact on the lateral stiffness of the tower, which has a direct impact on drift and dynamic properties.

The separation of the outrigger construction from the typical floor framing has significant benefits on construction planning, logistics, and most importantly eliminating the outrigger construction from the critical path by allowing the structural steel works normally, thus resulting in time and cost benefits.

3.3. ReInforced Concrete Core Wall

The centrally located circular reinforced concrete core wall section is proportioned to provide both high bending stiffness and overall torsional rigidity to the tower as shown in Figure 7. The inner diameter of the core wall is approximately 24.5 m in diameter from the raft foundation to level 56, 19.8 m in diameter from level 57 to 80, and 15.8 m in diameter from level 81-86. The core wall thickness varies from 2000/1100/800/600/400 mm from foundation to the top corresponding to levels L6/L58/L64/83/86/90 as depicted in Figure 7.

Note that Figure 7 depicts the changes in the diameter of the center core wall at levels 58, 83 and 87, thus leading to 1) special detailing for the heavily loaded core wall, 2) innovative approach to avoid any impact on the vertical transportation system, building services within the core wall, and the overall efficiency of the building

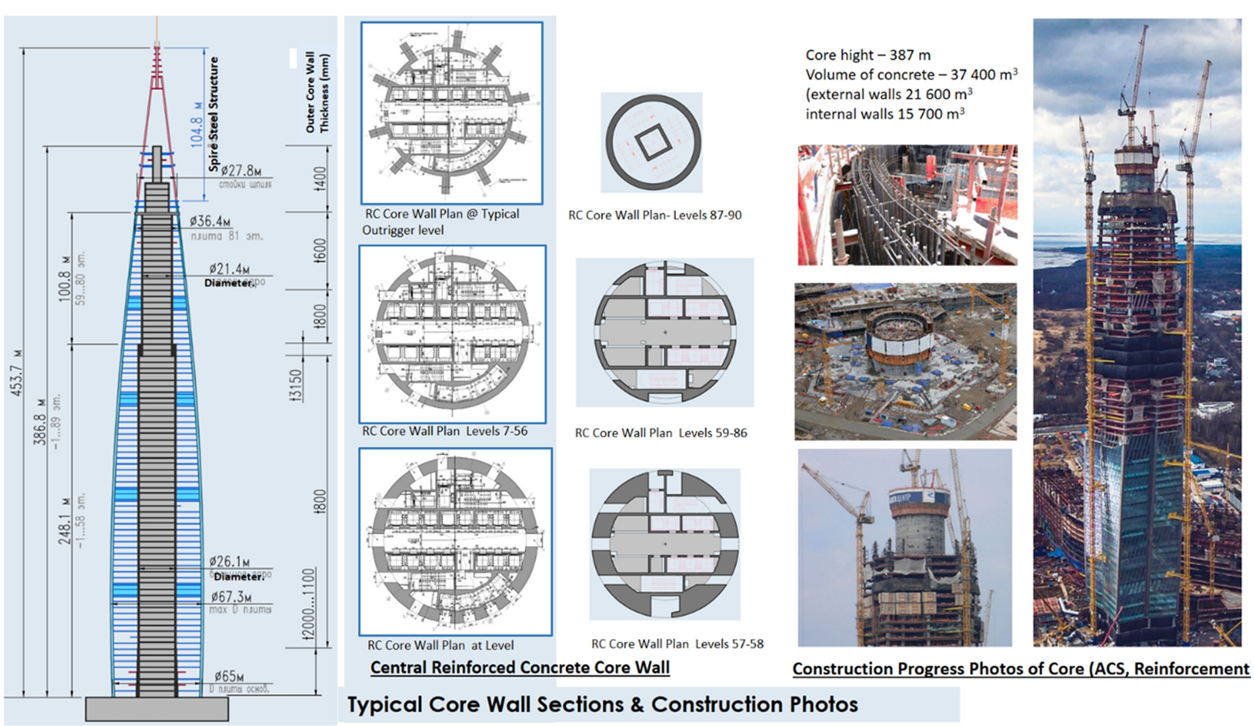


Figure 7. Circular reinforced concrete core wall geometry, sections, and construction photos.

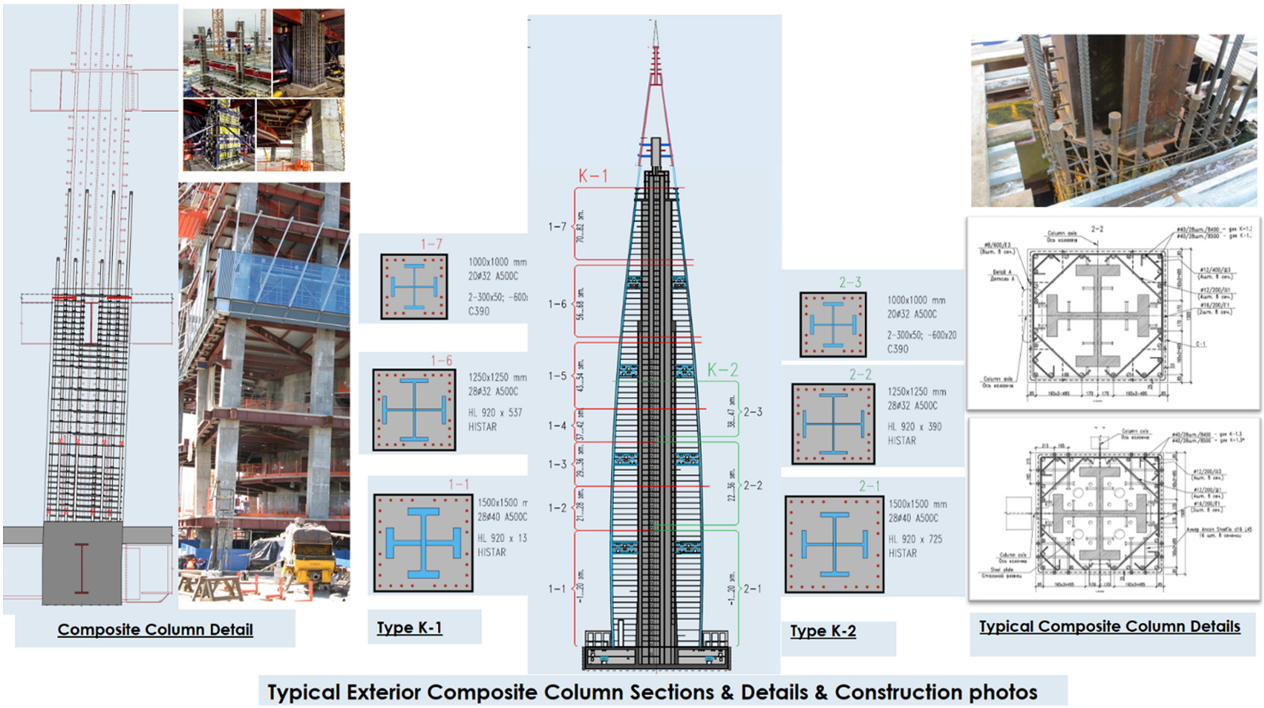


Figure 8. Typical exterior composite columns, details, and construction photos.

and 3) minimized impact on construction speed due to change in the core wall geometry.

3.4. Composite Columns/ Steel Reinforced Concrete Columns (SRCC)

Composite Columns (SRCC) are used to 1) improve

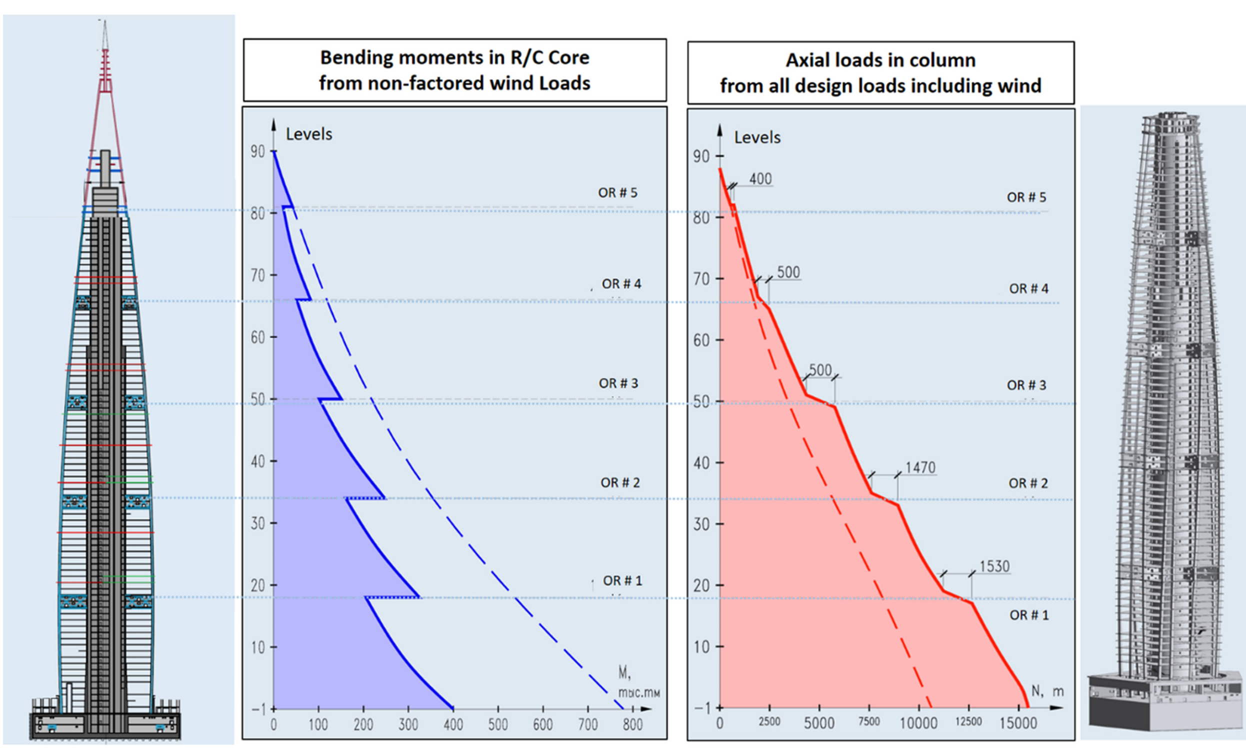


Figure 9. Bending moment in R/C core wall due to service wind loads and axial load at exterior composite columns for all design loads, including wind loads.

the constructability aspect of the columns to achieve a typical 3-4 working day floor construction cycle 2) integrate with and follow the architectural form, and 3) simplify the composite steel floor framing connection to the SRCC. The SRCC consists of centrally located HL 920-Histar high strength steel that is encased in high-performance concrete (B80). These are designed as concrete columns with the highest steel allowed by the Russian Standards. Since the Russian Standards do not have provision for composite column design, the building authorities and experts required special technical documentation and tests to allow their use in building construction. Figure 8 depicts a summary of the sectional details of the composite columns along with construction photos.

Following the architectural form, the SRCC are segmented into two-story modules, rotated every two floors at odd floors and detailed to transfer both the composite steel girder gravity loads and the biaxial lateral loads generated due to change in column geometry. Figure 6 provides a summary of the cumulative and floor-by-floor torsional loads generated from column rotation and inclination. These lateral forces are transferred from the steel-to-steel connection to the floor slabs through a shear stud and finally to the reinforced concrete core through a 150 mm solid reinforced concrete ring slab surrounding the core wall.

The SRCC sections are also optimized to minimize the differential shortening between the center reinforced concrete core wall and the SRCC in order to 1) reduce its impact on the outrigger design maintain floor levelness, and 3) minimize the impact of non-structural components. The actual building survey has shown that the differential shortening between the center core wall and SRCC is similar to the ones predicted from the 3-dimensional

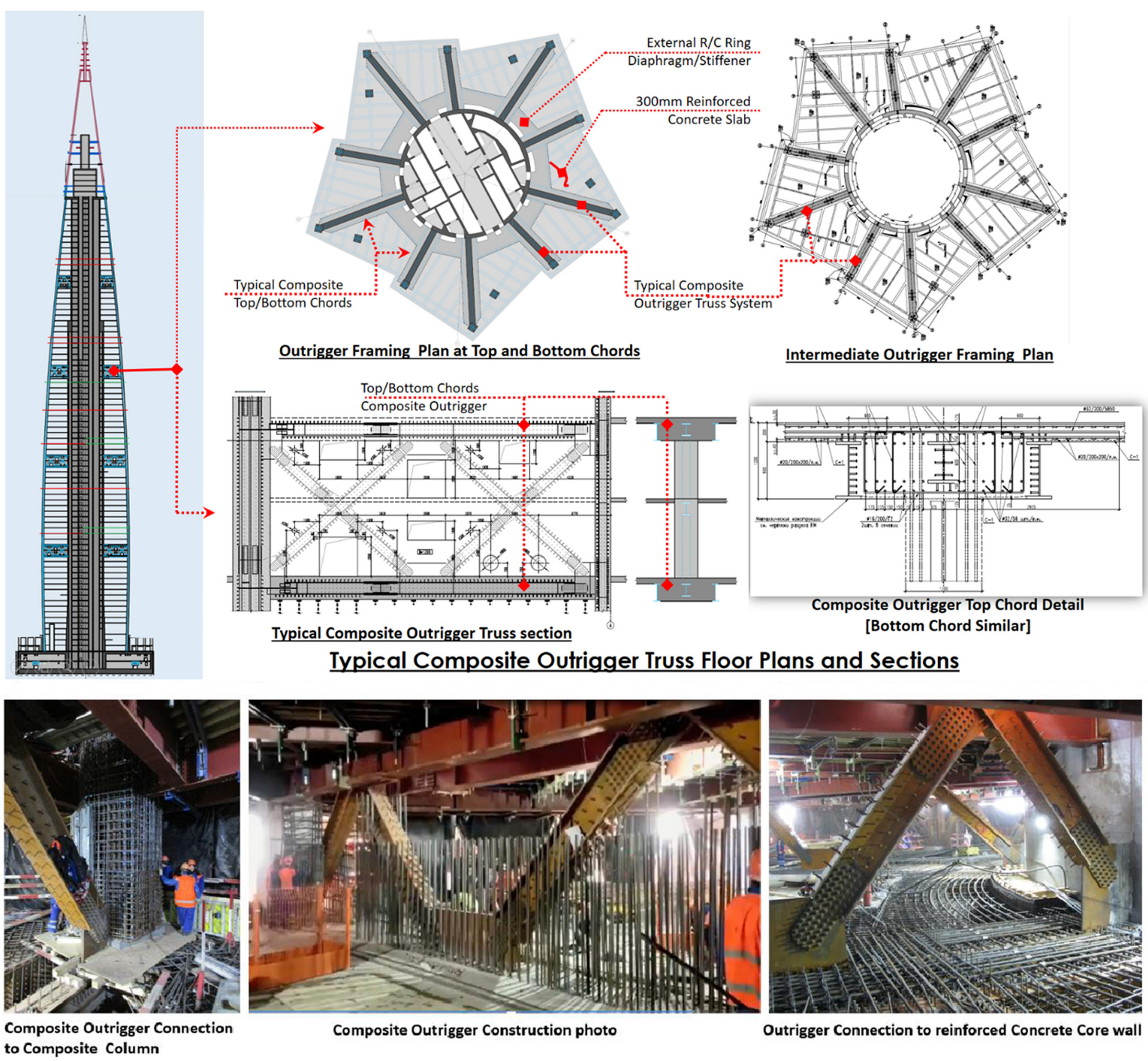


Figure 10. Typical composite outrigger plans, Sections, details, and construction photos.

finite element analysis. This allowed the possibility of connecting the outrigger to the exterior SRCC without time delay.

3.5 Composite Outrigger

As described above, the Mega-Frame structure of the tower, which consists of the center reinforced concrete core wall system, is connected to the exterior composite columns with 4-equally spaced two-story composite outriggers at the mechanical levels (levels 17-18, 33-34, 49-50, 65-66) and secondary outrigger, at level 80, as shown in Figures 9 and 10. The core wall is rigidly connected to the composite steel outrigger system by an externally stiffened plate diaphragm at the top and bottom chords of the outriggers.

This type of outrigger system has reduced the overall bending moment in the core wall, due to wind and lateral loads, by more than 50%, but increased the exterior columns wind and lateral loads by approximately 20-30%. The design of the composite columns was mostly dominated by gravity load requirements and not significantly affected by wind or lateral loads, which demonstrate the efficacy of the Mega-Frame system.

The composite outriggers are provided to not only maximize the Mega-Frame stiffness but to also provide redundancy and alternate load path to the exterior columns. The composite outrigger system is comprised of 1) structural steel diagonals that carry 100% of the shear, in addition to the concrete sectional capacity 2) Composite top and bottom chords that are cast monolithically with the 300mmm thick reinforced concrete slab to maximize the bending stiffness of the outriggers, and 3) externally stiffened reinforced concrete ring plate diaphragm to transfer the outrigger horizontal forces to the center core wall. This composite outrigger system is a new, unique, and innovative concept that is not only structurally efficient but has significantly improved the overall constructability aspects of the tower by avoiding the detailing complexities associated with traditional outrigger construction methods. Moreover, this outrigger system also helped to skip the outrigger construction from the critical path, which resulted in significant savings in time.

A 3-dimensional finite element analysis is developed to verify the outrigger horizontal design forces that dissipate into the center core wall through the externally stiffened reinforced concrete ring plate.

3.6. STructural Steel Spire

The lateral load resting system of the spire is founded at level 83 and is comprised of a 5-sided braced frame structure that tapers from +344.4 m (level 83) to +428 m. To simplify the spire construction, the spire structure is divided into: 1) an interior braced frame consisting of 5-steel pipe columns that linearly taper to a single structural steel ring at +428 m; the structural steel ring also provides support to the single pipe pinnacle and 2) the primary meshed façade framing system which consists of built-up rectangular steel tube sections that follow the architectural

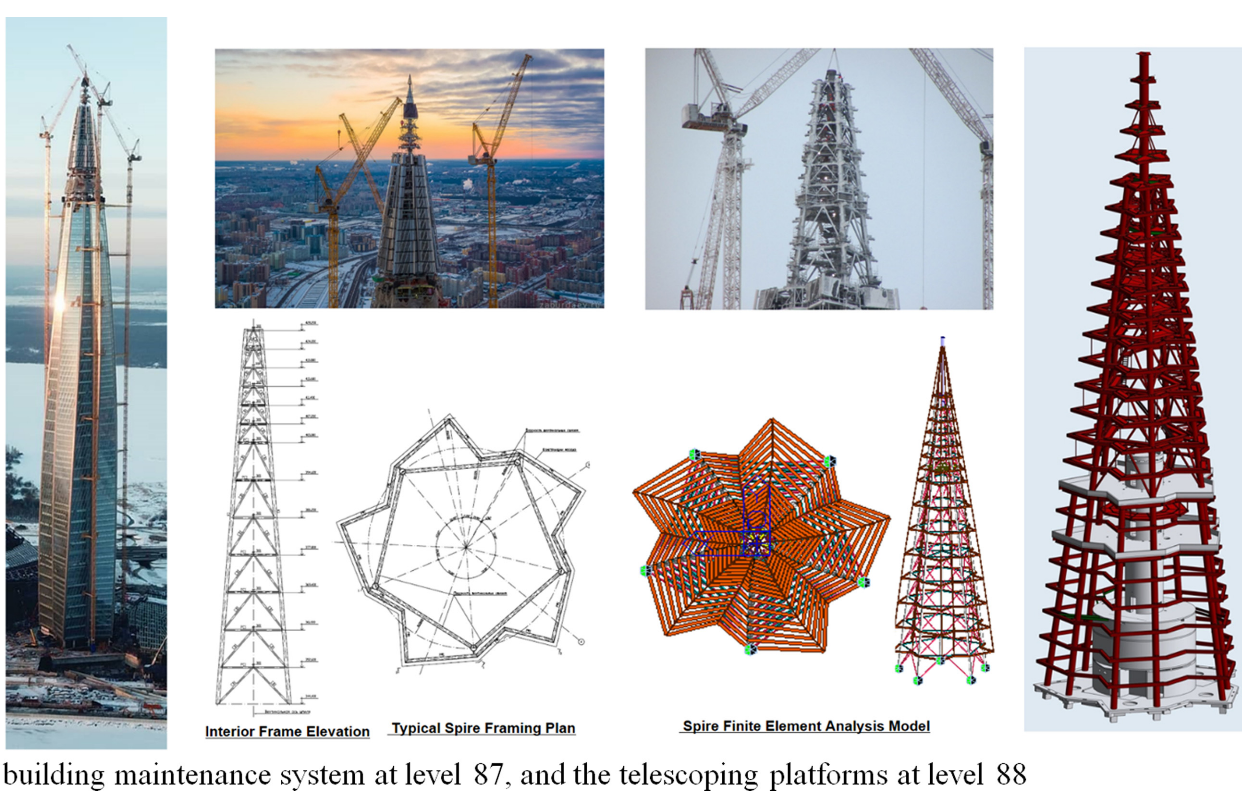


Figure 11. Structural steel spire structural concept and construction photos.

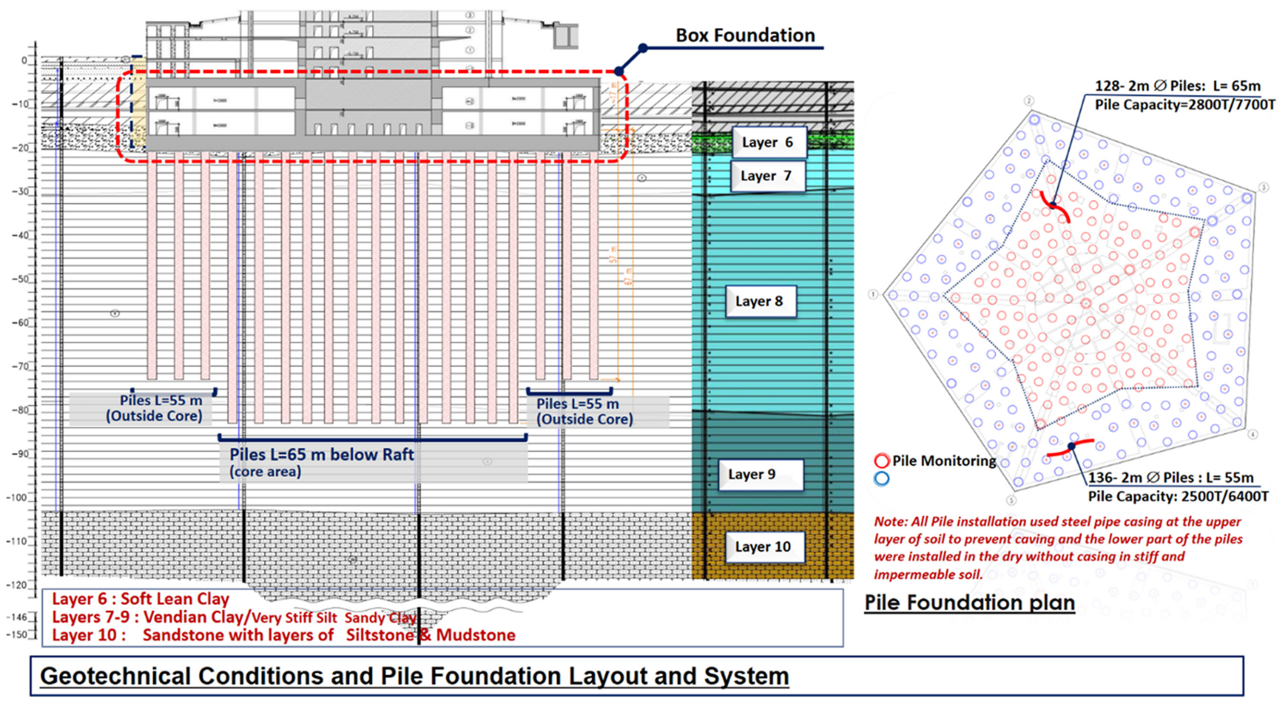
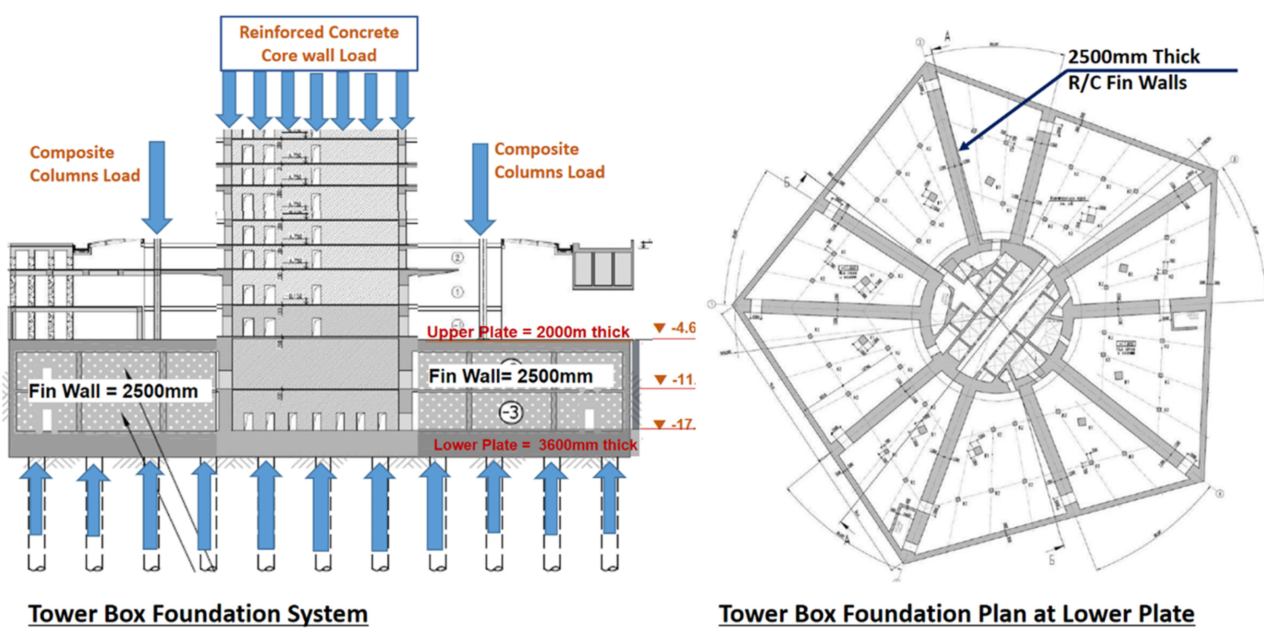


Figure 12. Geotechnical conditions, pile layout, and pile foundation system.



Box Foundation Material Summary: *a) 3.6m bottom raft (20300m*^{3}*-B60-water tight W8, frost resistance concrete), b) 2m top slab (10500m*^{3}*-B80-water tight W8, frost resistance concrete F150), c) 2500mm thick x16.6m Deep Fin Walls (13400 m*^{3}*-B80 concrete), and d) 400mm thick Middle Slab (2000m3-B60 Concrete)*

Figure 13. Tower Box Foundation System section and plan, and total material summary.

geometry of the tower. The spire façade framing system is cantilevered from the interior braced frame columns as shown in Figure 11. In addition to providing support for the spire façade framing, the interior braced frame also provides support to the observation floor framing systems at levels 83, 86, building maintenance system at level 87, and the telescoping platforms at level 88.

Above level 88, the spire façade is an open meshed façade that allows for free wind flow. This reduces the overall wind forces thus improving the dynamic response of the tower to wind excitation. Wind tunnel investigation of meshed (opened) and fully closed spire faade have been studied and an open meshed façade was selected as the primary spire façade system.

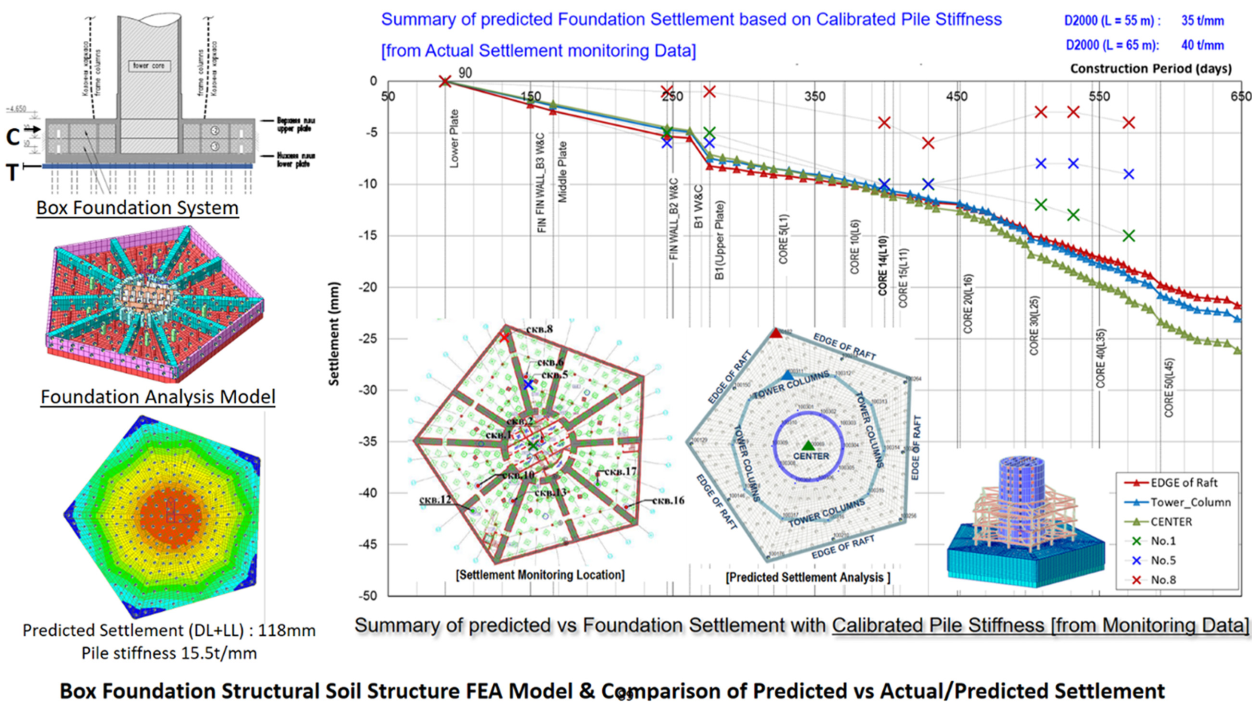


Figure 14. Summary of Box Foundation soil-structure FEA Model and comparison of predicted vs. actual settlement [from calibrated pile foundation stiffness from monitoring data].

The open-meshed spire façade presented itself with many challenges including but not limited to managing ice accumulation and removal and arresting the potential for ice falling. Several measures were taken to overcome these challenges, including capturing the falling ice by providing ice catching frames. Several testing programs, desktop studies, mockups, and in-situ testing were completed to confirm the expected performance of the system.

The spire was erected traditionally using pre-assembled block methods using the free-standing Tower cranes. The pinnacle's structural steel frame, façade, lightning protection, and other monitoring devices were pre-assembled and erected to the final position using the free-standing tower cranes under severe weather conditions. Figure 11 depicts the structural design concept of the spire and photos of the spire during construction under severe winter conditions. The spire was erected to 462 m safely in December 2018.

4. Tower Foundation System:

The Tower is founded on 264-2000 mm diameter reinforced concrete bored piles that extend to 65 m under the core footprint and 55 m elsewhere, as shown in Figure 12. These piles are used to reinforce the soil rigid mass block under the box foundation and re-distribute the 670000 megatonne load evenly to the effective pile-reinforced-soil-mass under the box foundation area. The 28 m diameter core wall receives approximately 70% of the vertical gravity load of the tower over a relatively small area. This is not sufficient to effectively transfer loads of the heavily loaded core wall and to manage the differential settlement between the core wall and exterior columns of the tower. So, a stiff reinforced concrete box foundation, consisting of the entire subgrade structure, is utilized to transfer the tower in direct bearing to evenly distribute the tower loads to the pile-reinforced-soil-mass and to control the total and differential settlement between the core and the columns. The average bearing pressure at the bottom of the box foundation is expected to be at approximately 900 kPa. This pressure is significantly reduced to approximately 200 kPa at 60 m below the base of the box foundation.

The reinforced concrete box foundation system consists of 3.6 m bottom raft plate (bottom flange) and 2 m top plate (top flange), that are connected by ten (10) 16.6 m deep high performance reinforced concrete (B80) radial fin-web walls. The 16.6 m deep web walls, spanning from the center core wall to the edge of the bottom raft plate, create a highly stiff and efficient two-way reinforced concrete box foundation system that allows for uniform distribution of the tower load to the piles-soil block mass. Figure 12 depicts the Tower Box Foundation System, load transfer concept and a summary of the structural material used for the box foundation.

A 3-dimensional Finite Element Analysis Model (FEAM) of the soil-structure interaction and a detailed Construction Sequence Analysis model was carried out to predict the foundation settlement and the actual load distribution to the piles. The soil-structure interaction

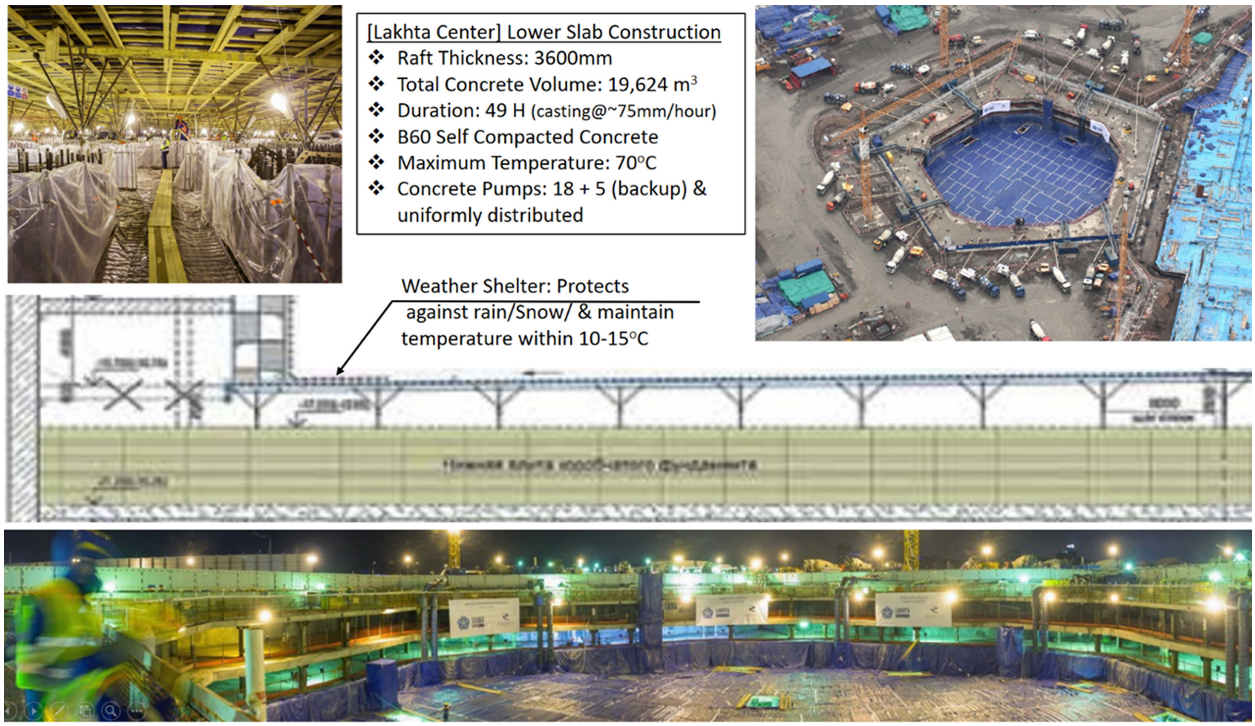


Figure 15. Overall arrangement of weather protection shelter and concrete pump arrangement.

analysis models included geotechnical parameters recommended by the geotechnical engineer, static pile stiffness of 15500 T/m, and the stiffening effects of the superstructure and box foundation. The predicted foundation settlement from these parameters is expected to be in the range of 120 mm at the center of the raft and 100 mm at the edge of the raft.

An extensive Geo-monitoring program consisting of more than 2600 devices was provided to monitor the overall behavior of the pile-reinforced-soil-mass block and it included 1) 336 strain gauges along the length of the pile 2) 40 soil pore measure transducers (in measuring wells) 3) 95 vertical mass displacement transducers (in measuring wells) 4) 10 pressure cell transducers at the bottom of the lower slab plate (raft) foundation 5) 2136 Box Foundation force transducers, and 6) an independent optical foundation settlement survey program that monitors the total foundation settlement at both the top of the lower slab plate and the box foundation.

The actually observed foundation settlement was significantly lower than those predicted from the soil-structure interaction model shown in Figure 14. Therefore, the actual pile stiffness of the 55 m and 65 m long piles was calibrated to reflect the actually measured settlement obtained from both the optical survey and geotechnical monitoring programs and was estimated to be approximately 35 t/mm and 40 t/mm respectively.

Construction sequence FEAM for the tower with calibrated pile spring stiffness was developed to verify its impact on the overall tower superstructure and foundation systems behavior. Figure 14 depicts the foundation settlement from the calibrated soil-structure FEAM construction sequence analysis model. Note that the overall settlement is uniform across the box foundation and significantly lower than those predicted in the base design. This successfully demonstrates the expected behavior from the box foundation system.

4.1. Construction planning for the Box Foundation

The Box Foundation System was placed in three stages. First, the 3.6 m bottom raft plate (20300 m^3) was cast in a single pour, followed by the fin walls and middle 400 m thick slab, and finally the 2 m thick top slab plate. Detailed construction sequence analysis, with heat of hydration analysis (accounting for concrete temperature rise, curing, and cooling) and restraining effects of boundary elements (D-wall, core wall, and fin walls), was performed to verify the behavior of the Box Foundation System and to confirm that the construction method and sequence would not result in thermal, shrinkage, or restraining cracks. Detailed planning and extensive testing programs were put in place for the construction of Box Foundation, especially for the 3.6 m bottom slab/raft. High performance/self-compacting concrete was used for the bottom slab and was cast continuously within 50 hours in cold weather conditions.

During construction of the bottom slab plate, the entire raft area was sealed, fully protected against the weather (rain/snow) and cast under controlled temperature conditions. Figure 15 depicts the overall arrangement of the

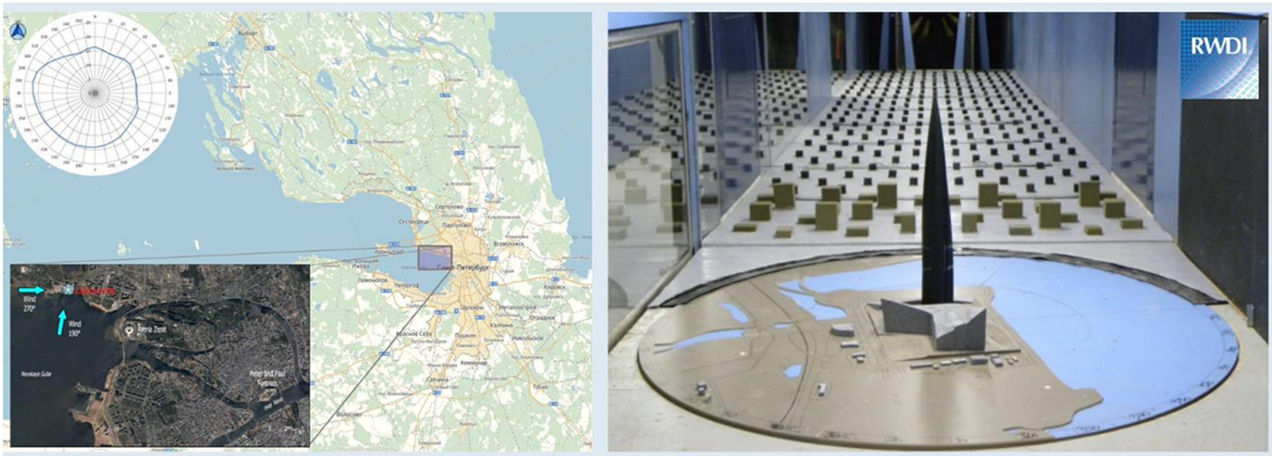


Figure 16. Lakha Center Tower location on the Finland Sea, Wind directionality rose, and wind tunnel testing setup at RWDI.

weather protection shelter and the concrete pump arrangement using stationary pumping to ensure the continuous pouring of concrete. Managing the temperature of concrete from the batch plant to site required that 1) the initial concrete temperature at deposit was kept within the range of 5-15°C, 2) the maximum temperature, due to heat of hydration, was kept within 70°C at all locations, and 3) the maximum differential temperature between any two points was kept within 20°C.

In addition to logistical issues of timely delivering concrete to the site, the raft was cast in a single pour and thus required careful planning to control concrete setting and hardening time to prevent any potential of forming cold joints. This required managing the initial concrete set and strength development to 1) 0 Mpa within 24 hours 2) 15 Mpa in 3days 3) at least 7 Mpa at 7 days, and 65 Mpa at 28 days.

Moreover, managing the concrete cooling and curing was equally important in order to prevent any potential for thermal cracking. This was controlled by limiting 1) the concrete cooling rate to within 2-3°C per day 2) the maximum differential temperature between any two points to 20°C and 3) the temperature difference between the top concrete surface and the outside air temperature within 20°C. The top surface of the raft was troweled and sprayed with aqua-dispersive film and finally covered with insulting and thermal blankets to control the surface temperature. Real-time temperatures were measured

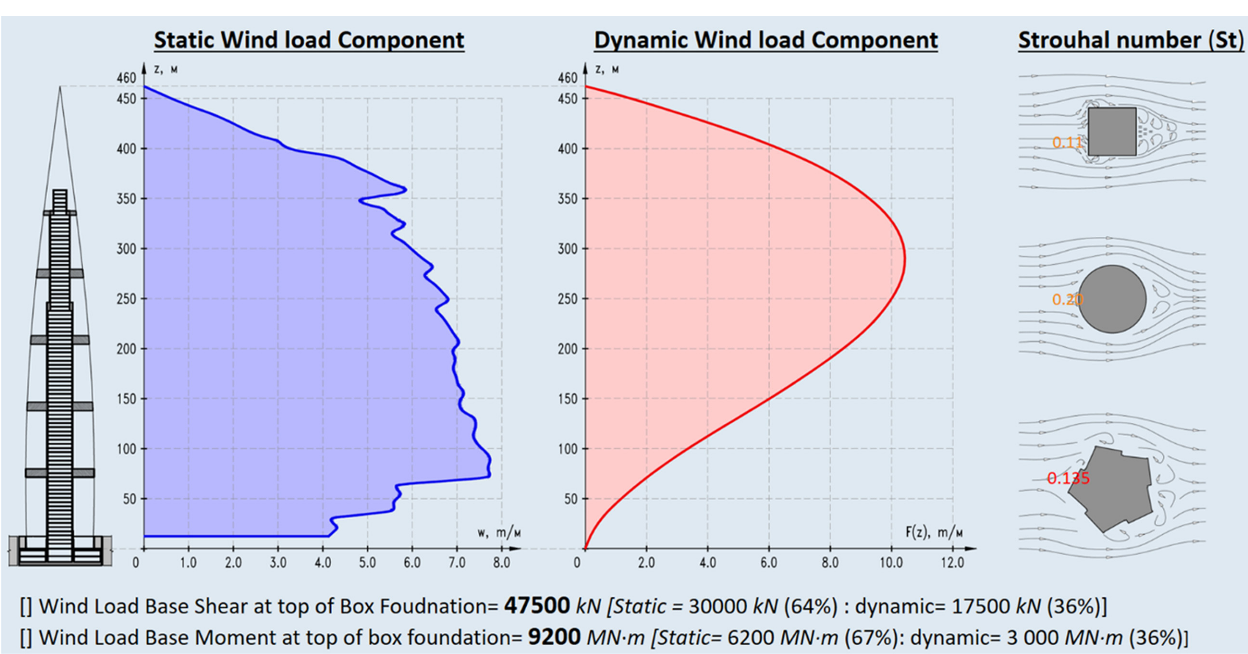


Figure 17. Static and dynamic wind load distribution along the building height, base design forces, and predicted Strouhl number for typical floor geometry.

across the lower raft plate and at three different levels of the raft thickness as follows

- level 1: at 100 mm from the bottom of the lower slab
- level 2: at the center of the raft,
- level 3: at 100 mm from the top surface

5. Wind Engineering Management

Wind Engineering is one of the primary concerns in the planning and design of tall buildings as it has a major impact on the overall tower design efficiency, wind loads, and human comfort and perception to motion due to dynamic wind excitation. The architectural design concept of the tower lent itself to mitigate the dynamic wind effects by the virtue of the architectural design form, which is twisted, tapered and bowed. These systematic and continuous geometrical changes resulted in dimensional variation to the floor plan and edge conditions on every floor. In addition, the open/vented/meshed/ façade at the spire allowed for free wind flow. These wind engineering treatments resulted in 1) disorganizing the vortex shedding formation along the building height 2) reducing overall dynamic wind forces, and 3) limiting the acceleration of the tower within the international standards for office buildings thus eliminating the need for auxiliary damping.

Several wind tunnel testing regimes and programs at RWDI, along with wind engineering desktop studies were performed to verify the expected favorable wind engineering treatment of the tower. This includes: 1) meteorological and climate analysis that involved characterizing the statistics of extreme winds based on historical records collected at local weather stations and predicting the wind profile 2) force balance studies on rigid models to determine the static and dynamic wind effects, which included closed and open spire schemes [base design forces, acceleration, velocities, wind aerodynamic coefficients, etc.] 3) pedestrian wind studies to determine the maximum velocities at base of the tower and its impact on pedestrian wind safety and comfort and the development of mitigation plans as deemed required 4) façade wind pressure studies to determine maximum design pressure envelope 5) stack effects within the building envelope 6) ice accumulation along the building height, and 7) detailed ice accumulation and mitigation plans for the open/meshed façade spire scheme.

The maximum wind design forces obtained from RWDI wind tunnel testing and studies were peer-reviewed and confirmed by the wind engineering experts in Russia and the design forces were modified (increased) to reflect the Russian standards and requirements. Figure 17 depicts the static and dynamic wind load distribution along the building height. Note that due to the tower geometry, the maximum wind distribution along the height is not at the top of the tower thus reducing the overall base design forces.

The 3-dimensional Finite Element Analysis Model (FEAM) yielded pile foundation flexibilities (both static and dynamic), flexibility of the outrigger floor slabs, and of the box foundation system. This FEAM model indicated that the total displacement of the tower under maximum design wind load (static and dynamic) was well within the H/500 drift limit and the three fundamental modes of vibrations (translational and rotational) were 8.65 sec, 8.46 sec, and 2.5 sec respectively.

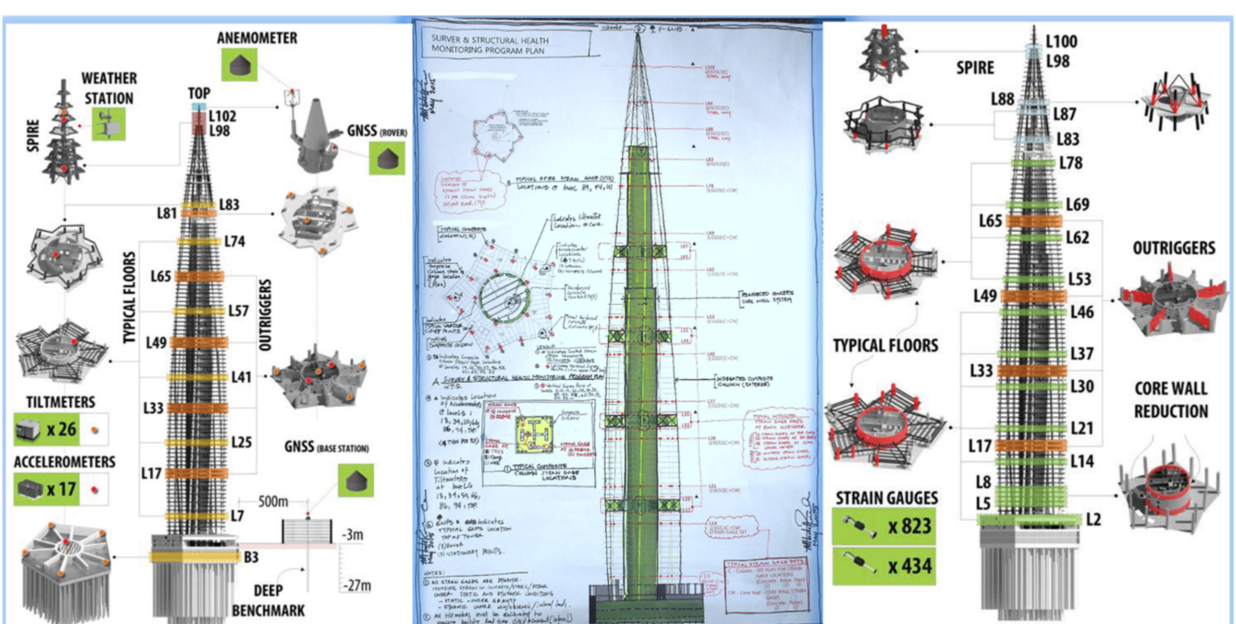


Figure 18. Survey and Structural Health Monitoring Program Concept.

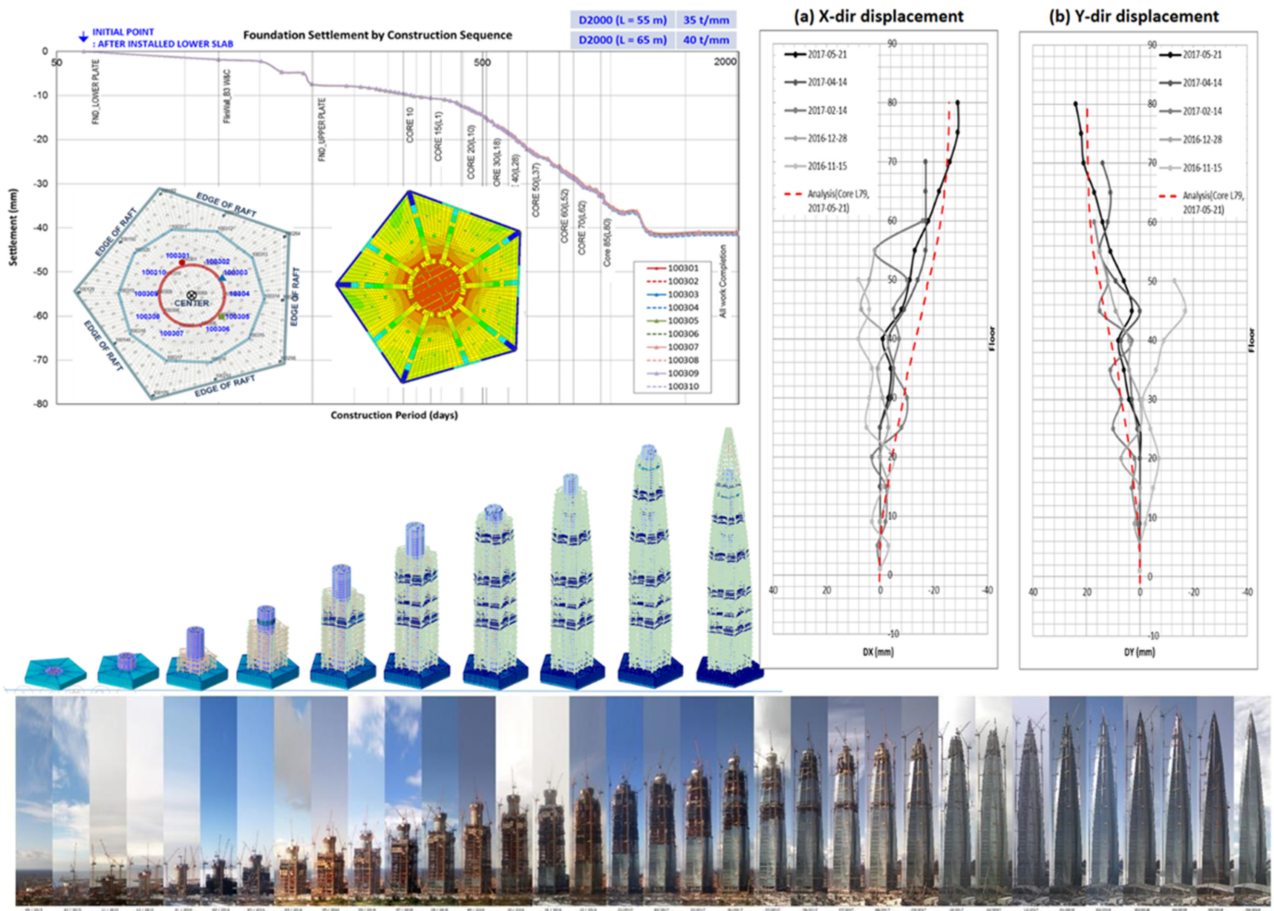


Figure 19. Construction Sequence Analysis and predicted foundation settlement and building lateral movement under gravity loads, including long term effects.

6. Seismic Engineering Management

Lakhta Center is located in very low Seismic according to the Russian Codes and standards. Since the tower has long period, it is prudent to review effects of far earthquake, with long period contents, impact on the tower. Site specific seismic hazard anlaysis that takes into account the regional tectonic environment, historic seismicity of the region, effect of near and rare and far earthquake was reviewed. However, because of the tower long periods and low energy seismic forces, the lateral load resisting system of the tower is controlled by wind forces.

7. Survey and Structural Health Monitoring program:

An extensive survey and real-time structural health monitoring programs were developed for the Lakhta Center Tower to verify the behavior of the tower structural system during construction and under permanent building conditions. This included verification of the assumptions and design parameters made during the design of the tower superstructure and its foundation systems. The monitoring program concepts shown in Figure 18 were developed in collaboration with the key stakeholders including Samsung C&T, Gorproekt, Infrosproekt, NIIOSP, and SODIS LAB. The survey and the Real-Time Structural Health Monitoring Programs (RT-SHMP) included the following:

- An extensive survey monitoring program to measure the foundation settlement, column shortening, and lateral building movement during construction.
- Installation of strain gauges to measure the total strain at the main structural and foundation elements, including bored piles, box foundation elements (lower slab, upper slab, fin walls, etc.), reinforced concrete core wall, exterior and interior steel-reinforced concrete columns (SRCC), composite outrigger components (top/bottom chords, outrigger steel truss, reinforced concrete wall panels, etc.), and spire vertical steel frame.
- Installation of temporary/permanent real-time monitoring programs to measure the lateral displacement and accele-ration of the building during construction and to verify predicted building displacement and dynamic characteristics (frequencies, damping, etc.). This

Figure 20. Construction progress phots: Lakhta Center is a beacon looking over the Gulf of Finland and has the appearance of a ship sailing into the seas.

system included bi-directional anemometers, tiltmeters, Global Navigation Satellite System (GNSS), and weather stations to measure wind speed, wind direction, humidity, and temperature. Figure 18 depicts the location of the RT-SHMP devices (IoT) along the building height. The installation of these devices with a central command center resulted in 1) development of a full-scale real-time aeroelastic model of the tower, allowing for real-time assessment of dynamic behavior of the building under wind, seismic excitation (if any), and unexpected external lateral loads 2) direct assessment of the strain/stress in the structural members of the spire including fatigue assessment 3) wind load distribution at the top of the tower, and 4) real-time information about the building movement (from GNSS) and dynamic characteristics (from biaxial accelerometers) to allow the building facility management team to make management decisions about any issues that may arise during the lifetime of the tower. In addition, the tri-axial accelerometers at the base of the tower allow capturing site-specific time history records of any earthquake that may occur in the region (near or far) and its potential impact on the tower.

During construction, a predictive 3-dimensional finite element analysis model was developed to simulate the actual construction sequence of the tower that included foundation flexibility (static and dynamic stiffness), actual material properties, and construction sequence of the tower according to the physical work performed by all the trades at the project site.

Correlation between the predicted model and the actual survey has shown that the foundation settlement trends where much smaller than predicted thus requiring calibration of the pile-reinforced-soil-mass block modulus and a re-analysis of the construction sequence to confirm the actual tower behavior. A comparison between the calibrated foundation stiffness has shown that the predicted settlement is within the actually measured settlement. Figure 19 depicts the predicted foundation settlement and the lateral displacement of the tower under own gravity loads in x and y directions. Construction sequence analysis model also indicates that the differential shortening between the core wall and the composite columns were within the predicted movement and the building lateral displacement (including creep and long term effects) were also in line with those predicted.

The authors will continue to process significant information and data collected relative to the overall building behavior and will share this in due course. Significant advances in RT-SHMP is continuously being developed using technologies such as IoT, AI, and new devices. The collection of these big data along with the development of predictive models of the actual performance of the tower in real-time is now feasible and can be made readily available to building owners..

8. Conclusion

The Lakhta center with its impressive architecture is designed as a simple, organic, asymmetrical, singular iconic landmark, that continuously and elegantly spirals counter-clockwise to 462 m to become the tallest building in Europe and the second tallest twisted supertall tower in the world. The tower is composed of five (5) individual “Leaves/Petals that twist and taper as they rise, thus resulting in a tower that rotates 90 degrees from the base to the tip of the pinnacle.

The complex geometry of the tower resulted in significant changes to the floor plate shape and floor area, which in turn resulted in significant structural challenges that required the development of innovative structural engineering solutions that: 1) integrates the structural system with the architecture fabric 2) is continuous throughout the building height 3) accommodates the secondary design forces due to building twist and taper without additional cost or time impact 4) maximizes the exterior views by minimalizing the number of exterior columns and avoiding exterior diagonals to maintain the aesthetics and the vision of the architectural design 5) utilizes all vertical elements to resist lateral loads (wind, seismic, lateral stability) and maximizes overall building bendingstiffness 6) is redundant 7) minimizes the effects of differential column shortening, and 8) considers constructability and avoids complex detailing.

Extensive wind tunnel studies were performed by RWDI and the variation in building shape (twist, bending, variation at edge conditions, vented spire, etc.) resulted in a favorable wind response that is within internationally accepted drift, velocity, acceleration, and human perception limits, thus eliminating the need for a supplementary damper.

The development of the structural and foundation systems of the Lakhta Center resulted in the following innovations to optimize the design and construction planning of the tower:

1. The lateral load resisting system of the tower consists of a centrally located reinforced concrete core wall, 26 m in diameter, that is linked with 10 exterior composite steel-reinforced concrete columns through 10 two-story, equally spaced, composite outriggers at 4 levels and one-story outrigger at level 83. Non-traditional composite outrigger structural system has been introduced to simplify the design and construction of the tower without the complex continuity through the core.
2. Development of a new composite outrigger system that allows the large outrigger chord forces to transfer to the core wall through an external stiffened diaphragm ring slab, which avoids the complexity of the traditional outrigger system that disrupts the design and construction planning of the core wall. This results in significant savings in time and cost.
3. Utilization of composite steel floor framing for long spans (up to 17 m) to allow for direct biaxial horizontal load transfers, caused by the change in geometry of the exterior composite columns, to central core wall through floor slab and reinforced concrete solid ring slab around the core without the complex detailing.
4. Utilization of multi-story stiff box foundation to evenly distribute tower loads to pile-reinforced soil mass. Soil-structure interaction analysis and actual foundation settle-ment have demonstrated the efficacy of the box foun-dation system in evenly transferring the tower loads to the pile foundation and in minimizing the differential settle-ment between the core wall and the exterior columns.
5. Foundation survey monitoring programs have demon-strated that the foundation settlement is significantly lower than those predicted during the design stage. This required a re-analysis of soil-structure FEAM to confirm the actual building behavior and verification of the com-pensation program.
6. Construction Sequence Analysis indicated that adjusting the pile spring stiffness to reflect the actual settle-ment has resulted in the close prediction of the observed structural behavior.
7. Development of state-of-the-art real-time structural health monitoring and survey programs to have a better under-standing of the overall building behavior under both gravity and lateral loads.
8. Optimizing the structural design of the tower has significantly reduced the embodied energy in the tower and contributed to energy savings and sustainability. Lakhta Center has already achieved Platinum Leed Certification.

The Lakhta Center Complex construction is now completed, commissioned and has become the tallest building in the Northern Hemisphere, Europe, and Russia. It is strategically located to provide connectivity and access to public infrastructure and to serve as a catalyst to the development of a 21st-century landmark/iconic business center. It is also well suited to complement and expand the existing historic and world heritage CBD to a new business district that will attract major corporations and bring significant economic benefits to St Petersburg. Lakhta Center will not only serve as a showcase for the utilization of future technological advances in materials, design, and construction but also as a catalyst for the continuous development of future technologies.

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Owner & Developer: Gazprom : A Joint Stock Company Gazpromneft Eastern European Projects

Architects (Concept): RMJM in collaboration with

Gorproekt

General Designer (Design Execution): Samsung C&T Corporation

Architect & Engineers of Record: Gorproekt

Structural Engineer: Samsung C&T Corporation in collaboration with Gorproekt & Inforceproject

Construction Manager: Samsung C&T Corporation

General Contractor: Renaissance Construction Company, Russia

Tower Façade: Gardner

Vertical Transporation: Schindler

Wind Tunnel Testing & Spire Icing Studies: RWDI, Guleph, Onatrio, Canada

References

A. Abdelrazaq. Validating the structural behavior and response of burj khalifa: synopsis of the full-scale structural health monitoring programs.International Journal of High-Rise Buildings, 1(1):37-51, 2012.

A. K. Abdelrazaq, W.F. Baker, K.R. Chung, J. Pawlikowski, Insoo Wang, K.S. Yom, Intergraiton of Design and Construction Planning of the Tallest Building in Korea, Tower Palace III, Seoul, Korea. CTBUH 2004 October 10-13 Seoul, Korea.

Ahmad Abdelrazaq, Frances Badelow, SungHo-Kim and Harry G. Poulos, 2011. Foundation Design of the 151 Story Incheon Tower in a Reclamation Area. Geotechnical Engineering Journal of the SEAGS & AGSSEA Vol 42 No.2 June 2011.

Ahmad Abdelrazaq, Moonsook-Jeong, Soogon-Lee, Taeyoung-Kim, Optimizing the Structural and Foundation Systems of the 151 Story Inchon Tower: The Development of New Generation of Tall Building System, 2nd R.N. Raikar International Conference and Banthia- Basheer International Symposium Paper on Advances in Science and Technology of Concrete, 18-19 December 2015, Mumbai, India.

Choi, H., Ho, G., Joseph, L. & Mathias, N. (2012) Outrigger Design for High-Rise Buildings: An output of the CTBUH Outrigger Working Group. Council on Tall Buildings and Urban Habitat: Chicago.

Travush V.I., Shakhvorostov A.I. Concreting of the bottom slab of the box-shaped foundation of the Lakhta Center complex. *Vysotnyye zdaniya*, 2015, No. 1, pp. 92-101 (in Russian).

Travush V.I., Shakhraman'yan A.M., Kolotovichev YU.A., Shakhvorostov A.I., Desyatkin M.V., Shulyat'yev O.A., Shulyat'yev S.O. Lakhta Center automated monitoring of deformations of load-bearing structures and foundations *Academia. Arkhitektura i stroitel'stvo*, 2018, No. 4, pp. 94-108 (in Russian).