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Technical paper

Kula Belgrade – Part 2 - Specifics of construction of Kula structure

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ABSTRACT

Kula Belgrade is the tallest building within the Belgrade Waterfront Project located on the right bank of the Sava River. It is envisaged as the future landmark of Belgrade and the pivotal point of Belgrade Waterfront development. It consists of a 168m high - 42 storey tower, a podium and an eccentric basement. It is one of the rare towers in the world in which the bottom and the top parts are mutually rotated by 90° in plan and where such a transition is achieved through 7 floors - a configuration that imposes significant demands to the structure. While the 1st part of the article addressed specific topics related to design, this 2nd part is about the specific topics related to the construction of Kula Belgrade's structure, including the enabling works, construction, and testing program of piles that comprised various types of tests, including the static compression test by Osterberg cell at two tower piles with a 1200 mm diameter. The article also addresses the construction of foundations, which included the concreting of a raft under the tower in volume of 4750m^3 cast in one turn, the execution of core walls in jump form, the distinctive transfer structure and PT slabs.

1 Introduction

While the first part of the article[1] addressed the specifics of the design process, this second part covers the key items related to Kula construction, including: testing and execution of piles, enabling works, concreting of raft (part below the tower – approximately 4750m^3 was concreted in one turn), works on the superstructure, specifictransfer structure and PT slabs.

The first works conducted on Kula site, prior to any listed above, were the works on unexploded object detection (UXO) by company PMC Millennium in 2016.

When the Piling Contractor (Novkol) entered the site later that year, they formed a working platform for the execution of testing and working piles at the level of partially excavated terrain by the UXO Contractor.The Investor „BW Kula", Ltd. awarded supervision on piling works to MACE. First tests of piles were done in 2016., while the construction of working piles took place in 2017. Darko Božić, BSc, CEng was the pile construction's responsible engineer.

Once the building permit for main works was obtained, early in 2019., the main contractor, Pizzarotti Millen-niumTeam (PZMT) overtook the site from Novkol and proceeded with further excavations to the final levels. Upon the excavations, the piles were cropped to design cut off levels by Novkol and handed over to the Main Contractor. Nenad Marković, MSc, CEng, was the responsible engineer for Kula construction.

Only then could the work on Kula structure have started. Supervision at this stage of the project was conducted by DNEC.

2 Testing and construction of piles and enabling works

Enabling works were conducted under a separate building permit, which included excavations as well as the testing of two tower test piles by the Osterberg cell method. Additionally, a whole series of pile tests were performed, with the aim of testing geotechnical parameters provided by the Geotech report [2], as a basis for design as well as piling construction methodology, so the overall pile testing process covered the period from the early stages of design up to the completion of pile construction.

In accordance with the Lex Specialis, execution of working piles was also conducted under a separate building permit, actually two of them – one for the tower and podium piles and the other for plaza piles.

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Figure 1. Kula Belgrade Progress of Construction– Sept. 2018 (bottom left), May 2019. (top left), May 2020. (bottom middle), January 2021 (top middle), January 2022 (right)

2.1 Piles testing

A description of the conducted tests is given below:

The osterberg cell test for static testing of tower piles (Ø=1200mm)

This test was conducted as the first in a series of tests while the design of the structure was still in its early stages in 2016. The aim of the test was to confirm the numerical parameters provided by Geotech report [2], as well as the construction methodology. Two test piles with a diameter of 1200 mm in the zone of the tower were tested. After the test, they remained in the ground but were not used as structural supports.

Having in mind the projected bearing capacity of 23.6MN (Geotech report [2]), it was concluded that a conventional experiment with anchor piles would not be economical, so the Osterberg cell method was adopted, which is standard in countries with a large volume of high-rise construction. To the knowledge of the author of this text, this was the first application of the O-cell test in Serbia.

The main advantage of this method is that by the installation of a hydraulic jack in the body of the pile, the parts above and below the jack are in balance with each other, so it is not necessary to make reactive piles and a structure for load transfer.

The tests were performed by Novkol with subcontractors Fugro and Loadtest from England. A force value of 44MN measured on the jack was adopted as a criterion of success for the test. This force corresponded to a pile working load of 22MN (calculated by SOM) with a safety factor of 2.0.

Upon the failure of the first test on two piles in June 2016., it was concluded that the excavation in the upper strata (top 22m) had to be under the protection of casing to prevent the collapse of soil onto the bottom of the borehole. Furthermore, any loose material that remained after the boring had to be removed in order to allow the activation of the base capacity.

At the repeated testing in November 2016, total forces of 55 and 66MN were achieved (Ref.[3]). Subsequent analysis of the test report by the company Terracon from Chicago estimated the load capacity of piles Ø = 1200mm of 25.8MN with a stiffness of 1100 MN/m, and for piles Ø = 1000mm of 5.9MN with a stiffness of 300 MN / m (Ref. [4]). The bearing capacity reported in Geotech report [2] of 23.6 / 5.0MN was adopted in the design, for piles Ø = 1200mm / Ø = 1000mm, respectively, while the stiffness values were adopted based on Terracon's recommendation. In addition to the fact that these tests confirmed the values of the design parameters, they also confirmed the construction methodology, so the same procedure used during the construction of test piles was applied during the execution of working piles.

Static testing of basement structure piles (ø=1000mm) by reactive piles method

All piles with a diameter of 1000 mm are adopted 25 m long, so that their base is at a depth of at least 8.5 m into a layer of marl. The aim of static tests was to directly confirm the numerical parameters of this type of pile, as well as the methodology of their construction.

The tests were performed by the Novkol company with the subcontractor Geomehanika from Belgrade in April 2017. Two groups of 4 reactive and one test piles were constructed for the purposes of the test.

Measured settlements at a working force of 5MN were 5 and 9 mm, while at a maximum test force of 10MN, they were 13 and 37 mm, respectively on piles TP1* and TP2*, or 1.3 and 3.7% of the diameter of the piles (Ref [5]). Both of these were in accordance with the adopted Weltman's criterion (settlements in range of 5-10% of the pile diameter), so it was concluded that both piles have a safety factor greater than two, and that they have the required bearing capacity. Excavation under the protection of the steel casing was adopted for this group of piles, too, for the full length of the pile.

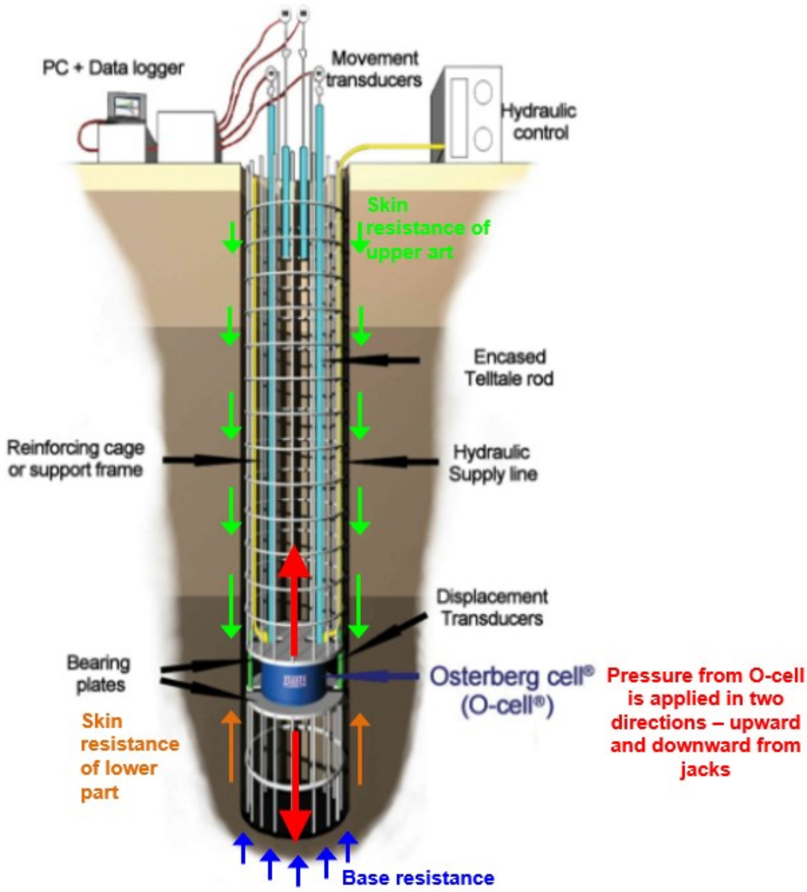

Figure 2. Illustration of principle of O-cell test - Downloaded from http://www.loadtest.com/services/ocell.htm) - comments in colored text were added by Author

Figure 3. O-cell installed into the pile rebar cage (left) and insertion of rebar cage with O-cell into the borehole (right)

Cross-hole logging method for pile integrity testing

The tests were conducted in August 2017. by the Novkol company with a subcontractor, SLP from Slovenia, on working piles in order to check the integrity and continuity of the concrete piles. Four PVC inspection tubes with a diameter of 100 mm were installed in the pile bodies.During the test, probes were inserted into pairs of tubes filled with water - the emitter in one, the signal receiver in the other. The ultrasonic waves were emitted in a combination of pairs - four at the perimeter and two across the diameter, thus covering the most of the cross section of the pile. 100% of piles Ø = 1200mm and 20% of piles Ø = 1000mm were tested. Values in the range of 2700-3300 m/s have been adopted for the minimum acceptable speeds of the ultrasonic wave. On the most of the piles, recorded average speeds were in the range of 3800-3900m/s, which was a very good result, while on 5 piles there was a loss of signal at the top, and on one pile a speed of 3200m/s was measured, at a depth of 25-30m (Ref. [6]). According to the interpretation of the results by the company CSL, the potential reason for such a result was the separation of embedded PVC pipes

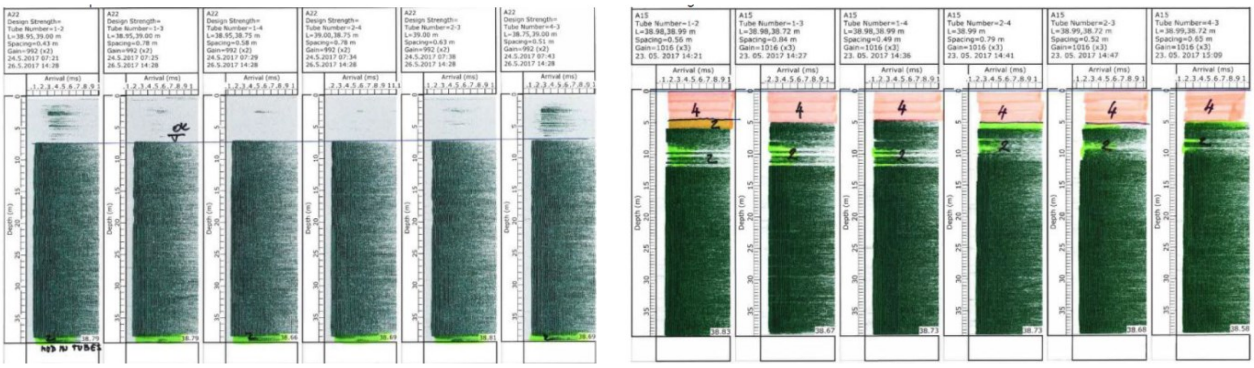

Figure 5. CSL testing records –for a pile withcorrect record (left) and a pile with a loss of signal at the top of the pile (right) – taken from SLP d.o.o. Ljubljana, report: CHA053-05B-2016 Beograd Waterfront Plot 19.1, August 2017.

from the concrete of the piles. In order to prove this, coring was conducted on all 6 piles, with worse results. Based on the visual inspection of the cores, it was determined that the piles had integrity, and based on the breaking of the cubes conducted in the IMK laboratory at the Faculty of Civil Engineering in Belgrade, it was shown that concrete also had sufficient strength.

Dynamic pile testing by falling weight and nmerical capwap interpretation of results.

The tests were conducted in August 7 by the Novkol company with the subcontractor SLP from Slovenia in order to check the load-bearing capacity of the working (production) piles. 20% of piles Ø = 1200mm and 5% of piles Ø = 1000mm were tested. As a criterion of test success, the force of at least 47.2MN / 10.0MN for piles Ø = 1200 / 1000mm was adopted, which corresponds to twice the calculated bearing capacity of piles (Geotech report [2]) of 23.6MN / 5.0MN, respectively.

The first part of the test measures the dilatation and acceleration of the shock wave caused by the fall of the weight and the wave bounced off the base, which are converted into wave force and speed and displayed on a graph as a function of time. Then Numerical CAPWAP analysis, which uses the equations of wave motion, determines the static bearing capacity of the pile at the base and shaft.

The results of the CAPWAP analysis were in the range of 48.6 - 68.7MN for piles Ø = 1200mm, while for piles Ø = 1000mm they were in the range of 6.7 - 23.4MN. On 5 of the 11 tested piles, which are located on the Northside of the plot, the force estimated by the CAPWAP method was less than the required 10MN, so the bearing capacity was estimated as less than 5.0MN (Ref. [7]).

Based on the additional numerical analysis that compared the calculated forces with the capacities obtained by tests, the following was concluded:

- Pressure: Only on 2 out of 208 piles with diameter Ø = 1000mm the calculated force was higher than the bearing capacity adopted by averaging the test results by zones. The maximum design force of 3.85MN exceeds the load capacity of 3.5MN (adopted for the zone between the J-L axes) by 9.8%. Based on the diagrams given in the test report, the piles at this level of force are still in the linear range, so that these 2 piles would have a settlement greater than 1 mm compared to the settlement corresponding to the allowable force value, so there would be no significant consequences for the raft and the rest of the structure.
- tensile: the design forces do not exceed the tensile capacity estimated on the basis of a dynamic test.

According to these results, it was assessed that the piles achieved sufficient capacities.

Figure 6. Testing by 42t falling weight from the elevation of 0.3 to 1.5m

PIT testing

This is another type of test that checks the continuity and integrity of the piles. It was conducted in 2019 by MACE, as a part of the procedure of handing over the piles to the Main Contractor (Ref. [8]). All 270 piles were tested.

In addition to the measuring of the speed of the wave caused by the impact of a hand hammer on the top of the trimmed pile, which is expected in the range of 3000-4500m/s, the test also evaluates the quality of response based on the clarity of signal reflected from the base. Results are divided into categories ranging from the best one AA (reflection is clear) to the worst one IR (an unclear record). The speed of the wave was in almost all cases above 4000m/s, never below 3500m/s, and the quality of the response was rated as AA or AB (mostly AB - there are no serious shortcomings).

2.2 Construction of the piles

Prior to any works on Kula foundations, existing terrain was on the levels between 74.5 – 75.5 meters above sea level (MASL). During the UXO works, the soil was excavated in the depth of 4-5m, to the level of app. 70.5 MASL and then the site was probed to detect potential unexploded objects that remained from WWI and WWII.

Dewatering was enabled by dewatering wells embedded into the existing embankment. They were designed by the Faculty of Civil Engineering in Belgrade and installed by *PMC Millennium*.

Once Novkol entered the site they formed the working platform on level 71MASL, by adding the crushed stone.

The construction method was fully based on the procedure established during the successful pile testing.

All 270 piles were constructed in 2017.

The ones that were anticipated for dynamic testing were overpoured to the level of the working platform, while the rest were overpoured for about 1-2m.

2.3 Excavations

When the construction of the Tower began in February 2019, the first step was the reconstruction of the dilapidated embankment. During these works, dewatering wells from the UXO stage were preserved. They stayed in function until the completion of the construction of the basement structure when they were turned off. Surface water drainage was performed by submersible pumps - as needed.

In the next step, a wide excavation was made to the final levels in the range of 68.1-65.8 MASL. It is interesting to mention that during these works Main Contractor ran into one unexploded object, so they had to be stopped for two days until the object was excavated and safely activated.

Additional excavations had to be made in the zone of the 4m deep elevator pits, where the final excavation level was 62.1MASL. The vertical excavations were conducted under protection by Larsen sheet piling, which was braced in the corners of the pit by the horizontal steel struts, and on the mid of the section, they were fixed to the piles.

Drainage of this zone was performed by a combination of dewatering wells in the embankment (preserved from UXO works) and three wells added in close proximity of the elevator pit, as well as submersible pumps used for surface waters.

After the excavation was completed, the piles were trimmed, the blinding was constructed to allow for the execution of waterproofing. Sika Proof A system was adopted. It is specific because it chemically reacts and bonds with fresh concrete to form an impermeable water-impermeable layer. Critical joints - where it was impossible to achieve continuity were treated by SIKASWELL water stop and provided with injection hoses as the 3rd line of protection. This waterproofing system was the most expensive product on the market at the time and requires extreme attention in execution as the reinforcement works are conducted overlaid waterproofing.

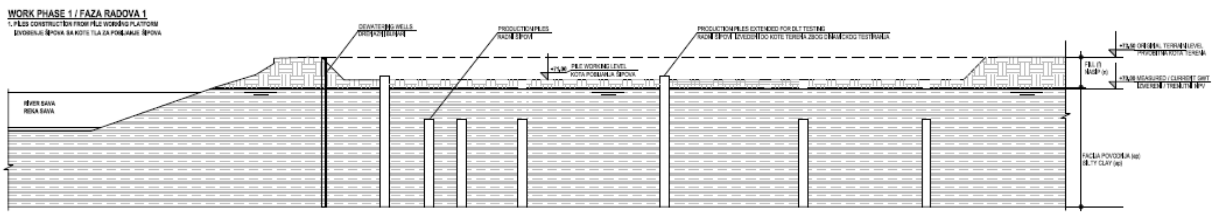

Figure 7. Illustration of the execution of piles form the working platform

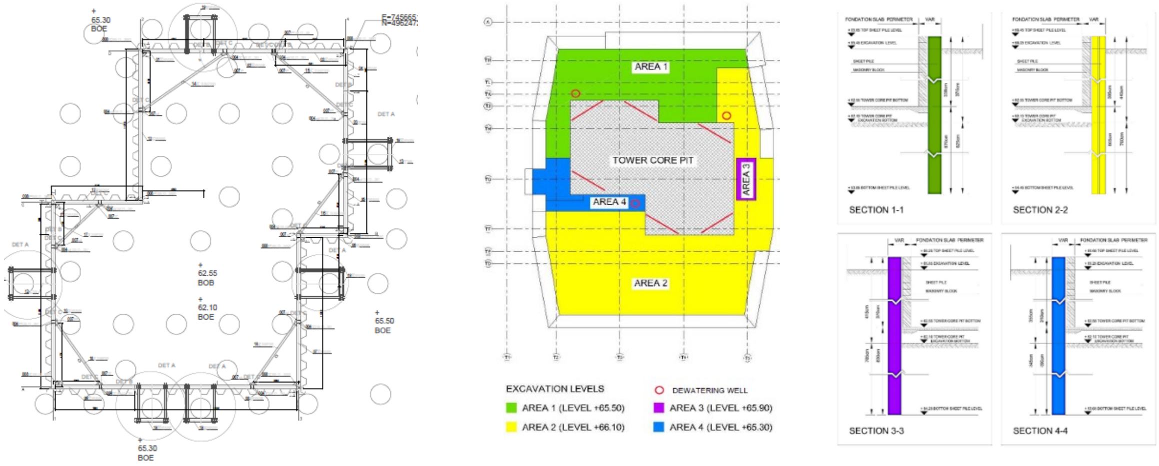

Figure 8. Layout of sheet piling with bracing structure (left), plan with the location of 3 additional dewatering wells (middle) and characteristic vertical sections (right) – taken from RS_BW_P19.1_C_025_PZM_MST_0003_Rev.01_Site Specific Method Statement for Installation of Sheet Piles around Tower Lift Pit

3 Construction of raft

The foundation slab, with a total volume of 14000m^3, was concreted in the multiple pours shown in the picture below. Parts outside the tower zone are divided into more than ten parts.

Part of the slab in the tower zone was divided vertically in two pours (1.1 and 1.2 in the above figure). The deeper one (1.1) was located in the zone of elevator pits. It was 2.8 m thick, about 400m^2 in plan with a volume of 1150m^3 and was cast the first.

The thickness of concrete in pour 1.2 ranged from 2.8 to 4m with an area of about 1600m^2, so the total volume of concrete was app. 4750m^3.

Such configuration imposed significant challenges both in terms of reinforcement and concrete works. In the former case, the Contractor had to provide stability of rebar cage. This was achieved by the adoption of two chairs Φ32 with 2 legs per square meter. They were horizontally interconnected and braced by inclined bars along the perimeter of the zone.

In terms of concrete works, the volume of 4750m^3 may be regarded as one of the biggest concrete in Belgrade, ever. Decision to cast it in one turn was based on two reasons - division into smaller parts was basically impractical and division by height was discarded as to avoid the formation of cold joints.

This large amount of concrete required careful selection of the components of the concrete mix. This primarily refers to the type of cement - CEM III / B (cement with over 65% slag) was used, which reduced the risk of thermal cracks in mass concrete. The reduced mobility / workability of concrete with such cement was overcome by the use of hyperplasticizers. The large concrete casting area, as well as the logistics of concrete supply from several factories, demanded the provision of a significant time reserve, which was achieved by using retarder admixtures that delayed the starting of the setting of concrete for not less than 16 hours.

This enabled concreting in horizontal layers 40-45 cm thick (in deeper parts with a smaller working area) up to 20 cm (in the upper parts) and to start the casting of the next layer prior to the setting of the previous layer. Adjacent vertical layers were interconnected by vibration.

Concrete grade C40 / 50 was produced by Gradient, Nexe and Karin Komerc factories, in a total of 6 factories with a production capacity of 45 to 77 m^3/h. Over 30 mixers participated in the delivery of concrete. Three pumps were used simultaneously, and the fourth one was in reserve.

Casting was done in August 2019 and took about 60 hours. Over 70 people participated in the action under the constant control of the contractor's quality team, as well as the constant supervision of DNEC.

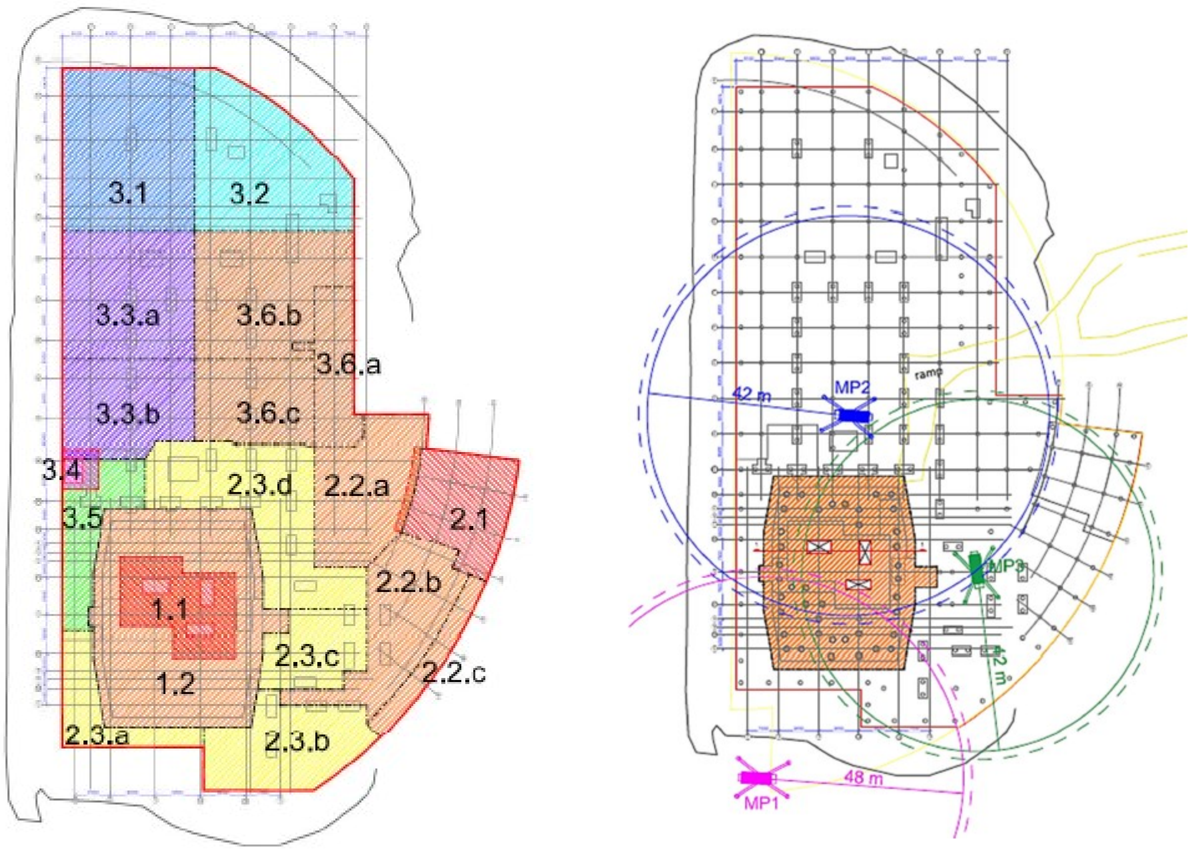

Figure 9. Plan of pours for Kula raft (left) andplan with location of concrete pumps for the pour under the tower (right) – taken from RS_BW_P19.1_C_025_PZM_MST_0006_Rev.02_Site-Specific Method Statement for Raft pouring

Figure 10. Reinforcement works on pour 1.2 – raft pour under the tower

Figure 11. Casting of raft pour 1.2 under the tower

4 Superstructure

During the summer of 2019. it was not only that the raft under the tower was cast, but also several parts of the raft under the basement structure were completed. This enabled simultaneous progress both on the basement and the tower superstructure.

4.1 Corewalls

Kula's core walls were constructed in jump form. Doka's automatic climbing formwork SKE 50 plus was used to jump / climb the core with hydraulic jacks. They were cast in Doka's shuttering system Top 50, which was constructed in height of 4.1m to allow for concreting in one turn of all typical storeys with a height of 3.45m and most of the non-typical storeys. In the case of storeys with a height greater than 4.1m such as GF, Level 2, 11, 12, 19 and 39 to 42, they were cast in two steps. In total, there were 52 castings and 57 climbing steps from B1 level up to the top roof to achieve the cores, starting from level -7.97 and reaching a level of +168m (relative to Architectural zero level).

Reinforcement of wall segments as well as for link beams (described in the 1st part of the article) was prefabricated on the ground, lifted and installed into the shuttering by crane. Horizontal U bars at the wall ends were added.

Reinforcement assembly also included bent out elements by Bindax Company. They were to allow connecting of slabs to core walls.

Based on design requirements, staggered lapping of vertical rebars (50% in one section) was adopted from raft to Level 5 (base of the core, large openings) and from Level 12 to 15 (transfer of forces between central and satellite cores), while on all other levels, 100% of vertical rebars were lapped in one section.

The concrete grade was C50/60. The mix was designed to enable optimum workability with fresh concrete. A concrete placing boom was installed within a core and was jumped together with the formwork. It was used for the concreting of all tower members: core walls, columns, slabs, and beams. Typically, for this construction method, core walls were ahead of slabs for 3-4 storeys. This gap in progress between the walls and slabs was limited by the headroom for the crane above, the jump form and the reach of placing the boom below.

In general, the minimum time for shuttering was 12hours. However this timing was adjusted for concreting in cold weather conditions when thermo couples were built into the walls to provide information about the temperature within a wall that allowed to contractor and engineer to decide on shuttering stripping time.

Figure 12. Construction of core walls in jump form

Figure 13. Link beam reinforcement with Bindax pullout boxes on top - installed into wall rebar cage (left and middle) and core wall reinforcement (right)

4.2 Transfer structure

Transfer columns

As described in the first part of the article, columns located in the corners of the plan are sloped in the transition zone (between Levels 12 and 20), while six central columns (East and West from the central core) branch at Level 14-15 into pairs of columns. The ones closer to the core keep verticality all the way, while the outer ones slope from Level 15 to Level 20 where they turn vertical up to the top of the building. The corner columns are circular, while the branched ones are rectangular.

A very high utility ratio, primarily in terms of axial forces was common to all transfer columns. They were designed as reinforced concrete columns, with a reinforcement percentage of 4% outside lap locations, which corresponded to the maximum percentage of reinforcement defined by Eurocode 2 [9]. Since the column rebar was found to be in compression for all design combinations,100% of the rebar was lap spliced per section.

Due to the high rebar ratio, as well as the specific geometry of the columns, in addition to high compressive strength, good workability of concrete was required, too. This led to the use of high-performance basic materials such as cement CEM I 52.5R and aggregates of eruptive origin. In order to achieve the required compressive strength, as well as the time required for transport which took about 1hour and weather conditions that were an additional unfavourable circumstance (summer), using the appropriate combination of chemical admixtures, concrete of consistency class S5 was delivered to the construction site. For concrete, grade C55/67 was adopted. To the knowledge of the author of this text, such a grade was to be applied for the first time in Serbia by the concrete factories that supplied the Kula site.

Upon the completion of rebar cage, columns of circular cross-section were made in a circular formwork called DOKATop 50, which consisted of two semi-circular parts connected by bolts. Sloped columns of rectangular cross-section were constructed in the DOKA Framax system. At the point of column branching (Levels 14-15), a joint, outer aluminium shuttering, enclosed both branches, while they were internally divided by a wooden partition.

Stability during the construction stage was provided by steel strut stabilizers.

Two days after the casting, the upper part of the formwork was removed. Their surface was covered with geotextile to ensure further curing of concrete. The lower part of the formwork was removed once 70% of the designed strength was reached, but not less than 5 days after concreting. After removal of the formwork, the columns were re-propped for additional 5 days.

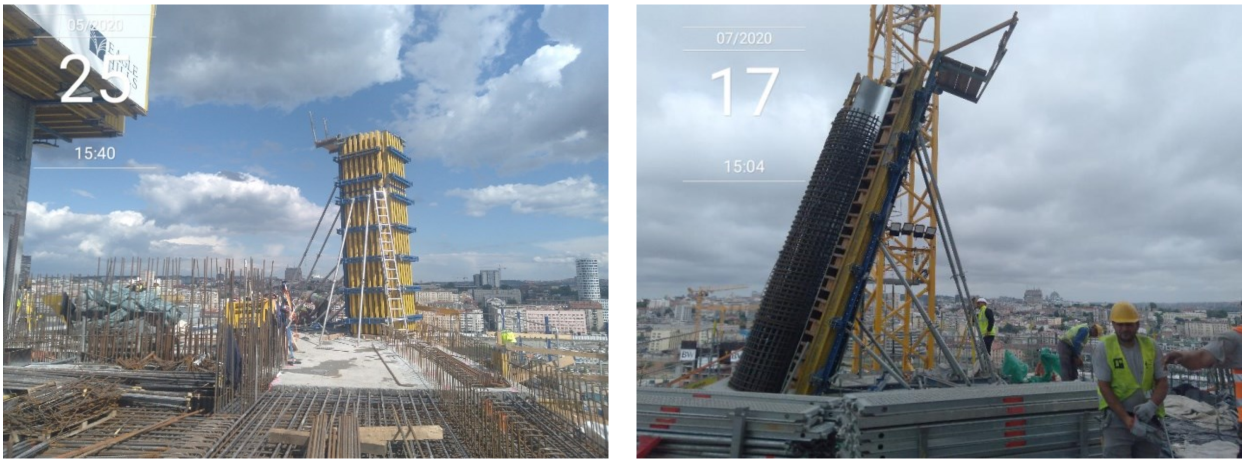

Figure 14. Construction of circular transfer columns Level 12-14 (left) and Level 16-17 (right)

Figure 15. Construction of transfer branching columns Levels 14-15 (left) and Levels 15-16 (right)

Transfer slabs and beams for lev 12, 14, and 20

As described in the previous text, kinking of transfer columns at Levels 12, 14 and 20 originated turn forces in horizontal planes,which in some cases acted towards the core and in some away from it, tending to split the slabs on these levels. The tying reinforcement and tendons adopted in prestressed beams restrained these tension forces.

In the case of rebar ties, there were 5Φ32 in the top and bottom zone on Levels 12,10 Φ32 in top and bottom zone on Lev14, and all of Φ32 bars on Level 20 that had to be spliced by mechanical couplers. As the Halfen MBT type, popularly called the „crocodile“ did not require the turning of bars, it was adopted. Instead, the bars are just inserted into a coupler and fixed to it by bolts, tightened until they snap. In case that any of the bolts did not reach the final position, the coupler was discarded and replaced by a new one.

Figure 16.Transfer beams on Level 12 (top row) and on Level 20 (bottom row) – details of coupled bars (bottom left) and discarded coupler (bottom right)

Figure 17. Lenton couplers for Transfer beam on Level 20 at core wall in Contractor’s shop drawing (left) and on site (right)

Another type of rebar splicing was adopted at the faces of the core walls where threaded Lenton couplers were used.

Another specific issue was the geometry of transfer slabs. While the slabs were shrinking from Level 12 to 20 in a North-South direction (direction parallel to the Sava River) they expanded in a perpendicular direction. Accordingly, there was an issue of access and supporting shuttering in the East-West direction because the upper slabs were projecting out of the perimeters of the lower ones. The issue was resolved by the application of Doka Staxo towers that were fixed to the structure by a system of anchors and lashing straps.

4.3 Slabs of pt on the hotel and residential levels

As described in the first part of the article, typical slabs on hotel and residential levels were designed as PT slabs. This included slabs from Lev03 to 11 (hotel levels), 16, 18, 19, and 21 to 39 (residential levels), so the great majority of Kula slabs.

The PT works were assigned to PT Subcontractor, company Strong Force Balkans, daughter company of Australian Strong Force, while the reinforcement works were conducted by Construction Centre Manojlović company from Serbia, while the main contractor coordinated them at all work stages.

Having in mind the dimensions of slabs which extended up to 45m in one direction, each PT slab was divided in the design stage into two pours. This was also practical in terms of the organization of work because it allowed for the parallelization of work. The Contractor achieved casting adjacent horizontal / vertical pours in a 2 / 4-day span, respectively, on some of the residential slabs. The slab under construction was propped and supported by three slabs below. The density of propping on the lowest of three levels was 50% of that of the two upper ones.

The PT system consisted of 12.7mm mono strands. They were inserted by a cable pusher machine into the flat ducts with 3 to 5 slots to form the tendons. A live anchor was mounted at the tendon end at the outer perimeter of the slab, or at the pan box in case it was not designed to reach the slab end. Dead end anchors were formed on site by the onion jack tool and stressing pump. Both ends were supplied by anti-burst spiral reinforcement. Tendon profiling was achieved by fixing the tendons to the plastic chairs at the required height, adopted at approximately 1.0-1.2m spacing, and stapled to the wooden shuttering with a heavy duty stapler to secure their geometry during the concreting.

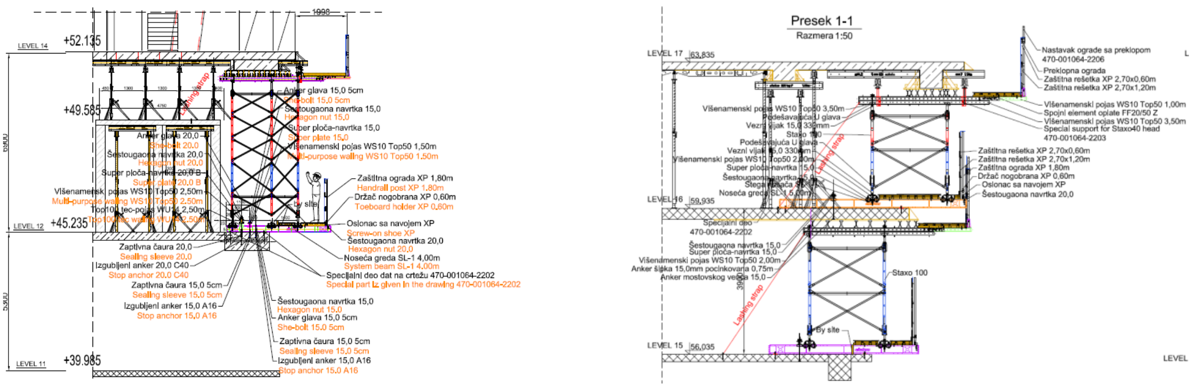

Figure 18. Illustration of extension of slab edges on transfer levels - Levels 12-14 (left) taken from RS_BW_P19.1_C_025_PZM_DDC_0048_Rev.00_Doka Formwork Plans_L12 formwork Platforms supporting L14 Slaband Levels 15-17 (right) – taken from RS_BW_P19.1_C_025_PZM_DDC_0058_Rev.01_Doka_Formwork Plans and Platforms supports for L17

Figure 19. Live end with pan box (left) and formation of onion head dead end (right) (both obtained from Contractor's document Method Statement for Post-tensioning worksRS_BW_P19.1_C_025_PZM_MST_0021_Rev.00_Post Tensioning Works)

Stressing was conducted in two stages. Stage 1 took place approximately 12 hours after the casting, but not before the concrete cube strength reached 10MPa. It was done to the level of 25% of the total jacking force. The remaining force was applied once the concrete cube strength reached 25MPa (the design concrete grade was C40/50), which was usually less than 48 hours after the casting. The total jacking force was set at 80% of the ultimate tensile strength of the strands, or 1480MPa = 0.8*1860MPa.

Elongations measured during the stressing were submitted for the engineer's approval in a form of a stressing record report, and only after the approval of the Engineer the cutting of strands was conducted.

The grouting was usually done within a 2-3 week span after the stressing of tendons under the engineer's supervision.

As for the reinforcement works, it is interesting to mention that the slab capacity in terms of punching shear was provided by double headed Halfen studs with a 12mm diameter welded to Halfen HDB shear rails, that were inserted into the installed slab reinforcement. Due to the requirements in energy efficiency regulations, the balconies had to be completely thermally insulated from the main slab. This was achieved by thermal connectors made by Forbuild Company. The connector consisted of a central part made of thermal insulation, through which two steel plates were embedded to enable the transfer of shear forces, while the moment action was enabled by straight rebars placed on the top of the section and three high strength bearing blocks placed at the bottom to transfer compression forces.

In terms of work related to formwork and shuttering, it is worth mentioning that all the slabs above level 11 had to follow pre-setting schemes adopted during the Value Engineering design by which the perimeter of the slab was supposed to be executed higher than the centre of the slab to allow the slab to achieve horizontal geometry once the differential settlements between the core and columns take place over the years of service. Typically, the difference in levels was 10mm up to level 20 (dark blue hatch on below figure), but it was 20mm on residential levels (21–40) (green hatch on below figure).

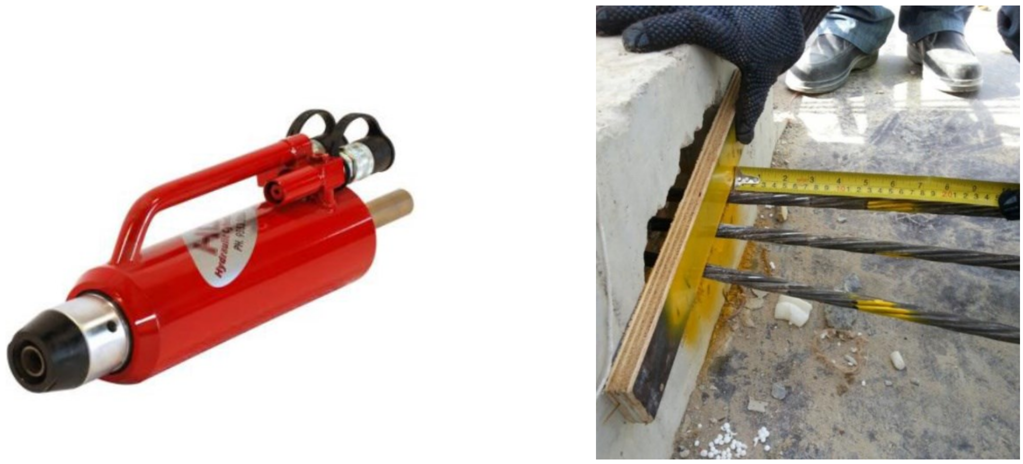

Figure 20. Mono strand stressing jack (left) and measurement of achieved elongations (right) (both obtained from Contractor's document Method Statement for Post-tensioning worksRS_BW_P19.1_C_025_PZM_MST_0021_Rev.00_Post Tensioning Works)

Figure 21. Halfen shear rails with double headed studs (left) and thermal connector at balcony location (right)

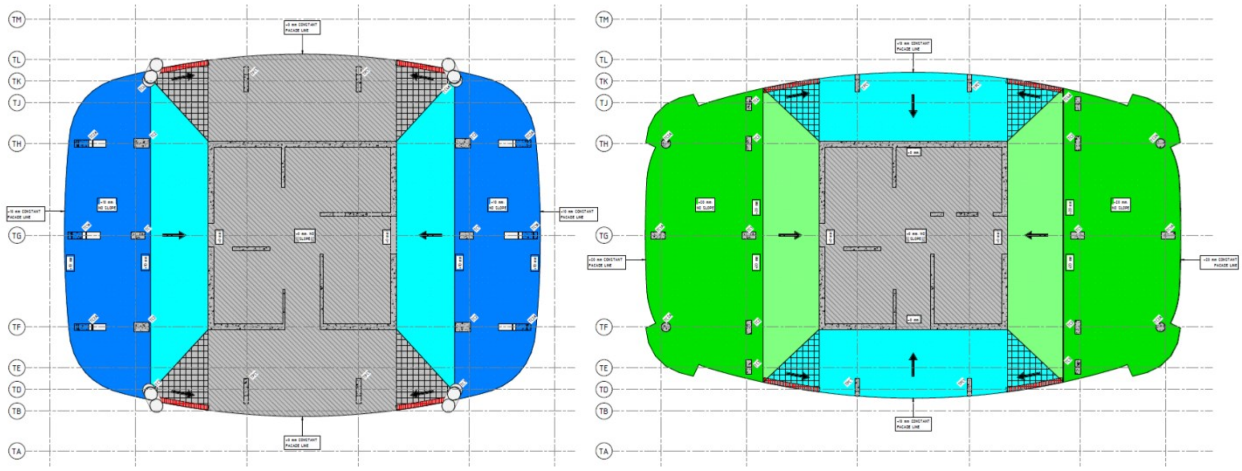

Figure 22. Pre-setting scheme for levels 18,19,20 (left) and 21-40 (right)

5 Conclusion

The work on the primary structure was completed in August 2021. The tower façade was completed by the end of 2022, so the podium façade, MEP, and architectural works are still ongoing.

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