Građevinski materijali i konstrukcije
Building Materials and Structures
journal homepage: www.dimk.rs

doi: 10.5937/GRMK2104269M UDK: 624.042.7(497.11)
Technical paper

Kula Belgrade – Part 1 - Specific topics of structural design

Nemanja Miljković[*1)], Mladen Milićević[1)], Svetlana Ristić[1)], Darko Popović[1)], Vanja Alendar[1)]

[1)] *Company DNEC d.o.o. Belgrade, Serbia*

Article history

Received: 13 November 2021
Received in revised form: 18 November 2021
Accepted: 24 November 2021
Available online: 30 December 2021

Keywords

Tall building,
Transfer structure,
Piles,
Composite structural members,
PT structural members,
Non-linear beam analysis,
Abaqus software

ABSTRACT

Kula Belgrade is the tallest building within Belgrade Waterfront project, located at the right bank of the Sava river. It is envisaged as the future landmark of Belgrade and pivotal point of Belgrade Waterfront development. It consists of 168m high - 42 storey tower, podium and 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 transition is achieved through 7 floors. Such configuration imposed significant demands to structure. The paper addresses design of structure, which has been divided in two stages – piles and structure above the piles. Former was provided by SOM company from Chicago, USA; latter by AECOM company from UAE, both as per American International Building Code. DNEC company from Belgrade was a member of Joint Venture of local companies in charge for nostrification of design and permitting process. Check of design was conducted per Eurocodes. During the construction stage, DNEC was in role of Engineer but was also actively involved in structural value engineering process in which the composite structural members (reinforced concrete with embedded steel) of transition zone were converted to reinforced concrete or post-tensioned members. Link beams of main core that comprised embedded steel plates were redesigned as RC beams, but due to the openings in web their adequacy was checked by non-linear analysis in Abaqus software.

1 Introduction

Kula Belgrade is the tallest building within Belgrade Waterfront (BW) project, located on plot 19.1, at the right bank of the Sava river. It is envisaged as the future landmark of Belgrade and pivotal point of BW development. Accordingly, the Investor „BW Kula“, Ltd. has engaged prominent architectural company Skidmore, Owings and Merill LLT (SOM), Chicago, USA as an Author.

In accordance with the Lex Specialis law, design process was divided in two stages: Stage I – Foundation design and Stage II – Design of Kula.

Structural Consultant in stage I was SOM's Structural department. Their geotechnical Consultant was Terracon from Chicago. Design of structure and foundations was based on International Building Code (USA norms).

Joint venture (JV) of local companies Energoprojekt Urbanizam & Arhitektura, Energoprojekt Entel and DNEC was in role of Local Consultant in charge for nostrification of design and permitting process. Based on initial meetings with Republic Revision Committee it was decided that the check of the structure is conducted as per Eurocodes which were not Serbian official code at a time, but were expected to become official by the end of the project. Responsible Engineer for foundation design was Professor Miloš Lazović. Geotech report [1] was provided by CIP Institute from Belgrade. Design of piles was completed in 2016 and Novkol company constructed all piles by 2017 under the supervision of MACE company.

Structural Consultant in stage II was AECOM company from Abu Dhabi. JV of local companies had the same role as in stage I. In cases where DNEC's checks as per Eurocodes found insufficiencies of structural members, they were strengthened to comply with Eurocodes. In cases where it was estimated that the structure was over-dimensioned in AECOM's design, no changes were made in order to keep AECOM's liability on packages submitted to Client.

Design was completed by the end of 2018 and construction started in February 2019, by JV of companies Millennium team from Serbia and Pizzarotti company from Italy (PZMT). Engineer's role as per FIDIC was assigned to DNEC.

Contractor has conducted structural value engineering (VE) exercise during the construction stage. His consultant was BG&E company from London, UK. Officially, DNEC was approver of VE design, but practically DNEC worked together with BG&E on this package.

By the time of the writing of this text (Oct. 2021) the main structural works are completed, while facade, MEP and architectural works are still in progress.

* Corresponding author:
E-mail address: nemanja.miljkovic@dnec.com

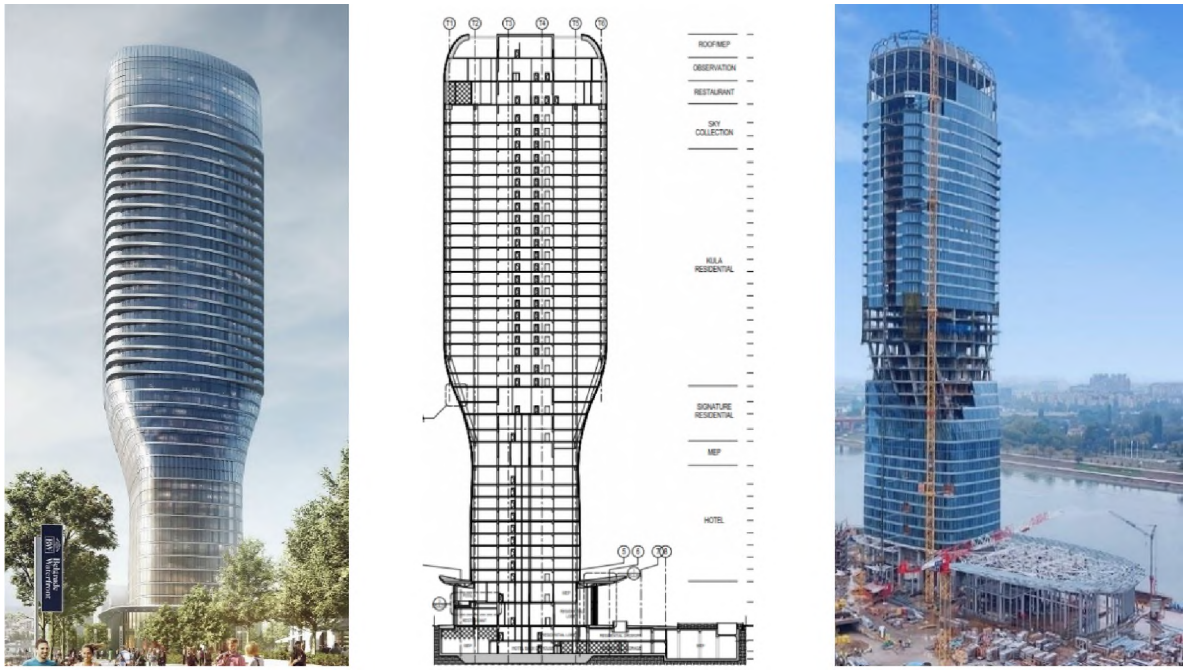

Figure 1. Kula Belgrade– render (left), vertical section (middle), progress of works in Oct 2021 (right)

2 About kula

Kula Belgrade is 168 m high, 42-level building located on the right bank of the Sava river, separated from it only by embankment. It consists of tower of mixed use, podium and basement.

Bottom of the tower is allocated for five-star hotel with 119 rooms. MEP level divides hotel part from upper residential part of the building, which comprises 220 branded apartments. 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 transition is achieved through 7 floors. Typical storey height is 3.5m and maximum one 6.6m.

Podium is 13m high one-storey structure connected to the tower on its North side. Overall dimensions in plan are approximately 50 x 50 m.

There is a two-level basement (mezzanine and B1), each with storey height of 3.5m. It has irregular shape in a form of a circle segment with 150 m long straight going parallel with the river. Maximum dimension in perpendicular direction is approximately 100m. It comprises parking, loading dock, drop off area and back of house (service) areas. Columns' grid is 8.5 x 8.5 m. Ground floor is divided into paved and landscaped areas. Basement is enclosed by the boundary wall which retains not only the soil but also the underground water.

Structure is integral with no expansion joints between the tower and podium and basement.

At the South side of the plot, Kula is connected to the adjacent Galerija shopping mall, by the 40 m long footbridge, designed by Arhipro company. Structurally, it is a simple beam that spans between the two buildings without intermediate supports. Supporting on Kula's side is enabled by RC beam on Level 03.

3 Key input parameters for structural design

3.1 Geotech report and pile testing

Based on Geotech report [1] and its amendment [2] typical soil profile consisted of:

- Infill „n“: Silty clayey materials, rarely sand and debris. Thickness of layer is 1-2 m.
- Still water facies „am”: Clays, silty muddy and sporadically sandy clay. Thickness of layer is 1-2 m.
- Floodplain facies „ap”: Clayey silt and clayey sandy silt. Thickness of layer is 10-12 m.
- Riverbed facies „ak”: Sand and gravel. Thickness of layer is 8-10 m.
- Marl „L”: Marly clay, marl and marlstones. Thickness of layer is 13-15 m.
- Limestones „K”: Reef organic limestone in upper layers, more compacted Urgonian limestone in lower ones.

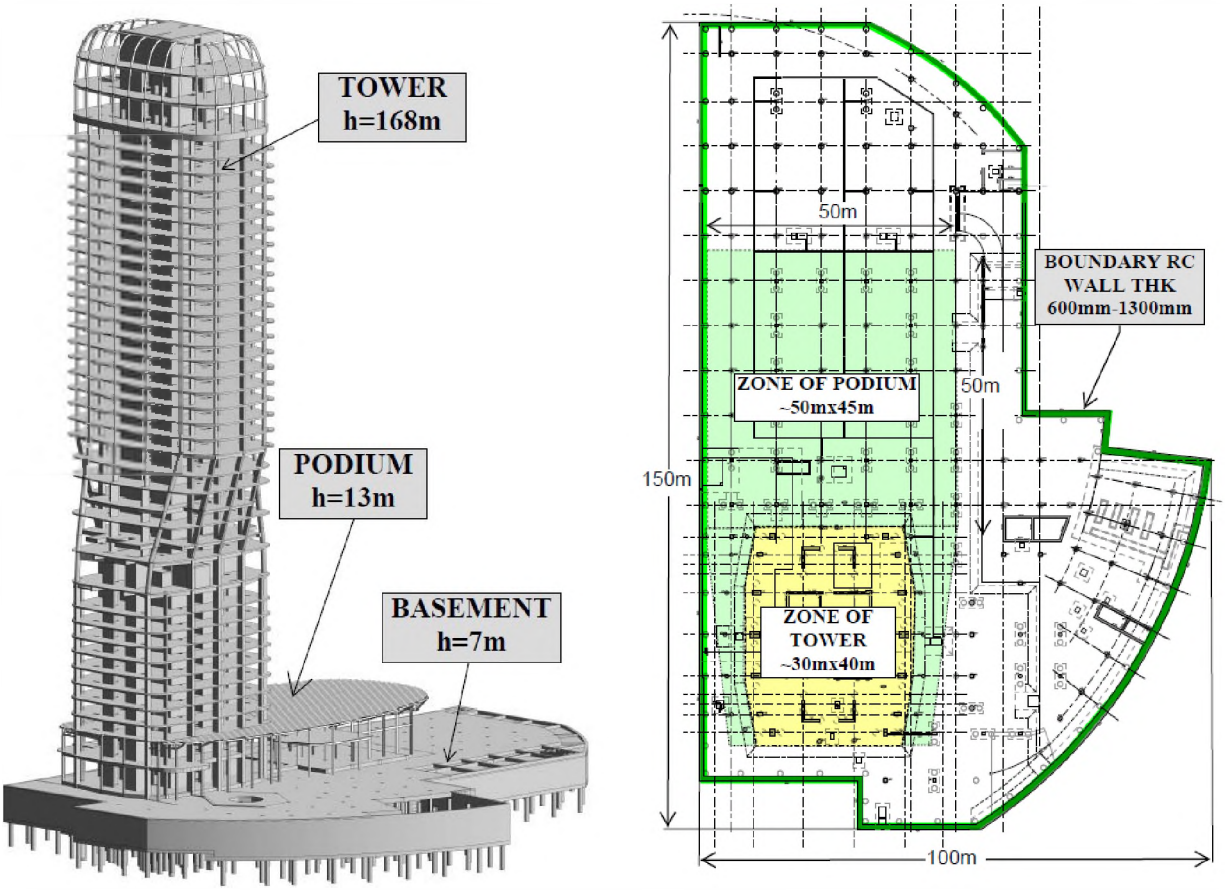

Figure 2. Kula Belgrade – functional units

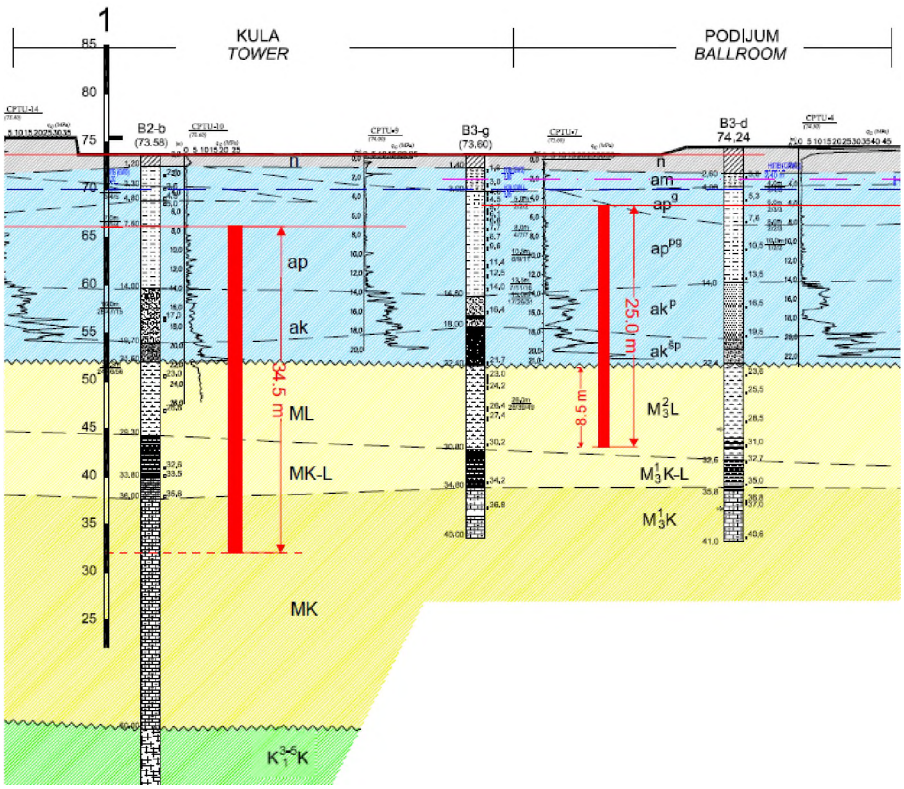

Figure 3. Typical soil profile with drawn piles Ø1200mm (left) and Ø1000mm (right)

Due to poor mechanical properties of upper layers, deep foundation in form of piled raft was recommended foundation option. Geotech report provided estimation of piles capacities of 23.6/5 MN for Ø1200/1000 mm piles anticipated for tower/basement areas, respectively. Capacity was confirmed by comprehensive pile testing program which included:

- Testing of two Ø1200 mm test piles by O-cell method
- Testing of two Ø1000 mm test piles by static test with anchor piles
- Cross hole integrity testing at 100% piles Ø=1200 mm and 20% of piles Ø=1000 mm
- Dynamic testing at 20% piles Ø=1200 mm and 5% of piles Ø=1000 mm
- PIT testing at 100% of constructed piles.
- Geotech report defined the design of water table at +74.0 meters above see level (MASL).

3.2 The report on the specific elastic response spectrum of local soil

Seismic action for analysis of Kula structure was defined by seismic micro-zoning report [3], based on seismological and geophysical testing conducted on Kula site. Key findings of this report are summarized in below bullets and elastic spectrum of accelerations.

- Reference Peak Ground Acceleration on Type A ground ..... a_{gR}=0.06g
- Importance Factor (EC8, T 4.3 Structure category III)..... γ_I=1.2
- Design acceleration ..... a_g=1.2·0.06g=0.072
- Ground type .....Type S2
- Factor for local soil .....Ss=2.3
- Dumping ratio (concrete structure) ..... ζ= 5%
- Lower period limit of the constant spectral acceleration branch. ..... T_b= 0.18 s
- Upper period limit of the constant spectral acceleration branch..... T_c= 0.42 s
- Value defining the beginning of the constant displacement response range of the spectrum ..... T_d= 1.5 s

3.3 Wind tunnel report

Canadian company RWDI conducted wind testing and provided Wind tunnel testing (WTT) report for the design of Kula structure [4].

Basic parameters:

- Designed wind speed (Maximum mean ten-minute wind speed, on height of 10 m, for recurrence period of 50 years) .. $V_{m,50,10}$=22.0 m/s
- Terrain category..... III
- Orography factor..... $C_o(z)$=1
- Turbulence factor. .....K1=1
- Structural factor.....$C_s \cdot C_d$=1
- Air density ..... ρ_0=1.25
- Windward side exposure coefficient ..... C_p=0.8
- Leeward side exposure coefficient ..... C_p=0.7

Based on comparison of wind forces, calculated per Eurocode and WTT report it was observed that the ratio of coded to test value was approximately 3/1, which suggested that Kula's shape is highly aerodynamic. However, the wind load reduction was limited to 20% based on recommendations provided in ASCE 7-10 – 31.4.3 [5], so the design of main structure was conducted with 80% of the coded wind load.

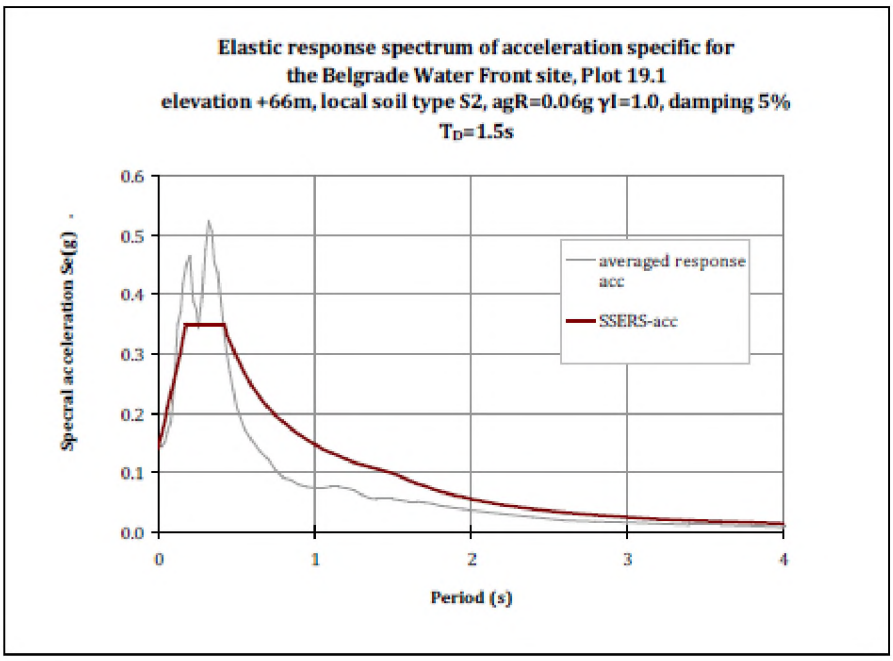

Figure 4. Elastic response spectrum of acceleration specific for Kula location (BW plot 19.1)

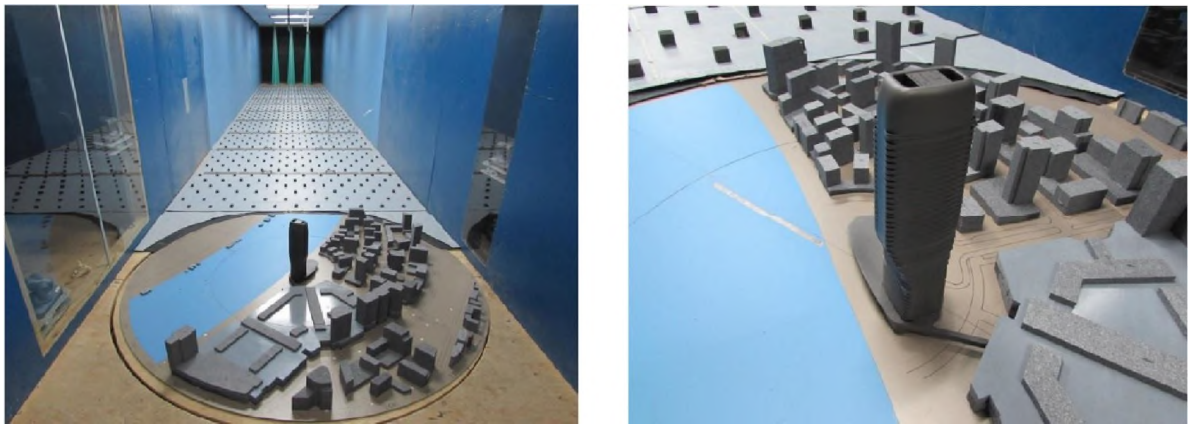

Figure 5. Wind Tunnel Testing

3.4 Materials

Concrete grades were adopted in range from C32/40 for ground floor (GF) and basement mezzanine (B1M) slab, C40/50 for raft, basement wall and superstructure slabs, up to C50/60 for core walls and as high as C60/70 for columns in order to minimize their cross-section dimensions and impact on architecture.

Reinforcement was uniformly adopted for all structural members as B500C.

Structural steel was uniformly adopted as S355 for all steel structural members.

4 Kula structure as per original solution

4.1 The tower structure

Structural concept was defined by SOM's structural team in the first stage of design. It was later taken over and developed to detail design level by AECOM. Both teams used IBC code while DNEC conducted checks as per Eurocodes.

Structural slabs were adopted as two-way reinforced concrete (RC) slabs. Thickness of the typical slab was 200 mm with 450 mm thick drop panels at columns. Concrete grade was C40/50.

Columns were designed as RC columns. Dimensions range from 1100 x 1700 mm at basement levels to 400 x 1200 mm on top floors. Concrete grade was C60/70. They were treated as secondary structural members in seismic analysis.

Lateral stability of the structure is provided by core walls. Dimensions of the main core are 17 x 17 m in plan and it runs from the raft to the top. It is backed up by two satellite cores adopted in lower levels. Thickness of walls is typically 500 mm up to Level 28 and 400 mm above this level. Concrete grade was C50/60.

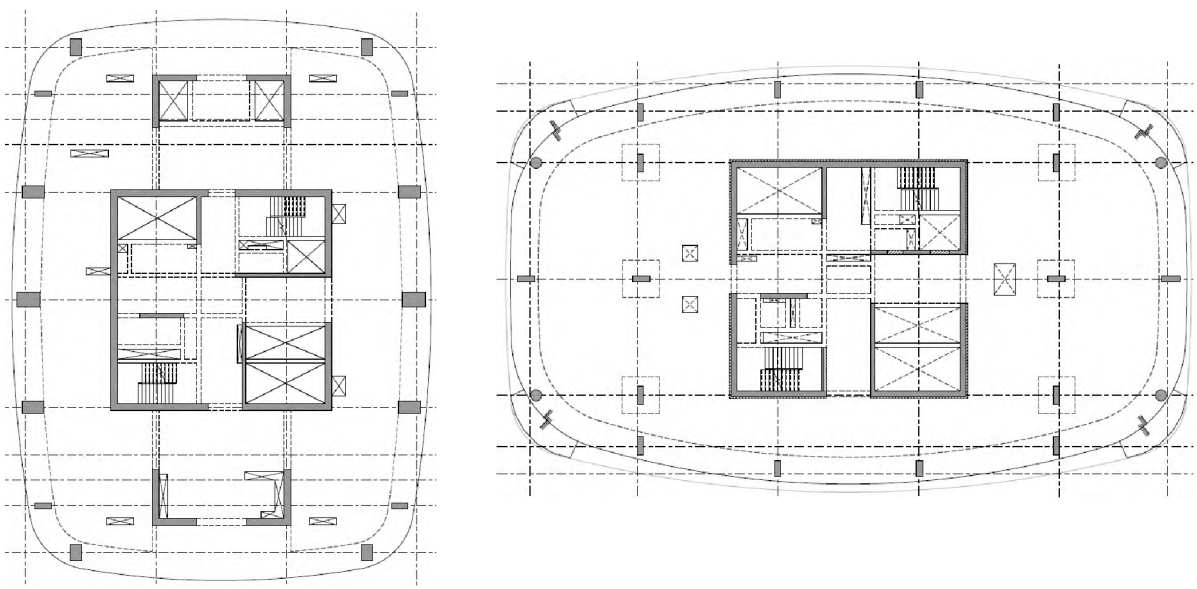

Figure 6. General arrangement (GA) plans of typical Hotel and Residential level as per original design

4.2 The transfer structure

Slabs gradually change their shape in the transition zone of the building, between Level 12 and Level 20. Below Level 12 they are elongated in direction parallel to the river, while above Level 20 they are elongated in perpendicular direction. Columns are vertical in bottom part of the building. Then the columns in the corners of the plan turn into sloping columns at Level 12, 14 and then they turn again on Level 20 to vertical part of level. Each of six central columns (located on the East and West side of hotel floor) branch at Level 14-15 in two columns (see Figure 7). 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 to become vertical up to the top of the building. These kinks in columns geometry originate turn forces in range of 5-10 MN, which tend to split the building so the special "transfer" structure was design to resist these forces.

Transfer structure consisted of Transfer beams on Level 12, 14 and 20, which were adopted as composite beams, with embedded steel plates in mid of section and reinforced concrete around them. Former part was originated by column kinks to take the turn forces, while latter part took local loads from floors and provided fire resistance to the entire section. Embedded steel plates were adopted within core walls in their full lengths to allow for continuity in plan, thus forming horizontal ties starting from column-beam joint at one end, running through the transfer beam, core wall, another transfer beam, terminating at symmetrical column beam joint on other end of plan. Columns in transition zone were adopted as composite columns. Their capacity and stiffness were enhanced by embedded steel profiles in transition zone.

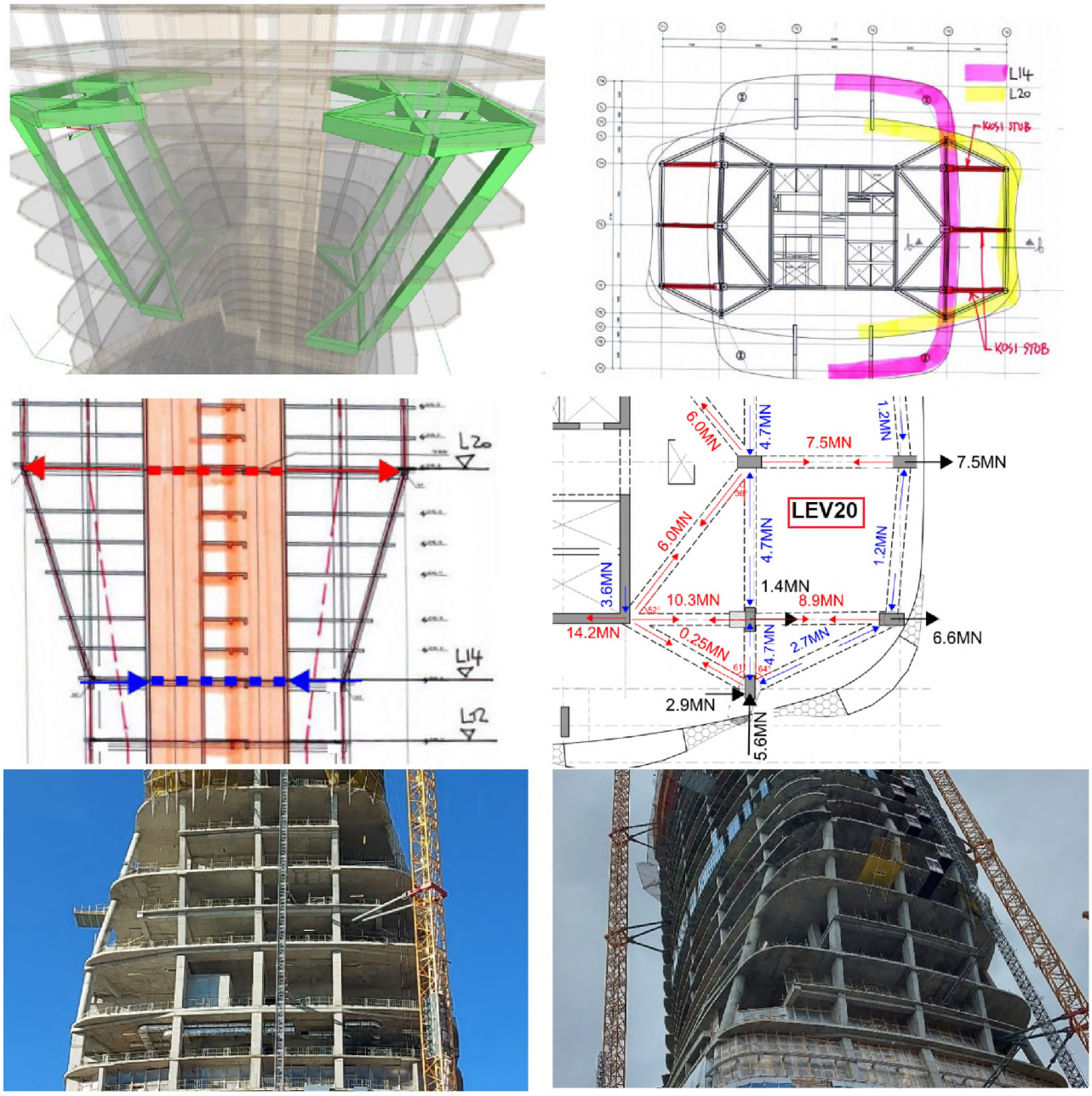

Figure 7. Transfer structure illustration of geometry and turn forces

SOM's idea behind such solution was to design embedded steel structure so it could sustain the weight of the upper structure, thus enabling the start of construction of floors above Level 20 as soon as the transfer steel structure alone would be erected, while the construction of RC parts of columns, beams and floors at transition levels would be done simultaneously with construction of upper floors.

4.3 The podium structure

Podium structure was designed as steel structure. It consists of roof trusses, two composite slabs (at Level 02 and MEP well) and steel columns. Lateral stability is provided by anchoring of roof structure to the concrete tower structure and by frame action of columns and horizontals, so the vertical bracings were omitted. Steel grade is S355.

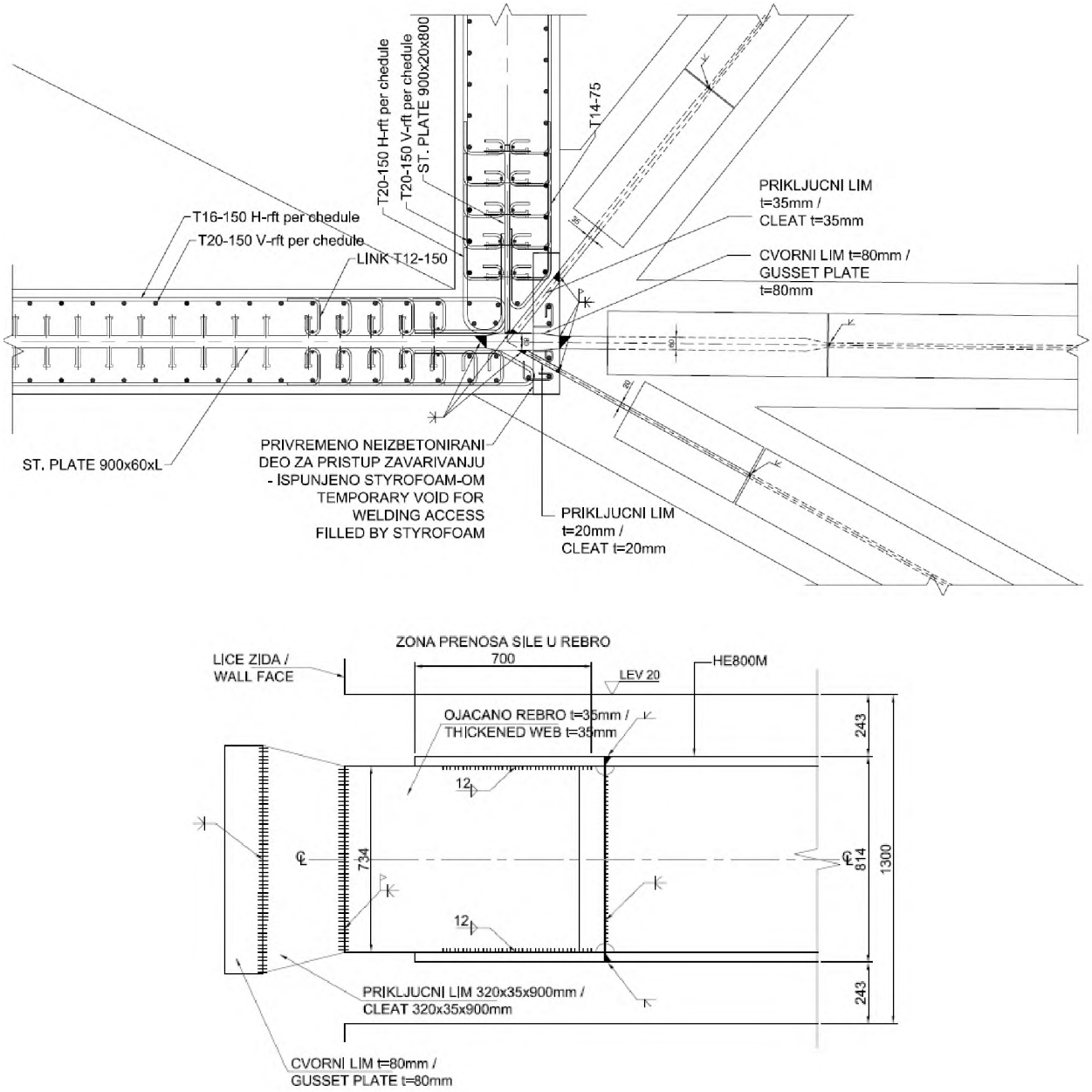

Figure 8. Transfer structure- joint detail at core corner – plan view (upper detail) and vertical section (lower detail)

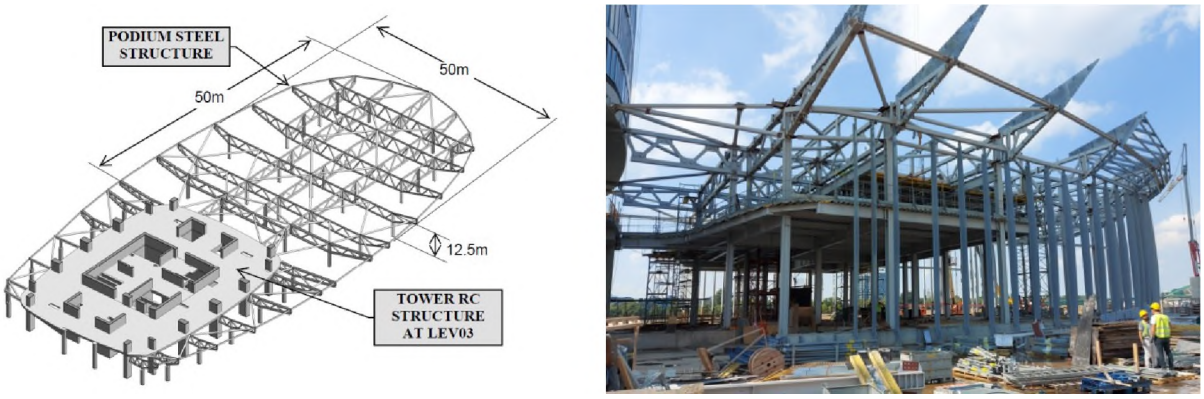

Figure 9. Podium Roof Steel Structure

4.4 The basement structure

Ground floor slab was adopted as two-way RC slab with RC beams in orthogonal directions. Typical beam size is 600 x 1000 mm, while the slab is 300 mm thick. It is heavily loaded by pavers and landscape build-ups but also with traffic load both in construction and service stage. Design concrete grade was C32/40.

Basement mezzanine slab is of mixed type. In the zone of the parking lot, it is 250 mm flat slab with 450 mm thick drop panels, whereas in the zone of the tower and around it is similar to ground floor slab – two-way slab with beams and same concrete grade.

Vertical loads are transferred by two storey RC columns 600 x 600 mm made of C40/50.

The perimeter of the structure is protected from soil and water pressures by RC wall. Depending on end conditions its thickness varies from 600 mm in case of two-way continuous member to 1300 mm in case of 10 m high cantilever in zone of ramps. Concrete grade was C40/50.

4.5 Foundations

Due to poor geotechnical properties of upper soil layers, adopted foundation solution is piled raft. The tower is founded on 62 bored piles with diameter of Ø1200 mm and length of 30.0-35.0 m. The reason for non-uniform length of tower piles was in non-uniform thickness of tower raft, which ranged from 2800 mm to 6800 mm in the zone of elevators shafts, while the toe level of tower piles was constant at +32MASL, or approximately 8-9 m deep into the reef organic limestone layer.

Basement structure is founded on 208 bored piles at 800 mm thick piled raft with diameter of Ø1000 mm and constant length of 25 m. Bases of piles are in Limestone/Marl for tower/basement piles, respectively.

Figure 10. Basement structure under Construction

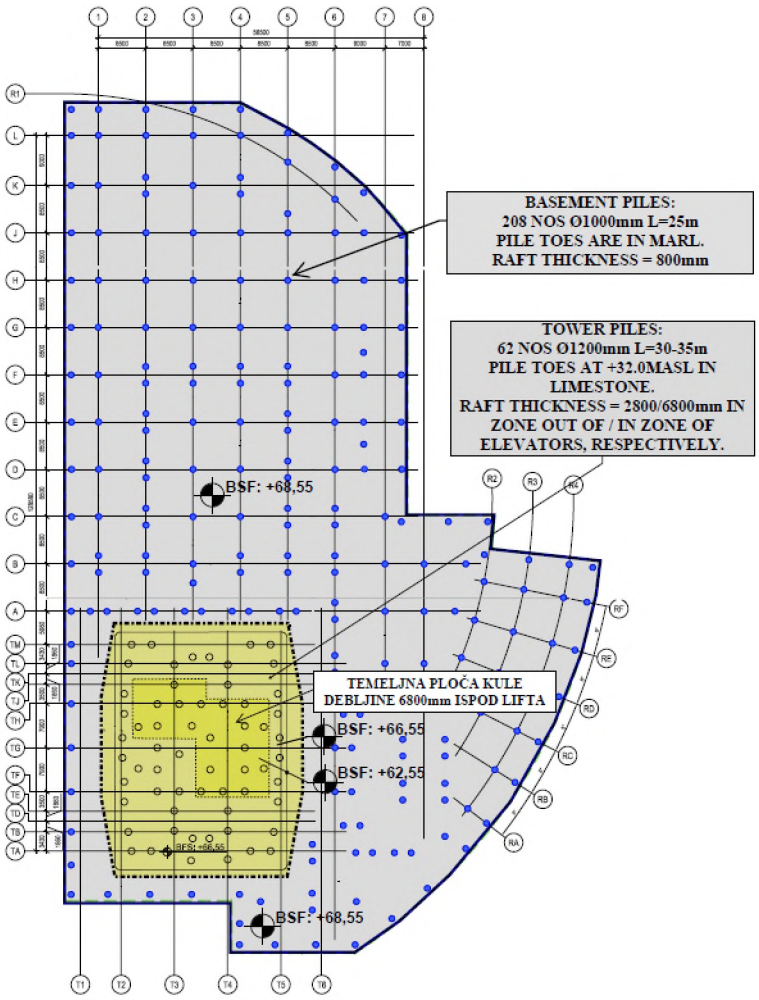

Figure 11. Piles layout

Axial capacities of piles are 23.6/5 MN, while maximum design forces reach 21/4.5 MN for Ø1200/Ø1000 mm piles, respectively.

Design water table at +74MASL resulted with uplift pressures in range of 75 kN/m^2 to 115 kN/m^2, under the Tower and 55 kN/m^2 to 78 kN/m^2 at Basement structure, where the piles acted as ties with tension capacities of 3 MN.

All actions in horizontal direction were assigned to piles alone. Capacity of tower piles is sufficient to resist the seismic force from the tower, while all together they take full seismic force from Tower, Podium and Basement. Due to eccentricity between the centres of mass and stiffness, raft tends to rotate in its plane, so the utmost rows of basement piles are subjected to horizontal forces to which they cannot respond elastically. However, these piles are not subjected to high axial loads - they are ductile and may respond elasto-plastically. Therefore, the adequacy of piles for horizontal actions was confirmed by iterative non-linear analysis. As an extreme case such analyses included the accidental case when top 5 m of soil strata is completely lost so the piles have no lateral support in this zone and the structure may be regarded as “pile dwelling house”.

5 Kula structure as per value engineering (VE) solution

While the construction of Kula was still in early stage, Contractor has decided to conduct a VE exercise. Basement levels were omitted as they were under construction at a time, while the following items were redesigned in tower structure:

- Embedded steel sections were removed from columns
- Composite transfer beams were redesigned to post-tensioned (PT) beams
- Slabs were redesigned from RC to PT slabs
- Embedded steel plates were removed from core walls and their reinforcement was optimized

5.1 VE of transfer structure

The main goal set by Contractor was to eliminate the embedded steel sections from beams and columns as well as the steel plates from core walls in order to simplify and speed up construction process. However, it was in the early stage of VE exercise when it was noticed that the differential settlements doubled in comparison with those in the original design. Comparative analysis has shown that this was originated by the removal of embedded steel sections from columns, which were subjected to high axial pressures as they were designed as secondary structural members and

accordingly had not limited normalized axial force (axial load ratio). Same parameter was limited to 0.4 in the core walls, as per Eurocode 1998 [6] requirement for DCM, to allow for their ductility. Therefore, it was the combination of big difference in core walls - columns stresses and reduction in stiffness of columns that made differential settlements to become an important item in VE design. Such finding was surprising at the first glance, since the differential settlements are usually insignificant in 40 storey buildings. However, having in mind previously mentioned branching of columns and the fact that six pairs of columns take the loads from 28 floors (Level 42 to 15) and transfer them to six single columns below Level 15, Kula may be regarded as 70 storeys building in terms of the axial loads at the base of six main columns.

In order to mitigate the effects of differential settlements BG&E and DNEC decided to reduce the thickness of transfer beams, especially in bays adjacent to the core, so the beams of 600 x 1300 mm were replaced by 1600 x 700 mm and 2000 x 700 mm in outer spans, and 1600 x 450 mm and 2000 x 450 mm in spans adjacent to core. Furthermore, layout of transfer beams was significantly rearranged on Level 020, as shown in Figure 12. Three beams highlighted in yellow in the Figure 12 were adopted as PT beams while other beams in plan were designed as RC beams.

Each of Level 20 PT beams comprised two multistrand tendons with 22 strands, Ø15.2 mm, made of low relaxation steel grade 1860 MPa. Ducts were grouted after the stressing so the tendons were bonded to beams.

Due to geometrical issues with beams at slab corners, the force in tendons was not fully developed at the point of the action of kink force. Accordingly, U-shaped reinforcement bars had been designed in quantity sufficient to fully cater the kink force up to a point where tendons would overtake it.

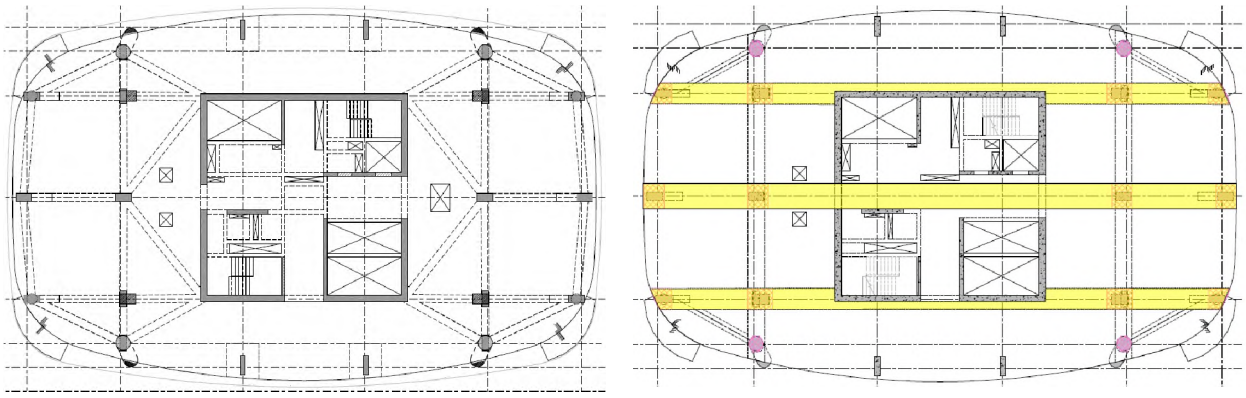

Figure 12. GA plan on Level 20 – from original design (left) and VE design (right)

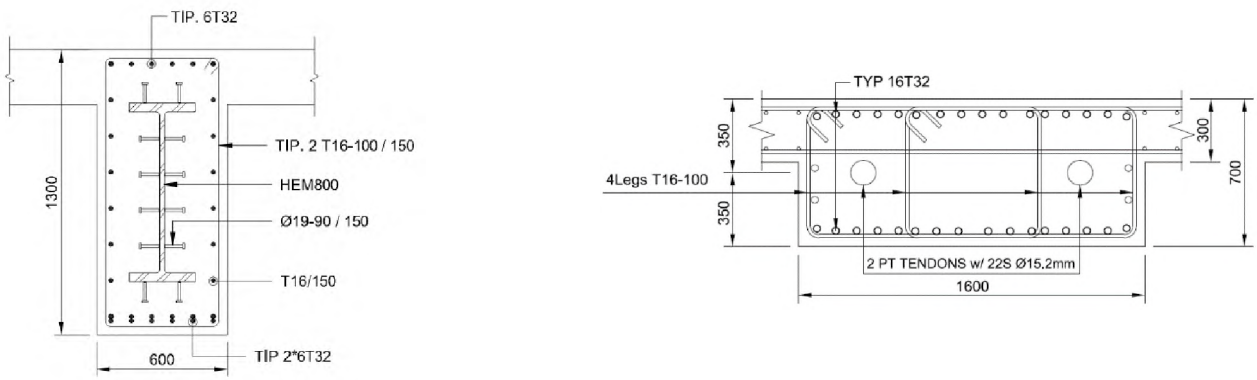

Figure 13. Transfer beams on Level 20 – cross sections from original design (left) and VE design (right)

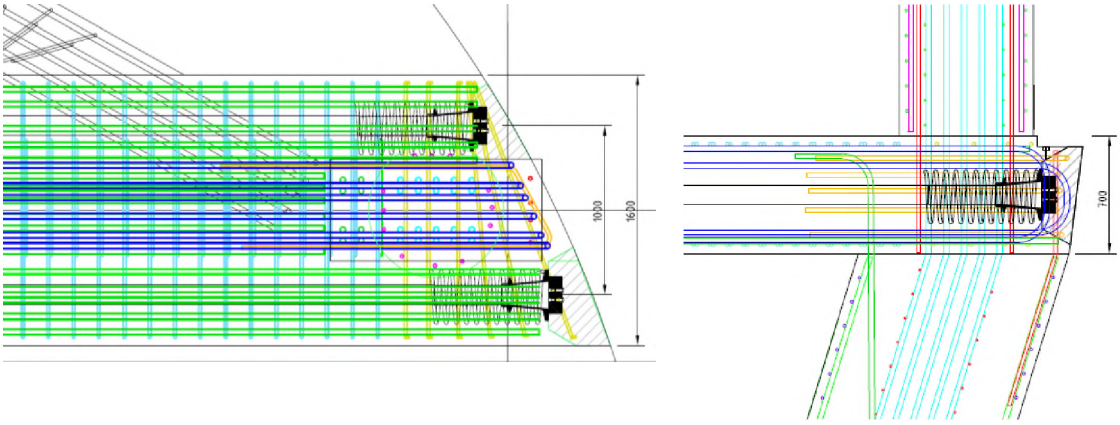

Figure 14. Transfer structure Level 20 – Node at façade in plan (left) and vertical section (right)

5.2 VE of slabs

Basic thickness of slabs was kept the same as in original design (200 mm) but perimeter beams were removed both in hotel and residential levels, and in residential levels replaced by 400 mm drop panels. However, the biggest change was conversion from RC to PT slab. High strength strands, Ø12.7 mm, made of low relaxation steel grade 1860 MPa were placed in flat ducts with 3 to 5 slots to form the flat tendons suitable for slabs. Ducts were grouted after the stressing so the tendons were bonded to slab.

Design conducted as per Eurocode 2 [7] and Technical Report 43 [8] resulted in non-conventional layout as there were no pronounced distributed versus banded arrangement in two orthogonal directions. Instead, tendons in both directions appear more as distributed with spacing in range of 6-10 times slab thickness.

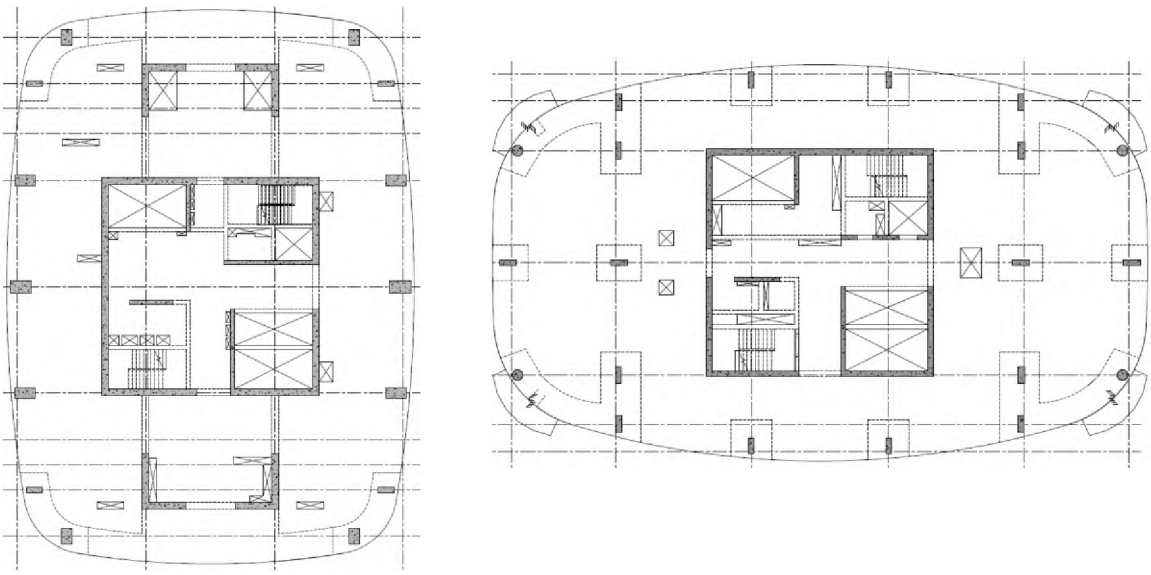

Figure 15. GA plans of typical Hotel and Residential level as per VE design

Figure 16. Reinforcement and PT works in progress

Previously mentioned issue of differential settlements greatly affected the design of PT slabs, too - mostly in the aspect of punching but also crack control. Local FEM models for slab analysis simulated the case of columns settlement – as illustrated in Figure 17. Typically, the East and West edges of slabs above Level 30 were lowered approximately by 40 mm with respect to the core. Bending moments originated by imposed displacements were combined with those from other load cases and used for punching checks. Punching resistance was provided by the increase of longitudinal reinforcement in tension zone and by shear stud reinforcement.

5.4 VE of core walls

Apart from embedded steel plates at transfer levels that were removed by VE of transfer structure, core walls of the original design comprised another type of embedded steel and that was steel plate in link beams. Namely, each link beam in main core, satellite cores, but also coupling beams between the cores comprised embedded steel plate. Thickness of the plates was typically 40 and 50 mm, while the depth ranged from 750 mm in case of typical storey, up to 8500 mm in case of link beam above the big opening in East core wall at Level 2.

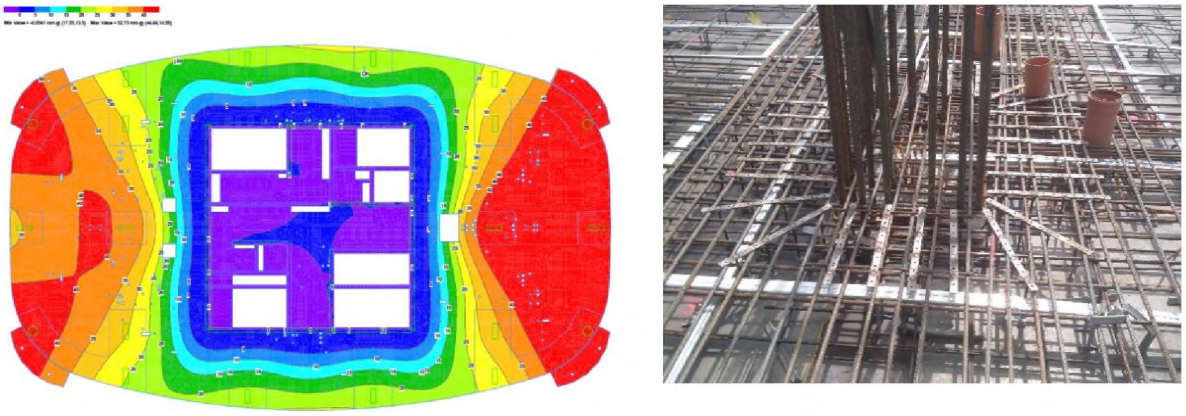

Figure 17. Simulation of differential settlements in FEM model (left) and Stud shear on rails installed in slab (right)

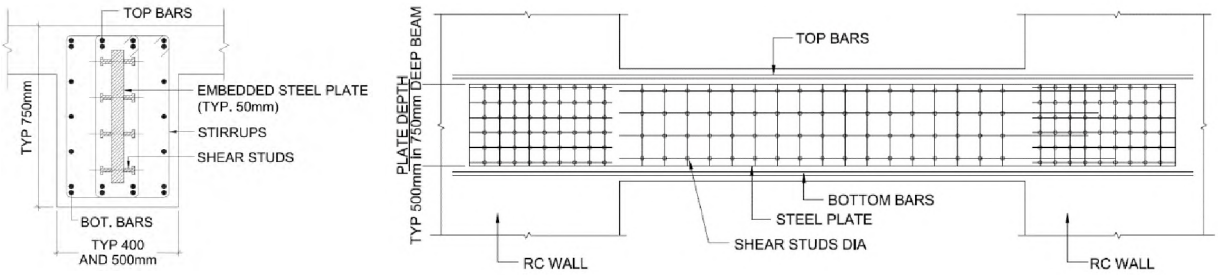

Figure 18. Link beam solution from original design – Composite link beam

During the VE stage, not only that link beams were converted from composite to RC beams, but they were also required to include web openings to allow for final MEP routing. Solution was to minimize the quantity of flexural reinforcement and accordingly shear demand obtained by capacity design and to detail shear reinforcement so it provided maximum possible shear capacity and ductility to the beam. Shear reinforcement was adopted as a combination of closed hoops (stirrups) and diagonal rebar which formed diamond shaped framing around the openings. Cross ties were adopted to confine the trimmer bars placed above and below the openings.

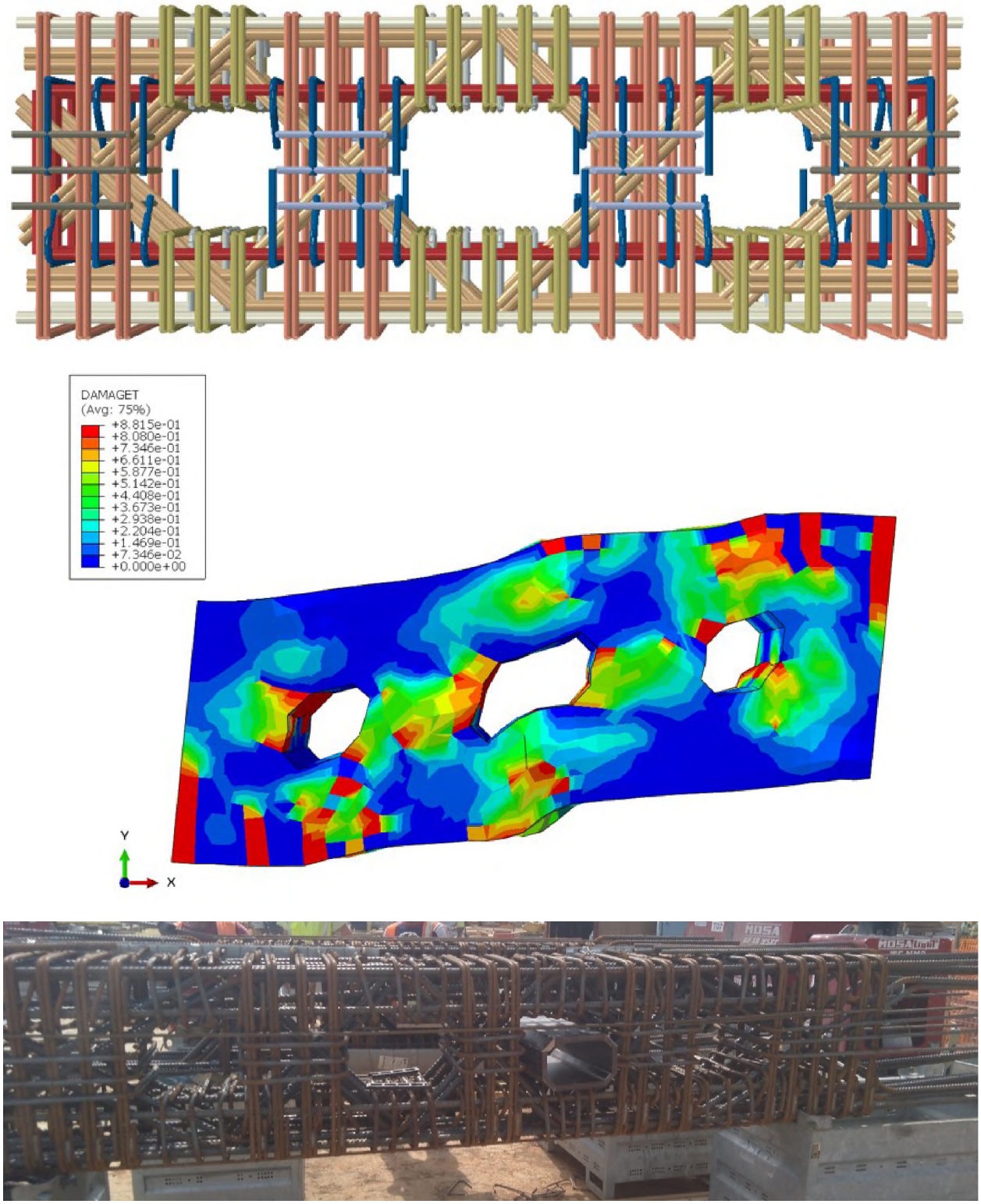

Figure 19. Model of Link Beam in Abaqus, Version 2017 (above), deformed shape with tensile damage distribution at 9 mm of displacement and 0.0075 rad of rotation (mid) and prefabrication of link beam rebar cage on ground for later erection and installation into core wall (below)

The checking of achieved level of capacity and ductility was done by non-linear analysis executed in Abaqus software by Faculty of Civil Engineering at Belgrade University [9]. Micro modelling approach was implemented, so each reinforcement bar and stirrup were modelled with its own shape. Concrete part was modelled, too. Nonlinear stress-strain and stress-displacement relations were defined for the behaviour of concrete, while bilinear plastic behaviour was applied for reinforcement material definition. Rotations and displacements at the left and right ends of the beam were assigned as loading that simulates the earthquake loading transferred from the walls to the coupling beam. Analysis has shown that the plastic behaviour occurred at the ends of the beam and it remained there. Brittle damage of concrete was avoided, while reinforcement activation led to the ductile behaviour of specimen, so it was concluded that the adopted link beam solution was adequate.

5.5 Overview of ve process

Although the significant saving was achieved by removal of embedded steel sections from transfer structural members, the overall savings was somewhat reduced due to the increase of reinforcement added for punching resistance and to control the cracking. However, the VE process may be regarded as successful as it resulted in simpler, thus faster construction of transfer levels and core walls and also faster construction of superstructure slabs, which Contractor achieved to cast in 2-3 days cycles at typical levels.

6 About the 2nd part of the article

While this part of the article addressed the specifics of design process, the second part will present the key items related to Kula construction including: enabling works, the execution and testing of piles, concreting of raft (part below the tower – approximately 4750 m^3 of concrete was cast in one turn), works on transfer structure, etc.

References

[1] Geotech report – Book 1 and Book 2 (Laboratory testing) – CIP Institute Belgrade, December 2015.
[2] Geotech report – amendment – CIP Institute Belgrade, April 2016.
[3] Slavica Radovanović, Seismic study - Report on results of defining specific elastic response spectrum of local soil at the Belgrade Waterfront location. Plot 19.1, February 2016.
[4] Wind tunnel testing report, RWDI, Canada, April 2016.
[5] ASCE Standard 7-10, Minimum design loads for buildings and other structures, ASCE/SEI, 2010
[6] Eurocode 8 - EN 1998-1-1:2004 and SRPS EN 1998-1/NA, December 2018
[7] Eurocode 2 – EN 1992-1-1:2004 and SRPS EN 1992-1-1/NA, November 2015
[8] Technical report 43 - Post tensioned concrete floors – Concrete Society, Second Edition, 2005.
[9] Marko Marinković, Non-linear analysis of RC coupling beam Rev00. 01.06.2020.