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# The Vienna Donau City Tower – 2000mm Flat Slabs as Outrigger Structure for Unique Landmark Building



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## Abstract

*The Donau City Tower I, with an architectural height of 220m, has become Austria's tallest skyscraper, and is the new landmark building in Vienna's skyline. The slenderness of the tower is extraordinary. Combined with a folded façade, Dominique Perrault Architecture created a unique shape. The occupation of the DCT 1 is divided into three parts: In the lower third of the tower there is a hotel area, in the middle is an office area, and in the top third of the tower are the residential floors. A reinforced concrete structure was used for the DCT 1. Flat slabs span between the core and the columns. Lateral stiffness to resist wind and seismic loads was achieved by two elements: a strong concrete core with walls of up to 1m thickness and outrigger slabs of 2m thickness above the MEP plant floor levels which activate the columns. For load takedown into the ground slurry walls, arranged in squares, were applied below the foundation slab. Strong comfort criteria were required by the client, thus a tuned mass damper was applied in order to reduce wind accelerations. Thanks to the close cooperation of all parties, the building could be successfully completed and was among the finalists of the CTBUH Award: Best Tall Building Europe 2014.*

**Keywords: Mixed-use, Outriggers, Slenderness, Structure, Tuned Mass Damper**

The Donau City Tower I, with 220m architectural height Austria's tallest skyscraper, is the new landmark building in Vienna's skyline northeast of the river Danube. The project was challenging considering the structural aspect ratio. The slenderness of the tower is extraordinary, with a relation of structural width to height of nearly one to ten. Combined with a folded façade Dominique Perrault Architecture (DPA) created a very unique shape.

The Tower 1 is the first part of a building complex that has been developed from the idea of "two pieces of a gigantic monolith that seem to have split into two unequal halves, which then open to create an arch with undulating and shimmering façades that bring the newly created public space to life in the void created there" (DPA). The construction of the second tower DCT 2 is currently postponed.

The occupation of the DCT 1 is divided into three parts: In the lower third of the tower there is hotel, in the middle part office use, and in the top third of the tower there are residential floors.

The structural design of the DC Tower was done by a corporation of the engineers of Bollinger Grohmann Schneider (BGS), with Gmeiner Haferl (GH), both located in Vienna. Due to the complex structural challenges, it was agreed to perform all calculations separately in both offices by independent analysis and different software (BGS: Etabs, GH: SCIA). Thus it was possible to check the structural systems and the analysis results thoroughly and independently from each other.

The design of the DCT 1 follows the Austrian Code B4700, which complies with Eurocode 2 for concrete reinforced structures [5]. The structural challenges, which mainly result from the slenderness of the building, were to provide a structural system which was feasible to resist gravity loads, wind and seismic loads as well as to limit deformations and accelerations due to wind loads.



Figure 1. View from the Southwest River Side  
(Source: Michael Nagl DPA Adagp)

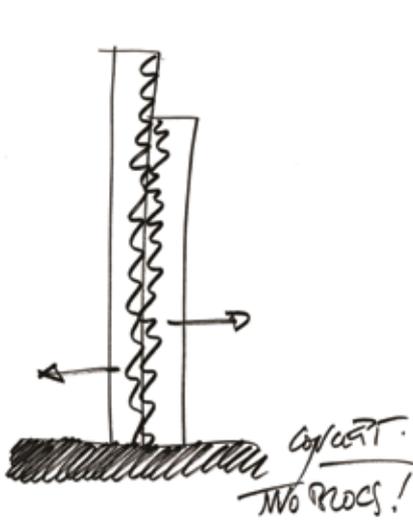


Figure 2. Design of DCT 1 and 2 (Source: DPA)

### Vertical Load

For vertical load takedown, flat slabs span between the core walls and the columns in the façade transfer the loads into the foundation.

### Slabs

The reinforced concrete slabs are supported by the core and the columns. The typical span is ~6.5m on two sides of the façade and ~8.50m rectangular to the narrow sides of the façade. The thickness of the typical slab is 250mm, which is quite slender for the maximum span of 8.5m, but it has been shown to be appropriate when the bending-resistant supports of columns and the core walls were considered. Double head studs had to be applied in some areas in order to raise the shear capacity. All wide spanning slabs have a precamber of 20mm. Column shortening had to be considered by support deformations as well as additional shear loads from wind for the design of the slabs.

The design of the slabs as flat slabs without any beams or thicker parts enables a very efficient and flexible architectural design. MEP ducts for heating and cooling are integrated in the slabs. The floor to floor height is ~3.5m, the clear height is 3m. The minimization of the story heights allows a number of 60 floors within the limited height of the building.

Two different systems were assumed regarding the contribution of the 250mm slabs on the lateral system. Both systems are limit states and consider that the real stiffness of the slab structure is not precisely predictable. The first system assumed that the slabs do not contribute to the lateral system, which was implemented by non-bending-resistant joints at the core walls and the columns. This is the governing system for the outrigger slab and the core wall design. The

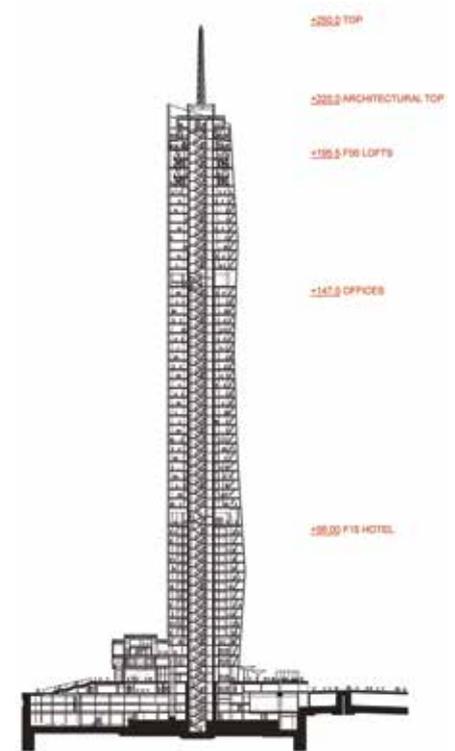
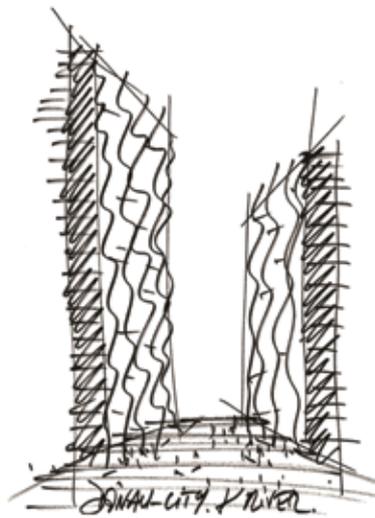


Figure 3. Section of DCT 1 (Source: DPA)

other assumption was that the slabs stiffness will be around 30% of the non-cracked section in ultimate limit state. This system was applied for the design of the slab, slab to core and slab to column connection, regarding bending moments, shear capacity, and axial forces on the columns. It was also considered for serviceability limit state design.

### Cantilever at the Folded Façade

The folded façade is one of the most iconic architectural elements of the Donau City Tower. The south east façade is split into ten undulating vertical ribbons which have different inclinations. As a result its distance to the column axis is changing from floor to floor and from row to row. The cantilever of the floor slabs is different in every floor (see Figure 7). To enable a cantilever of up to 6.2m the slabs are supported on small inclined columns. The angle and the position of these additional columns ensure that the resulting cantilever is never more than two meters. The inclined columns had to fulfill fire resistance requirements as well as the requirements of the architects regarding the surface. Thus prefabricated concrete columns were applied for these columns.

### Columns

In the variation studies on the lateral system it was shown that the column stiffness was an important factor which influences the overall lateral stiffness of the global structure. Nevertheless the first design approach for the columns was to minimize the size of the columns as much as possible, which is a common demand from the client. Thus the feasibility of columns, which should be composed of massive steel plates within a thin concrete shell, was analyzed.

The load capacity of these columns was sufficient for the vertical and horizontal load

takedown from the first view. But it had to be considered that there would be a different time dependent vertical deformation of the composite columns compared to the concrete core walls. These effects mainly result from creep and shrinkage of the concrete elements and they were described for structural systems of different materials in [1] and [4]. The basis of the analysis of these time- and load-dependent processes on the DC Tower was the German Concrete Design Code DIN 1045-1 [6], [7], which complies with the current Eurocode 2 regarding this. The effect of creep and shrinkage was applied accordingly for the construction stage of each story and for the columns and the core walls.

The construction stage loads were applied as temperature loads for the structural analysis in the etabs stability model. The effects of creep and shrinkage in the core walls were significant. Due to core wall shortening, parts of the loads in the core walls were planted via the outrigger structure on the columns, which meant that there would be a considerable load increase in the outrigger system and in the columns.

The following distribution of axial forces in the columns resulted:

- ~45% due to dead load and live load;
- ~25% due to wind loads;
- Nearly 30% due to creep and shrinkage of the core.

Due to the loads from creep and shrinkage,

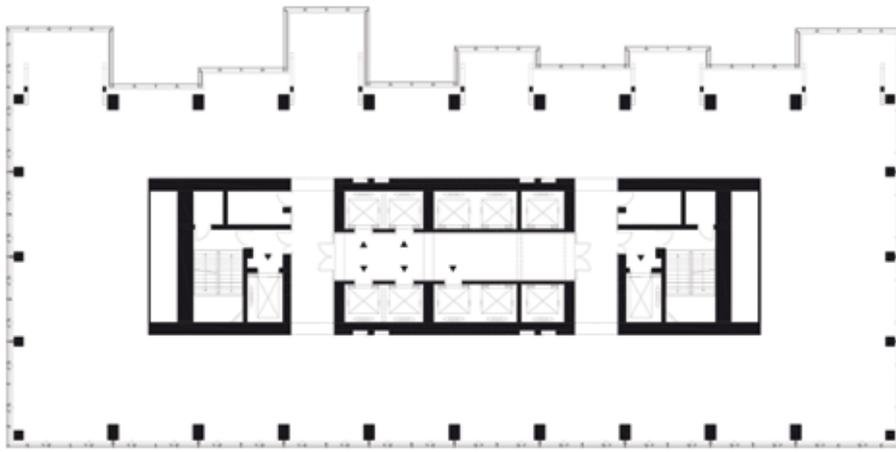


Figure 4. Typical Office Floor Slab (Source: DPA)



Figure 5. Slabs at the Folded Façade (Source: Bollinger + Grohmann)

the steel amount in the columns had to increase significantly. This again requires higher load capacity and stiffness in the columns, which again caused increased loads, and so on.

The differential stress distribution from gravity loads was not the governing factor for differential shortening between the columns and the core walls. In typical concrete high-rise buildings the column shortening is bigger than the core wall shortening. For the DC Tower, two important factors had to be considered: Composite columns with a high proportion of steel would have a different time dependent shortening compared to the concrete columns. During the construction process the core walls, which obviously have less stress due to gravity loads than columns, will shorten less than the columns. But due to the shrinking of concrete, this effect will change. The core wall shortening occurs over a long period and due to the outriggers' stiffness core loads will be transferred from the core to the columns. The following options were discussed to solve the problem:

- Design the core walls with structural steel. This option is not very common in Austria. Additionally, due to highly different costs of structural and rebar steel, this option was not sufficiently competitive;
- Design the columns and core as reinforced concrete columns in high-strength RC with similar creep and shrinkage behavior.

In the end the second option was agreed: High-strength (C70/85) RC columns and core walls were used. The column size was increased in the hotel floors to 1,200 x 1,200mm and in the office floors to 700 x 1,000mm. The RC column size in the residential floors is 600 x 600mm.

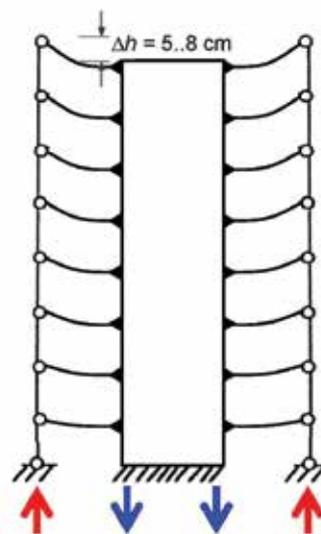
#### Column Axis Set-off

The geometry of the folded façade required a column axis set-off between 16th and 18th floor. Thus the columns in story 16 – 17 are inclined, which required a horizontal load transfer from the top of these columns to the bottom floor via the concrete core. The horizontal pressure forces on top of the inclined columns were transferred via the RC concrete slabs' bearing capacity. At the bottom of the inclined columns the tension forces had to be transferred through the core to the opposite edge of the slab and from there into the core walls. This horizontal load transfer was provided by a combination of prestressed and non-prestressed reinforcement.

#### Gesamtsystem Stauchungen $t = \infty$



Figure 6. Load Transfer from the Core to the Columns (Source: Bollinger + Grohmann)



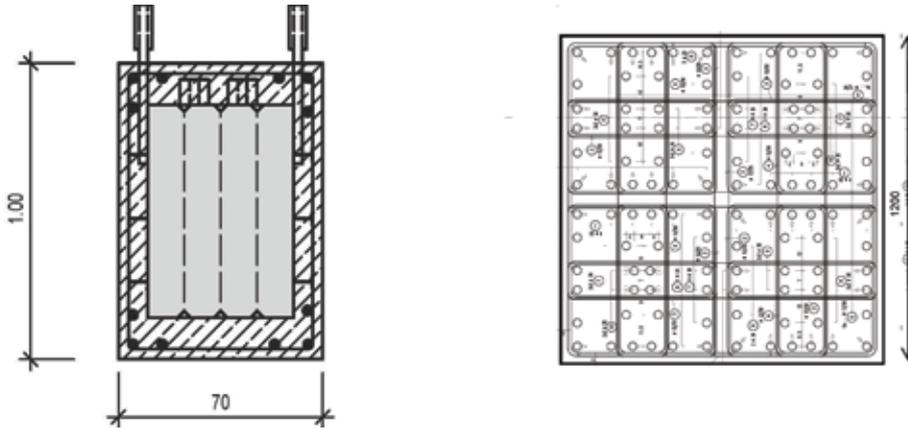


Figure 7. Column Design Study (Source: Bollinger + Grohmann)

### Lateral Stiffness

Maximum gust wind speed of 130 miles/hour and seismic loads according to Mercalli scale VII had to be considered for the DC tower. Due to the shape and the slenderness of the building it was important that a feasible lateral system could be developed in the early planning phases. Thus the main focus in the beginning of the project was on the feasibility studies of different lateral systems.

### Feasibility Studies

Feasibility studies were performed to find out which structural system performed best regarding horizontal loads. The following systems were analyzed:

- A core wall system;
- An outrigger – based system with the core walls and two outriggers in the MEP plant floors;
- A megastructure truss system at both short façade faces additional to the core walls;
- Additional shear walls in the hotel floors;
- Combinations of each variation.



Figure 8. Inclined Columns at the Folded Façade (Source: Bollinger + Grohmann)

All systems were checked regarding p-delta-effects, frequency and wind deformation. The variations also included the influence of increased column stiffness.

The design criteria were:

- The wind deformation limit of  $h/500$ ;
- A maximum interstory drift of  $h/400$  due to wind;
- A maximum interstory drift of  $h/100$  due to seismic loads;
- A natural frequency of a minimum of  $\sim 0.15$  Hz. A system with a considerably lower value was predicted to be prone to dynamic wind effects like lateral sway rectangular to the wind direction.

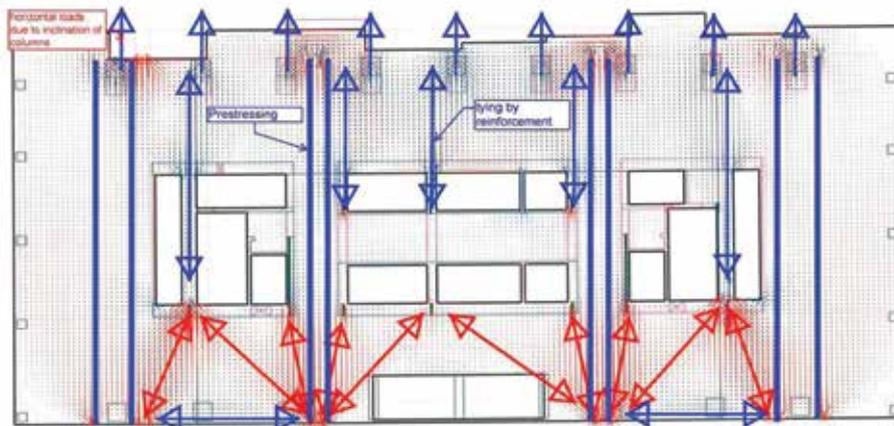


Figure 9. Truss System of the Slab Above 15th Floor (Source: Bollinger + Grohmann)

	Core	Core and outrigger	Core, outrigger and megastructure	Lateral stability via core and outrigger, double column cross-section (2b)	Lateral stability via core and outrigger, double column cross-section, reinforced outrigger (2c)	Lateral stability via core and outrigger, double column cross-section, replacement of the undermost outrigger with slanting struts (2c)
Calculated frequency [Hertz]	0,08 Hz	0,15 Hz	0,18 Hz	0,18 Hz	0,17 Hz	0,17 Hz
Stiffness [%]	17%	67%	100%	62%	67%	67%

Figure 10. Feasibility/Variation Studies Lateral System (Source: Bollinger + Grohmann)

The results from these feasibility studies were that the outrigger system is highly efficient compared to all other systems:

- The core itself was too weak to limit wind deformation to less than  $h/500$ ;
- The stiffness of the core and outrigger system was in a range that seemed to be capable to resist wind and seismic loads within the given criteria;
- The effect of an additional megastructure together with core and outrigger was less than expected. The amount of additional steel required for

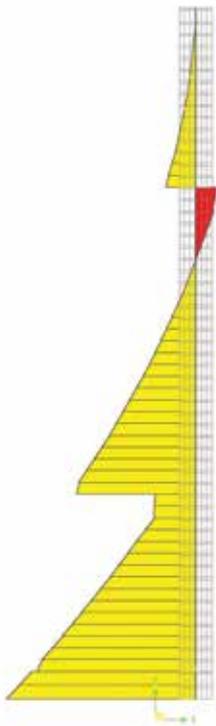


Figure 12. Core Bending Moment due to Wind Loads (Source: Bollinger + Grohmann)

the megastructure was disproportional to the beneficial impact.

So the outcome from the feasibility studies was to follow the system with the outriggers. The column size should be limited to the size which was required for ultimate limit state design. Dynamic effects should be limited by a Tuned Mass Damper system if required, which was analyzed by the wind engineer in the following phase.

### Wind Loads

The wind loads were applied according to Eurocode 1-1-4 for the early design phases. Wind studies had been performed during schematic design.

### Wind Tunnel Test

- The wind tunnel tests were performed by Wacker Ingenieure, Birkenfeld, Germany. The wind loads at a height of 220m are based on a gust wind speed of ~130 mph and a return period of 50 years. The design

wind loads considered all dynamic influences;

- For comfort criteria, the design wind loads were reduced by a reduction factor of 0.81 to a return period of 10 years. The limit of acceleration which was agreed was 1,5 % g, which was exceeded slightly in the analysis. Thus a tuned mass damper was considered to provide the safety regarding wind comfort.

### Outrigger Structure

The outriggers were placed in the MEP plant floor areas, which are located between the residential and office floors (40th - 41st floor) and between the office and hotel floors (16th - 17st floor). Thus the outriggers did not interfere with the rentable space. From a structural point of view the location of the outriggers at one and two third of the height was very efficient. The first outrigger from the top did not only reduce the wind bending moment of the core, but even reversed the sign of the moment. The second outrigger reduced the bending moment about 70%.

The first approach for the outriggers was to apply structural steel diagonal members for the load transfer from the core into the columns. Together with a stiff belt in the columns' axis this method was the most efficient for the global structural stability system. From a structural point of view there were two options to realize the structural steel outriggers. One was to apply structural steel plates with stiffeners as steel shear walls. The other was the diagonal frame element. Both systems had implications on the MEP device location and duct planning in the technical floors. Additionally it had to be considered that in Austria the construction costs of structural steel were in a range of factor 2-3 compared to reinforcement steel. Thus another solution should to be found which fulfilled the requirements regarding the global stability system.

Further variation studies were performed and one idea was to check the minimum thickness of a flat slab, providing the same stiffness as the structural steel outrigger system. The MEP had to check the feasibility of the remaining space in the plant floors and we had to check if this structural system could really work. The most important effects of two massive outrigger slabs were:

- Additional mass of 4 tons/m<sup>2</sup> from each outrigger slab;
- Minor natural frequency;
- Increased seismic loads due to higher

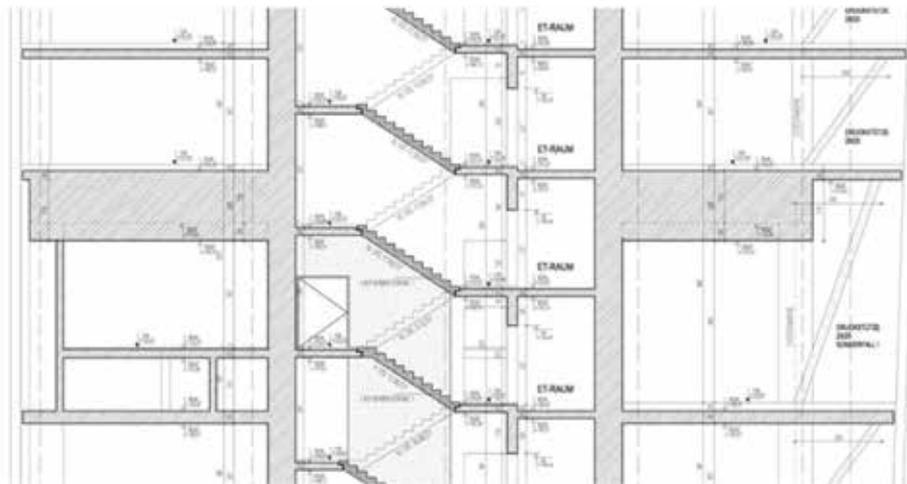


Figure 13. Section: Outrigger Slab (Source: Gmeiner Haferl)

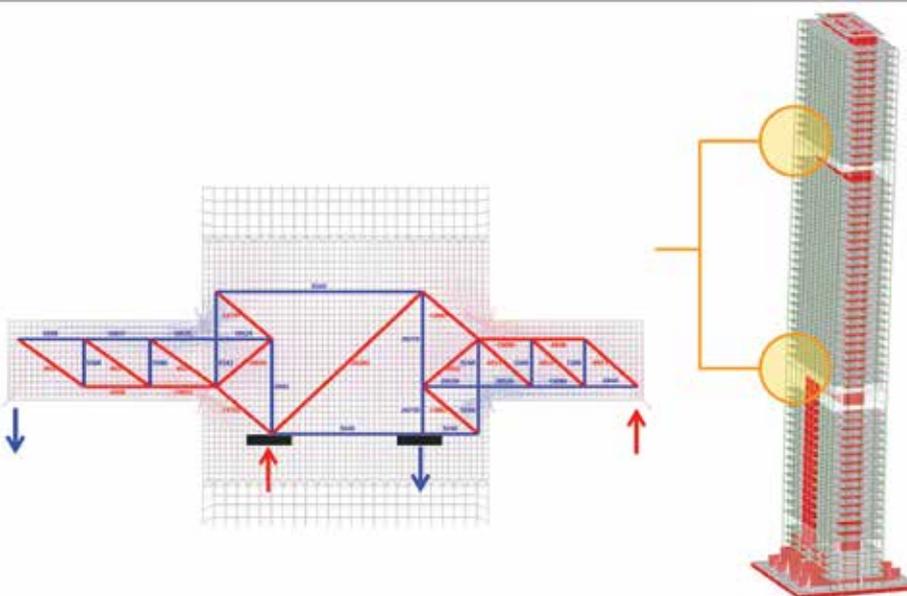


Figure 14. Idealized Truss System of Core Walls and Outrigger Slab (Source: Bollinger + Grohmann)

masses in the strong axis of the structural stability system;

- Increased wind loads rectangular to the strong axis of the structural stability system due to dynamic effects.

The load transfer from the columns via the concrete outrigger slabs into the core was analyzed with the global stability FEM-systems. For outrigger slab design, all results were checked according to an idealized truss system for verifying the load paths from columns to core walls and for shear forces design.

### Construction Phase of the Outrigger Slabs

The outrigger slabs were poured in two construction phases: In the first phase the lower part of 40cm of the slab was reinforced and poured. Then the rest of the slab was completed with longitudinal bars and shear reinforcement. The concrete gravity loads were distributed on the next four slabs below.

### Seismic Loads

A response spectra analysis was performed as the basis for the design of the seismic elements. The ductility factor was applied to 1.5 according to the Austrian code B4015. For the most structural elements, the wind loads were the governing lateral loads. For some of the spandrels which connect the core walls the seismic load combination was governing.

### Tuned Mass Damper

The 300 ton mass pendulum system was applied in order to fulfill the serviceability requirements regarding the maximum acceleration in the residential floors. It is located above the elevator shaft on top of the building. For the design of the TMD, it was essential to consider that real buildings have different Eigenfrequencies compared to the analyzed Eigenfrequency. Thus it was necessary to provide a structure which could react on this different behavior. The Eigenfrequency which was measured for the DC Tower was around 0.19Hz, which means that more or less all the concrete elements of the lateral system were still in an uncracked condition, which would change under strong wind impact. The idea which was realized was to apply horizontal supports for the cables of the mass, which are vertically moveable. By this way the TMD may be adapted to different structural behavior of the tower by changing the structural length of the suspension cables.

### Foundation

The DC tower is located directly nearby existing buildings and an existing

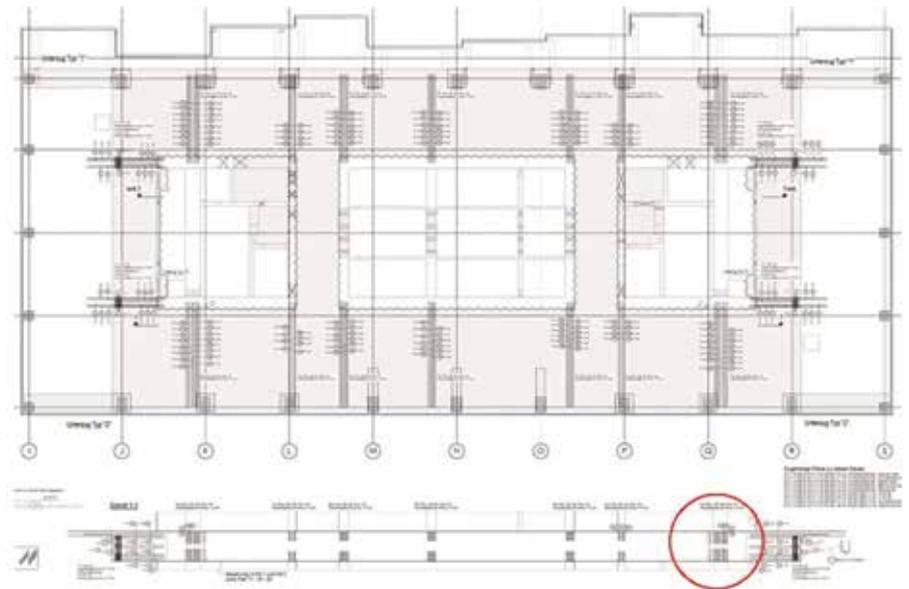


Figure 15. Rebar Design: Rebar Connection of Core Walls and Outrigger Slab (Source: Bollinger + Grohmann)



Figure 16. 3D-Rebar Design: Core Walls and Connection to Outrigger Slab (Source: Bollinger + Grohmann)

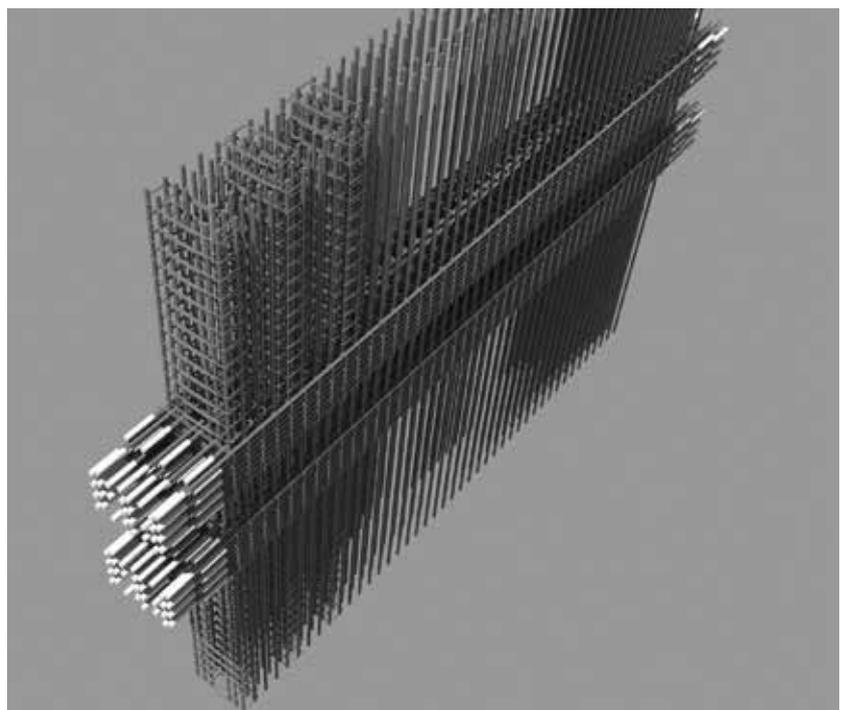


Figure 17. Rebar Connection Core Walls to Outrigger Slab (Source: Bollinger + Grohmann)

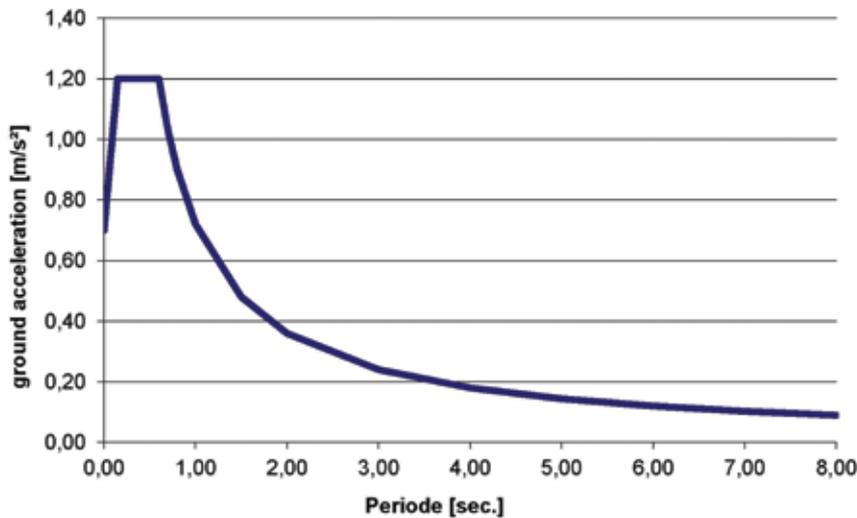


Figure 18. Response Spectrum for DC Tower Seismic Design (Source: Bollinger + Grohmann)



Figure 19. Tuned Mass Damper of the DCT 1 (Source: Bollinger + Grohmann)

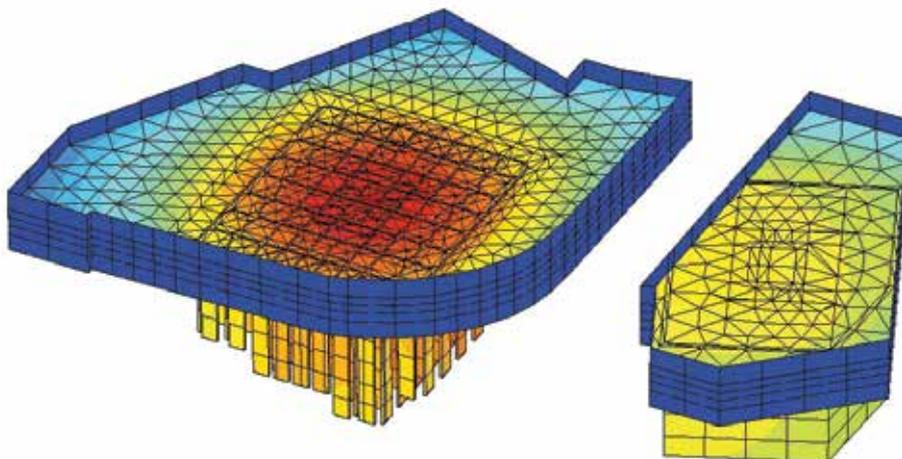


Figure 20. Distribution of Settlement (Source: Graz University of Technology)

motorway. It was very important to limit the deformations to a feasible value. The foundation system is a 4,000mm thick raft foundation combined with diaphragm walls with a depth of 20 – 30m. It is common practice in Vienna to apply the system of the diaphragm walls for the foundation if there are strong requirements regarding the settlement in combination with high gravity loads.

After the excavation the soil is stabilized by bentonite slurry, which is feasible in clay layers with low permeability. In the next step the reinforcing cage is placed and the diaphragm walls are poured. For the DCT foundation system, the diaphragm walls were applied instead of piles. Due to the large section these walls provide much more stiffness than the piles and they are easier to construct even in very deep depth.

For the DC Tower project, the system was not considered as a pure deep foundation only. The stiffness of the raft and the building structure were considered and the soil underneath the raft as well. The diaphragm walls were applied as single springs in the structural models according to the results from the settlement analysis, the soil stiffness was applied as area springs.

The calculations of the settlements which were done by the Computational Geotechnics Group, Graz University of Technology, Austria, considered the loads of the DC Tower 1 with and without the loads of the DC Tower 2. The approach was similar to a combined pile-raft foundation analysis: After the first results from the settlement calculation were given to the structural engineers the diaphragm wall and soil stiffness were applied in the structural system. Then the results were compared and after two iterations of calculations done by the geotechnical and structural engineer the results complied.

### **Construction**

The construction works of the pit and the deep foundation began in summer 2010. The foundation works on the raft started in autumn 2010 and topping out was in October 2012, which was primarily foreseen in May 2012.

### **Settlement**

The settlement was predicted to ~80mm maximum in the core area of the DCT 1 and 25mm at the edges of the foundation pit. Last monitoring results confirmed that real settlements were in a range of 80% (core wall area - 90% (edge of construction pit) of the predicted settlement.

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