DIPLOMARBEIT

MASTER'S THESIS

Prototype for Wind Turbine Towers out of Double Walls

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under the supervision of

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Kurzfassung

Die anthropogenen Treibhausgasemissionen müssen reduziert werden und deshalb wird die Gewinnung von erneuerbarer Energie staatlich subventioniert. Dies führte zu der raschen Entwicklung der neuesten Windturbinen und deren Turmkonstruktionen, welche eine kostengünstige und zuverlässige Quelle für erneuerbare Energie darstellen. Der neueste "onshore" Trend sind Hybridtürme mit einem Abschnitt aus Betonfertigteilen. Diese Entwicklung geht auf die Nachteile von reinen Stahl- und monolitischen Betontürmen ein und liefert derzeit die günstigste Bauweise für hohe Türmen. Die verwendeten Betonfertigteile ermöglichen einen sehr raschen Turmaufbau und deren Ausgangsstoff (Beton) ist sehr günstig im Vergleich zum Stahl. Jedoch ist die Geometrie der vollwandigen Betonelemente wegen dem Transport limitiert und die Tragfähigkeit des Turms wird über eine sehr hohe Vorspannung gewährleistet.

Eine neue Baumethode zur Errichtung von Turmbauwerken aus Doppelwänden wird derzeit an der TU Wien entwickelt. Das Ziel dieser Innovation ist es, die günstigen Eigenschaften von Fertigteilen und Ortbeton miteinander zu verbinden. Die Doppelwände werden zu polygonalen Hohlringsegmenten zusammengesetzt. Diese Segmente werden danach übereinander gestapelt, und der Hohlraum wird mit selbstberdichtendem Beton (SVB) ausgefüllt. Somit entsteht eine monolitische Konstruktion ohne Fugen mit hoher Tragfähigkeit. Nachdem der Turm aufgebaut ist, werden äussere Spannglieder - wenn erforderlich - an der inneren Seite des Turms montiert, um die Gebrauchstauglichkeit weiter zu erhöhen.

Um die Eingenschaften der neuen Methode zu evaluieren wurde ein 16.15 m hoher Prototyp errichtet (Fischer, 2015). Der Prototyp bestand aus sechs polygonalen Segmenten. Die Segmente hatten unterschiedliche Höhen. Die Höhen sind so gewählt worden, dass verschiedene Details ausprobiert und geprüft werden konnten. Um die teschnischen Herausforderungen bzw. die Herangehensweise mit diesen umzugehen darzustellen wird die Errichtung des Prototypenturmabschnitts Schritt für Schritt beschrieben. Auch auf eine Untersuchung der Herstellgenauigkeit der Doppelwände wird viel Wert gelegt. Hierbei zeigte sich, dass die Bauweise mit den Abweichungen umgehen kann, selbst wenn manche Abweichungen wie die Dicke der Doppelwandplatten sehr groß sind. In jedem Fall können Empfehlungen für eine genauere Herstellung der Elemente gemacht werden. Wobei die erfolgreiche Errichtung des Prototyps schon gezeigt hat, dass die Bauweise vielversprechend ist.

Abstract

Recent regulations subsidize the energy industry to invest more into renewable energy sources in order to reduce man-made green house gas emissions. These actions led to the rapid development of modern wind turbines and their tower constructions as it turned out to that wind energy is an efficient, affordable and reliable source of renewable energy. The most recent onshore trends are hybrid tower constructions with a concrete section out of precast elements that address to the downsides of the erection of steel and monolithic concrete towers. The precast elements enable a very fast tower erection. However, their geometry is limited due to the transport and a big pre-stressing force is needed to keep the elements in place.

A new construction method for wind towers out of semi-finished concrete parts is being developed at the TU Wien. The innovation however aims to combine the advantages of the precast and cast in place building methods. The hollow double wall elements can be assembled into polygonal ring segments and fixed to each other forming a tapered shape. The segments are then placed on top of each other, whereby the hollow space is simultaneously filled up with self compacting concrete (SCC). Thus a monolithic reinforced structure without inner joints is achieved, providing a satisfactory structural resistance. After the tower is finished, external pre-stressing tendons can be placed on the inner side of the tower if necessary, further improve the survivability of the construction.

To evaluate the performance of the new technology a 16.15 m high prototype was built (Fischer, 2015). This prototype consist out of six prismatic segments. Not all segments had the same height. The heights were chosen in a way so that different stages of the construction process could be tested and evaluated. To introduce the technological challenges and approaches to them the production of the precast elements and the erection of the prototype is described in this thesis step by step. Measurements were taken on each single element in order to examine the magnitude of the production deviations of the new building method. The conclusions demonstrated that in most cases the production works with the acceptable deviation rate, however there are a few exceptions; such as the thickness of the single shells. These errors have a significant influence on the required technology by means of transportation and crane capacity. To eliminate the failures or to reduce the failure rates at least, suggestions are given for the specific work phases. All in all the successful erection of the prototype tower section demonstrated the feasibility of the proposed construction method.

Contents

Chapter 1

Introduction and scope of the work

With the increasing need for renewable and clean energy in the modern world, harvesting wind energy has became one of the most popular source of it. It provides a clean, reliable, affordable and sustainable source of electricity. The wind parks have an impact on the landscape that is accompanied by noise pollution, however taken into account their geographic situations neither of these features influence the densely populated areas.

The European Commission recognized the favourable properties of wind turbines, and therefore favours wind energy as a solution for a secure, competitive and decarbonised energy system European Commission (2011). The Energy Roadmap 2050 is a normative discussion of the European Commission to reduce man-made green house gas emissions by 2050 by 80 %. In order to achieve this objective the renewable energy sources would have a very high share in gross final energy consumption in 2050 (up to 75 %).

These future goals urged the industry to develop more efficient, higher and cheaper wind turbines. In the last decades the research and development came up with numerous methods and solutions to maximize the energy output from "harvesting" wind. As the speed of the ground-near air stratum is increasing with the altitude, one of the most important tasks when building a wind power plant is to have increased hub heights and bigger rotor diameters. With this idea in mind wind turbines can be built anywhere, even in regions with low ground-near air movement. The acceleration of wind speed results in the cubic, additionally the increase of the rotor diameters results in a quadratic growth of electrical power P in [kW]. As a rule of thumb; each meter of height provides a 0.5 - 1 % gain of annual electricity output.

To have it's shares from the most recent innovations, a new method for building wind towers is being developed at the Institute of Structural Engineering at the Vienna University of Technology. The concept offers an onshore construction method based on precast double wall concrete elements, filled with on site concrete. This solution aims to combine the favourable properties of both of these methods. The result should be a wind tower structure with lower construction costs in comparison to the nowadays common methods, still providing the same rate of integrity and reliability.

To measure the feasibility of the concept, a mock-up segment has been already built and tested in the master's thesis of Janjic (2014). This thesis continues his work, evaluating the production and the erection of a 16.15 m high prototype tower. The aim is to optimise the production and construction processes and to identify further technical challenges.

The work is organized as follows. Some of the most recent methods, including prototypes and patterns as well are introduced and discussed in Chapter 2. Tower methods regarding precast elements are introduced mostly in this chapter, as their merits and flaws are the most relevant for this thesis. Chapter 3 is devoted to the introduction of the new construction method, discussing the concept and the prototype. Furthermore it contains the production of the elements and the erection process of the tower prototype. As closure for the chapter the evaluation of the given technology regarding it's accuracy and tolerances can be found. Chapter 4 provides a summary and conclusions of the given results of the new technology, eventually further suggestions are discussed for future adaptations. Additionally the plans of the designed prototype and the results of measurements regarding the various elements can be found in the annex.

Chapter 2

State of the art in the construction of pre-fabricated wind turbine towers

As renewable energy sources are getting more important, wind turbines are getting higher in order to provide a higher energy output. It has been found out that designs using prefabricated parts are the most economical and therefore these tower building methods will be the subjects of this chapter. A more general and widespread summary of all tower building methods can be found in Janjic (2014).

2.1 Wooden towers

One of the latest developments for wind turbine bearing structures are the ones made out of timber. A german company called TIMBERTOWER (2014) already provides two different tower types with either hub heights of 100 m or 140 m (TT100 or TT140). For additional data see Table 2.1.

The polygonal towers have a foundation on which a wooden framework is placed, that allows to mount it with cross-laminated-timber (CLT) walls, furthermore the top of the tower is equipped with a steel adapter, connecting the wooden structure with the turbine. Two different variants of foundations were developed. The first one is a conventional circular concrete slab with a polygonal socket, equipped with steel embodiments, enabling to mount the wooden framework to the foundation (Fig. 2.1). The second variant is a new developed CLT foundation which have never actually been built. The use of a wooden foundation compared to a concrete variant reduces the abortion costs and can be therefore an interesting foundation type where also new challenges arise.

Figure 2.1: The upper part of the concrete foundation serves as the starting platform for the timber tower, taken from TIMBERTOWER (2014)

In the next step a section of a pre-assembled wooden framework is placed at the top of the foundation. These frameworks are placed on the top of each other, enabling the transport of service personnel, data and electrical wiring (Fig. 2.2). In addition this framework allows to mount it with CLT wall elements (Fig. 2.3). These CLT wall elements are the main bearing structure of the tower. They shall transmit the bending and normal force loadings induced by dead weight, wind and the operation of the turbine into the foundation. Herein the most outstanding detail is the connection of the CLT elements in the vertical direction. At the horizontal joint of the slabs several steel sheets with holes are placed in slots and glued to the wood (Fig. 2.3). The production company claims that this connection and the usage of CLT elements make the structure last longer, up to 40 years, under fatigue loading, than the nowadays common building techniques out of steel or concrete. In order to protect the CLT walls from environmental attacks, all CLT elements are covered by a synthetic coating from the company SIKA (Sikaplan SGK). The stocking of framework and CLT elements is continued till the top. There a steel adapter is mounted on the CLT elements connecting an ordinary turbine with the wooden tower.

Such a building technique is always accompanied by advantages and disadvantages. The usage of renewable and CO2 saving building material is the biggest positive aspect. Moreover, wooden structures can easily be dismantled. Whereby all this has to be rated under the premise that the structure is protected from the environment. Any error in the covering coating or any joint may lead to a damage in the structure what can have a big impact to such dynamic loaded buildings. Therefore it may be assumed that the maintenance cost may be higher and the lifespan of the structure could also easily be shortened by environmental effects.

(a) (b)

Figure 2.2: The truss structure supports the power cables, an elevator and a ladder, (a) showing the framework from the outside, (b) from the inside, taken from TIMBERTOWER (2014)

Table 2.1: Main technical details for the towers TT100 and TT140, taken from TIMBERTOWER (2014)

	TT100	**TT140**
Hub height (m)	100.0	140.0
Total height (m)	138.5	196.0
Number of CLT plates used	54	96
Total weight (t)	192	400
Base diameter (m)	7.0	11.3
Rating (MW)	1.5	2.5

Figure 2.3: Assembling the sheathing panels in a spiral orientation, taken from TIMBERTOWER (2014)

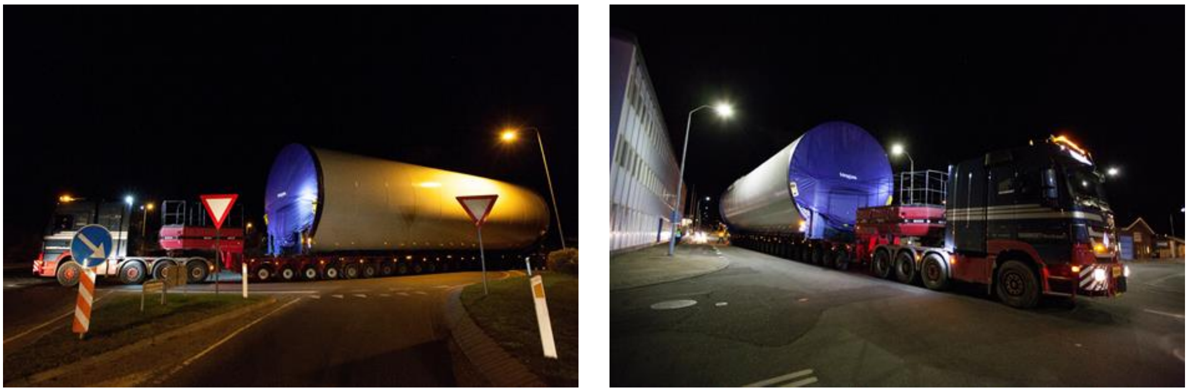
2.2 Steel and hybrid towers

The nowadays most common wind turbine towers are made out of steel or a hybrid structure with a concrete section carrying a steel section. Until a certain height of about 100 m constructions out of steel are cost efficient and decent solutions, but as the towers had to become higher due to the higher energy output, the disadvantages of steel towers become clear. The vibrations caused by the wind and the rotor movement threatens the constructions due to resonance failure. One of the main designing requirements is that the structure's natural frequency isn't equal to the frequencies resulting from wind loading or the operations of the turbine (Hau (2014)). The disadvantages of steel towers is that due to their thin wall thickness their natural frequencies are close to the one resulting from the turbine operation. To increase the stiffness, the wall thickness of the lower part of the tower can be increased, but this is accompanied by difficulties in the production, transport, welding and thus the costs are increased.

To solve this problem, Grolman (2003) suggested to use cast iron segments for the lower section of the tower, and to have a conventional steel section on top of it. Not only the production costs could be reduced - as the cylindrical pipe elements can be produced without bending or welding -, but cast iron has a significant damping capacity, reducing the effect of vibrations.

A few companies however developed various tower systems entirely made out of steel. One of them is the so-called "Large Diameter Steel Tower" (LDST) by the company Vestas (2014). This method is intended to be a cost effective innovation, which enables tower hub heights up to 140 m and more. Increasing the tower altitude however not only increases the energy output of the turbines, but also the force - resulting by the wind and the usually bigger turbine rotor - on the base of the tower. The difficulty with steel towers is, that the nowadays manageable diameter for such structures in terms of transportation is approximately 4.5 m. In order to be able to withstand the increased loads with this maximal diameter, thicker steel plates have to be used in the lower parts of the tower. The walls there can easily reach a thickness up to 65 mm. Obviously there's a limit for constructions built by this method, as the thicker steel plates are hard workable, and at a certain dimension the quality of the material cannot be guaranteed.

Vestas (2014) however found a specific solution for this problem. Instead of increasing the wall thickness endlessly in the lower areas, they decided to use a wider base diameter. The growth of the base radius enables the use of only 35 mm thick plates all along the tower. This also means, the two bottom sections - the ones with large diameters - are delivered in three separate lengthways elements (Fig. 2.4).



(a) (b)

Figure 2.4: The lengthways segments are delivered to the construction site, taken from Windpower Offshore (2013)

So the whole 140 m high tower can be put together by 9 main parts. Two times three lengthways elements for the bottom two segments and three standard one piece circular steel tubes for the upper sections (Fig. 2.5). With this concept the bottom parts are easily transported on a flatbed truck and assembled on the construction site. The two LDST sections can be built in two days with the help of two mobile cranes. The segments are connected on site by bolts in vertical flanges. The whole erection can be accomplished nearly independently from the weather.

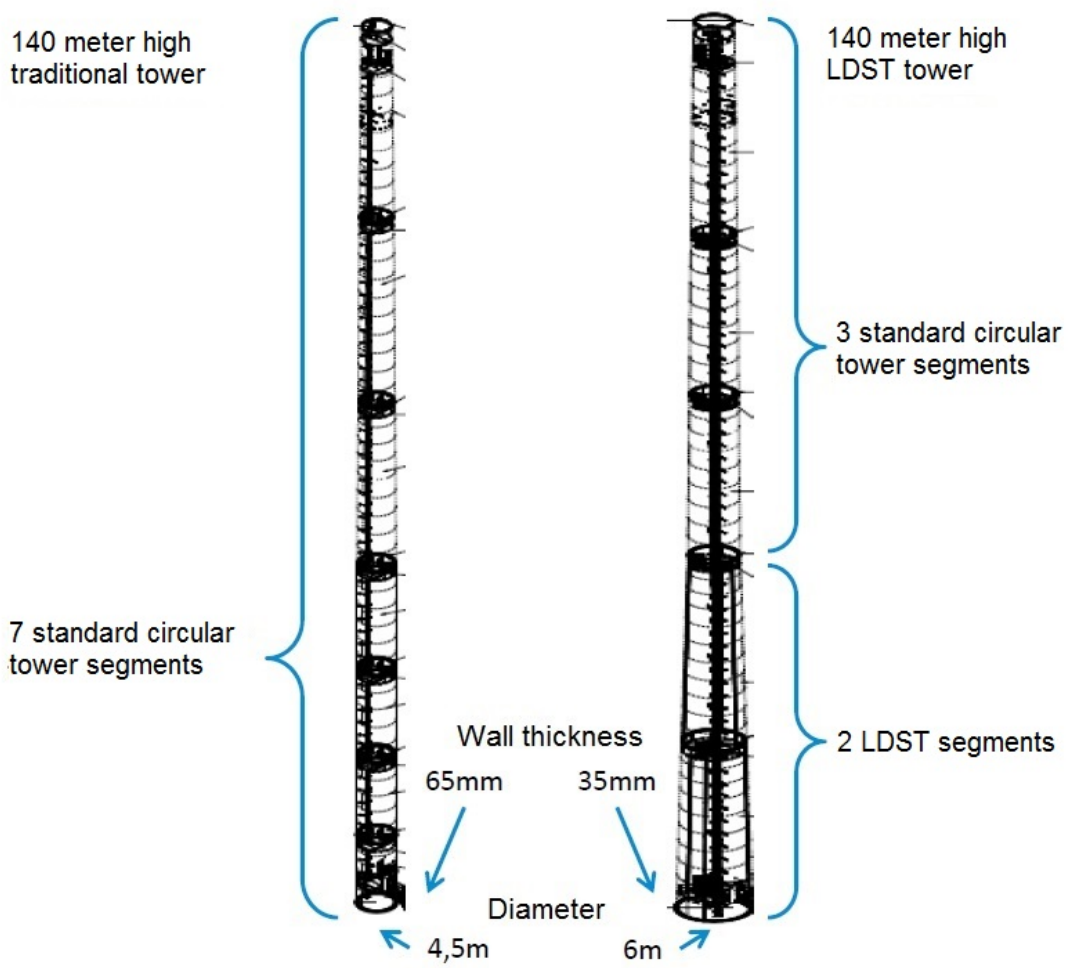


Figure 2.5: Comparison of a traditional tubular steel and the LDST tower, using the thinner lengthways elements to form a larger diameter for the lower segments, taken from Vestas (2014)

Another approach to reach greater heights was developed by the company Voestalpine (2013). Instead of using the most common circular shape, they preferred the lattice tower construction. The main load bearing structures are massive columns, which are positioned at the edges of a polygon; either a triangle, a square or a hexagon. These pillars are connected by smaller diagonal beams, forming the lattice system and bracing the construction (Fig. 2.6).

Figure 2.6: Lattice tower of voestalpine, taken from Voestalpine (2013)

An obvious advantage of this method are the relatively light elements which easily can be transported to the construction site, where the erection can start immediately. The erection begins with the positioning and proper alignment of the main supports at the bottom, only after the other steel trusses are positioned by bolts (Fig. 2.7). The company states that the lattice tower is more cost-efficient than any other hybrid, steel or concrete tower with a similar height of 110 m. Furthermore the lattice construction enables the easy altitude increase of the tower, thus towers over 130 m can be built.

The weak points of the method are undoubtedly the huge amount of bolted joints, and the not so redundant system. Any mistakes in the production or the erection can easily result in severe damage. Thus the works have to be handled with adequate precision and a certain quality management is required. The company however is confident, that the

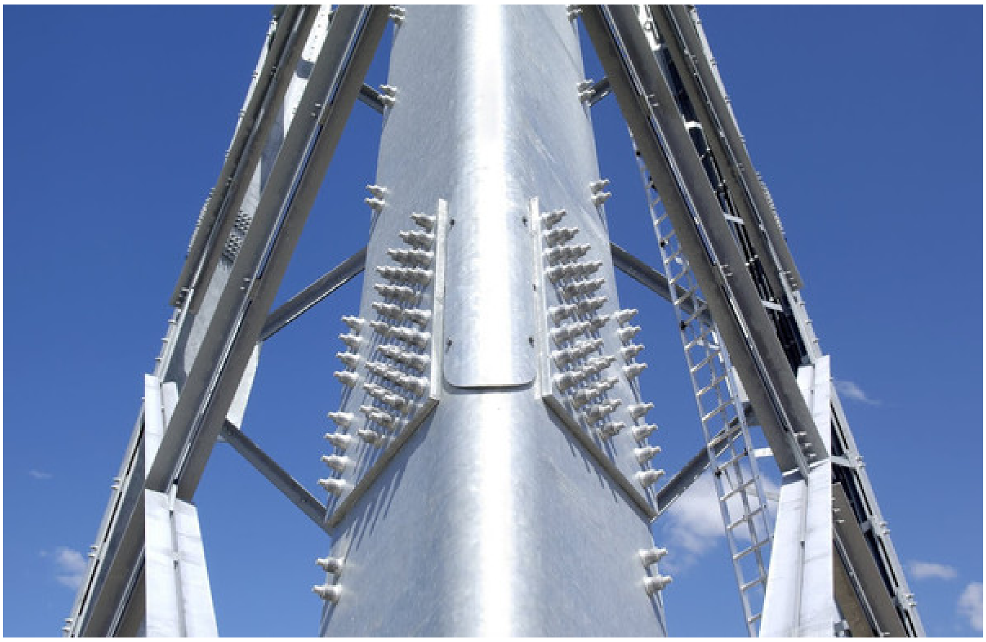


Figure 2.7: Showing the finished lattice tower, taken from Voestalpine (2013)

galvanised steel elements can withstand corrosion, and the system will be functional for 20 years without any repairs. Visual inspections every few years are suggested nevertheless. The bolted joints have a positive side too. The dismantling of the construction is significantly easier compared to the other erection methods, what contributes to an economical overall tower design.

One important aspect for wind turbines is their optical appearance in the landscape, because if the acceptance by local residents is not high enough the construction method will not be used. Thus the visual aspect of this specific tower system has to be mentioned as well. Although it's mostly a subjective opinion, the company states that the transparency of the system integrates their tower better to the landscape than any other variation. The partial transparency may be an advantage here, but the need for a bigger base radius can be a disadvantage at the same time.

2.3 Concrete towers out of prefabricated parts

One of the most common erection methods for high towers nowadays is using prefabricated concrete parts. Max Bögl (2012) counts as one of the leading companies in precast concrete towers worldwide. In 2010 they managed to erect a hybrid tower with a hub height of 128 m using a pre-stressed concrete bottom section and an upper one made of steel. Since then they offer similar variants with a hub height between 123 and 143 m. The 3.80 m high and 0.30 m thick prefabricated concrete elements, that form the lower part of the tower can be produced either as cylindrical or conical rings. By mixing these elements towers with different shapes and heights can be erected. As first step after the production a CNC milling machine mills the top and bottom surfaces, providing the designed accuracy for dry horizontal joints between the segments. After the milling a special coating is applied on the outer vertical surface of the rings as a protecting layer providing an appealing appearance. Once the elements are finished, they can be transported either to the construction site or to a temporary storage site (Fig. 2.8). As the concrete sections can weight up to 62 t, the company has a fleet of over 300 special transport vehicles, and also uses ships to reach the targeted destinations.

Figure 2.8: Max Bögl's production plant full with precast ring segments for wind turbines at the storage site near the coast in Osterrönfeld, Germany, taken from Max Bögl (2012)

The erection begins with the construction of a circular concrete foundation, that is designed in a way, that the tower crane's base can be applied to it. The foundation has to withstand the total weight of the tower - around 1500 t - and dynamic loads (Fig. 2.9).

The assembly of the tower is divided in two phases. Once the lowest segment is exactly placed, a traditional telescopic crane lifts the other elements into their place, using dry joints without any mortar. After the telescopic crane has reached it's limits, a climbing

Figure 2.9: A foundation with the connection to the tower and the tower crane's steel foundation for a hybrid Max Bögl tower, taken from Max Bögl (2012)

tower crane is being built, attached to the foundation. Once it is in working order, the construction process is the following: the crane lifts a new crane segment, positions it then climbs upwards a segment's height length. Into the due to the movement created space the uplifted crane section is glided (Fig. 2.10). With each periodic increase of altitude a new concrete ring can be placed. In the winter of 2011/12 the company managed to erect three turbine towers with a hub height of 140 m with such a modified rotating tower crane.

(a) (b) (c)

Figure 2.10: A telescopic crane (a) and a climbing tower crane (b) with it's operating principle (c), taken from Max Bögl (2012)

The top of the concrete tower is equipped with a special steel adapter ring that enables the transition between the concrete and steel sections. The external post-tensioning tendons are attached to this connecting ring on the one end, and to the tensioning-cellar in the foundation on the other. Once the tendons are post-tensioned the precast section of the tower becomes load-bearing for the turbine operation.

An other german company, called Ventur (2013) approaches the wind turbine tower erection differently. Although they use prefabricated elements for their system as well, they chose pre-stressed concrete as raw material.

The Ventur system has an octagonal tower formation, with flat, trapezoidal wall parts. These parts have a height of 10 m and are placed on top of each other with a 5 m offset. The walls are tapered, resulting a tapered tower structure. After the first eight panels have been placed, all the other pieces are simply lifted to their places from the outside (Fig. 2.11). As they are produced to have brackets in the upper 5 m section, and recess in the lower, the accurate interlocking and connection between the surfaces is provided. To aid the erection, the system uses an inner mobile working platform, from which the workers can easily guide the concrete panels (Fig. 2.11).

(a) (b)

Figure 2.11: Erecting process of a Ventur tower; (a) with the placement of an element, and (b) the inner working platform, taken from Ventur (2013)

In order to provide a monolithic structure, the joints between the elements are filled with a special mortar, which is used to seal waterproof tanks. Once the concrete elements are all placed, an adapter is positioned on top of them. This segment has two main functions; to support the upper steel section of the tower and to provide a fix point for the post-tensioning tendons. The post-tensioning is performed from the mobile inner platform. The special form of the elements demands a high accuracy of the manufacturing process. To ensure that the panels have the expected dimensions, steel formworks are used (Fig. 2.12). This enables numerous variations of the geometry especially important for openings or customer-specific needs.

Figure 2.12: Steel formworks provide the expected accuracy of the elements, taken from Ventur (2013)

This system has some unique advantage at the construction site. The tower doesn't have to be pre-assembled on the ground, because it can be built directly from the transporting vehicles. All that is needed on the site is a crane that is able to lift the single parts. The company states furthermore, that their method is cost-efficient for towers in a range of $120 - 200\,\mathrm{m}$ height.

Another concept for precast concrete towers is being developed by a spanish company, Esteyco (2014). The main idea is to erect a tower as a telescopic structure, consisting of cylindrical segments with different diameters (Fig. 2.13). The segments are lifted with heavy-lift strand jacks to their foreseen place. The industry could overcome the limitations currently generated by the capacity, cost and availability of large mobile cranes, as well as by the structural limitations of conventional tubular steel towers, states Mendizabal (2014) the technical manager of the company. A cylindrical tower with a planned hub height of 140 m has a shallow foundation with an outer diameter of 20.5 m. An additional advantage of the method is that there is no need for a prestressing chamber in the foundation.

The manufacturing shall take place in mobile factories what results in the reduction of transportation costs. The advantage of the cylindrical shape is the reduced amount of different moulds. For a tower with a hub height of 140 m, 14 panels and 4 mould types are required. The geometry furthermore provides manufacturing simplicity and needs a minimal workload on the construction site. The precast elements are post-tensioned with strands, that significantly reduces the required reinforcement.

(a) (b)

Figure 2.13: Precast parts for a telescopic tower with (a) shell geometry and (b) the interlocking of the segments during the erection, taken from Mendizabal (2014)

The main body of the tower is put together by cylindrical segments, each formed by three precast elements, connected along the vertical joints. The segments are assembled on the foundation in a concentric layer with the help of a telescopic boom crane. The preassembly process takes place from the inside to the outside layers. The first assembled ring is the uppermost section of the construction. Subsequently every section corresponding to the lower levels are positioned and assembled, sealing one level at a time. To enable the concentric positioning of the segments, the vertical joints are designed to be easily accessible from the outside, thus minimizing working time on site as well. A falsework is used to position the elements. The nacelle and the rotor are also assembled on ground, either using the same telescopic boom crane as for the positioning, or two lower capacity cranes depending on availability. As the whole process takes place at a maximum height of 40 m, the assembly can be done with an increased rate compared to other methods that are slowed by the high altitudes. A conservative estimation of the production rate is 1 tower per week, states the company. The internals are all placed at the production in the factory, minimizing finishing works. The main power cables and elevator cables are installed on the top section and positioned after the tower is lifted. The internals such as ladders and electrical systems are compatible with the telescopic configuration as well.

Figure 2.14: Assembling a prototype segment out of three precast elements showing the vertical joints of the telescopic tower, taken from Mendizabal (2014)

The essential part of the erection follows the assembly. Starting with the innermost segment (the top of the tower) the segments are continuously lifted with heavy-lift strand jacks. These fully reusable hydraulic jacks provide high capacity and accuracy at low costs. Once the first section is lifted until it reaches the horizontal socket in the following segment, the sections are connected with prestressing bars (Fig. 2.15). After the two parts are fixed together, the hydraulic jacks are repositioned to lift the following "layer" of the concentric rings (Fig. 2.16). This procedure is continued until all segments are in their place.

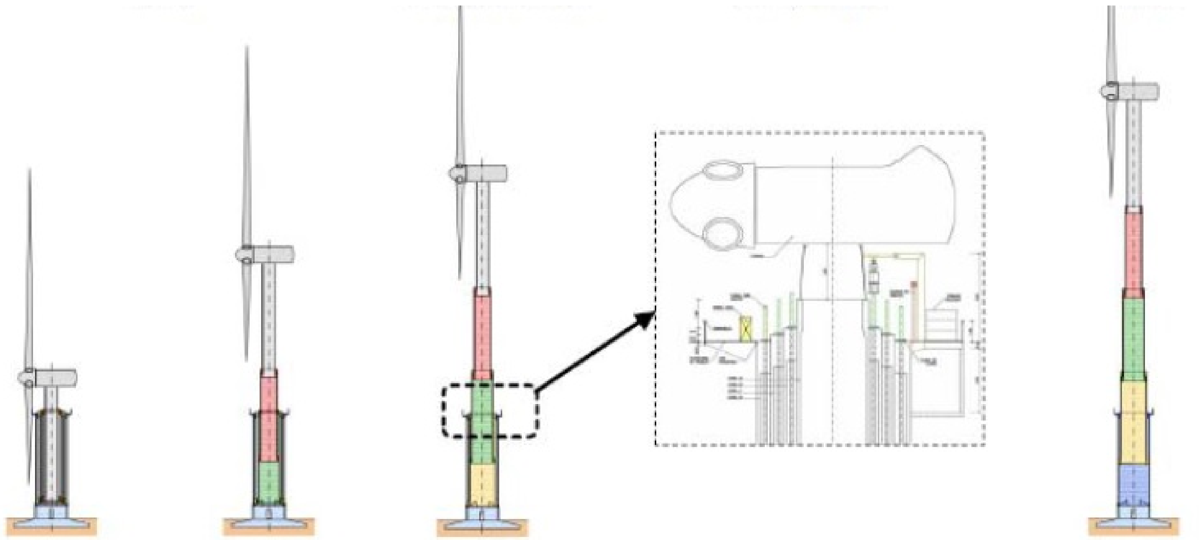


Figure 2.15: Erection procedure of the telescopic tower concept; the sections are lifted one by one with the help of reusable heavy-lift hydraulic jacks, taken from Mendizabal (2014)

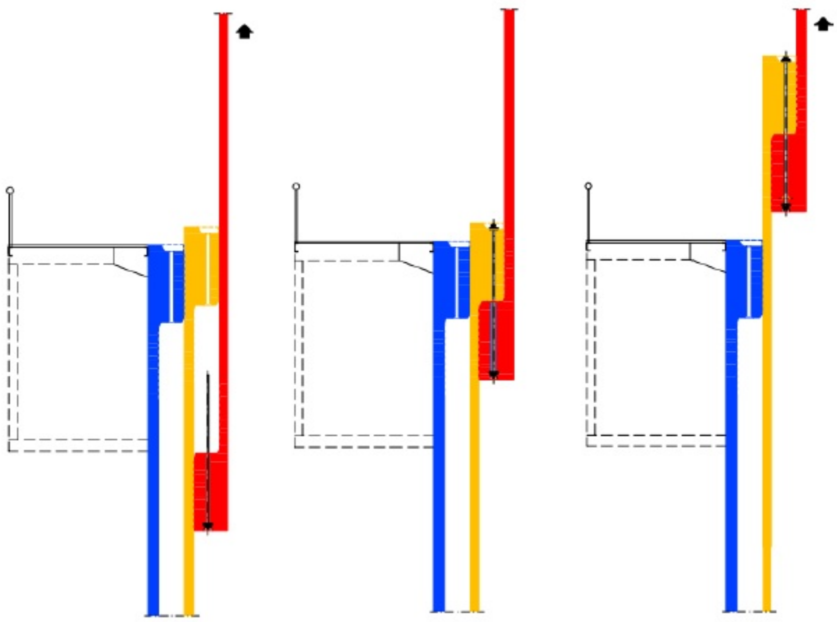


Figure 2.16: Erection procedure of the telescopic tower, (left) the first section is lifted, (middle) applying the prestressing bars into the joint and (right) lifting the interconnected sections, taken from Mendizabal (2014)

The most critical structural element of the whole concept is the question of the reliability of the prestressed horizontal joints. As these connections are responsible for the structural integrity of the tower, their failure mechanics and load capacity have to be examined. To simulate the behaviour of the joints, they were tested. The connections were examined with standard tests for tension and shear, furthermore numerous fatigue tests were made, including the simulation of long term loads with a large number of cycles and extreme case failures. The laboratory test subjects were complementary to the full scale prototype (Fig. 2.17). The experiments show that the joints have two main resistant mechanisms. One of them is provided by into the horizontal joints installed prestressing bars, interlocking the two segments, the other one is the shear friction of the concrete blocks.

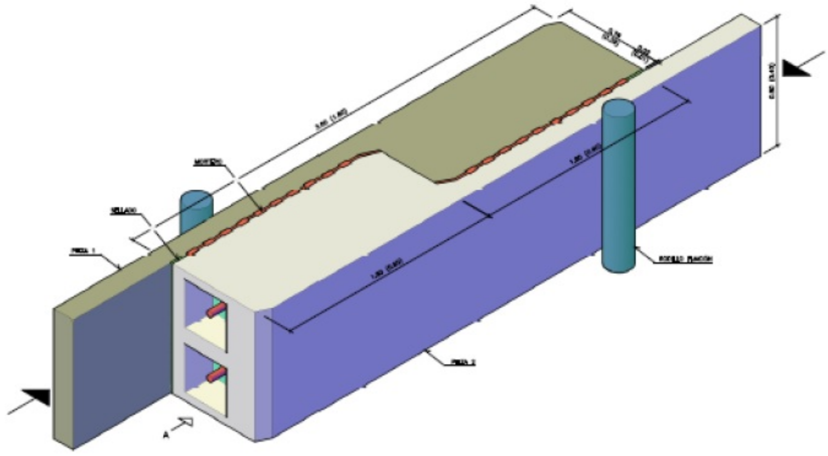


Figure 2.17: The laboratory set-up to examine the behaviour of the horizontal joints, taken from Mendizabal (2014)

The telescopic system is designed for high wind turbines where there is no need for heavy duty cranes. However the concept is accompanied by certain downsides, mainly at the horizontal joints. Especially the relaxation of the post-tensioning strands and the fatigue mechanics. If these problems are handled properly, the telescopic tower building method can be a promising erection method.

2.4 Overview of the most recent patents regarding modular tower constructions

In the following subsections an overview of some of the most recent patents regarding modular tower building is given. These patents mainly focus on various solutions for erecting towers using prefabricated and or post-tensioned concrete elements.

2.4.1 Pre-stressed concrete tower for wind power generators (US 7739843 B2)

A tower structure made of pre-stressed concrete elements is discussed in the patent of Cortina-Cordero (2010). The pyramidal tower structure has three semi-circular segments at the edges and three ribbed flat elements between them. The same curved mold is used for each rounded segment, and the flat walls are fabricated over rhomboidal templates, thus the pyramid form is given.

One of the main ideas of the patent is to make it possible to fabricate the segments on site, thus eliminating the transport costs. Given the maximal dimensions of a diameter of 6 m and a height of 4 m, the in-situ production is possible.

An other objective is to further simplify the erection. For that purpose a sectionalised metallic erecting column is installed at the axis of the tower. The steel column serves as alignment aid for the elements which get attached to it (Fig. 2.18). Furthermore it forms a staircase during the construction phase to provide access to the upper working levels.

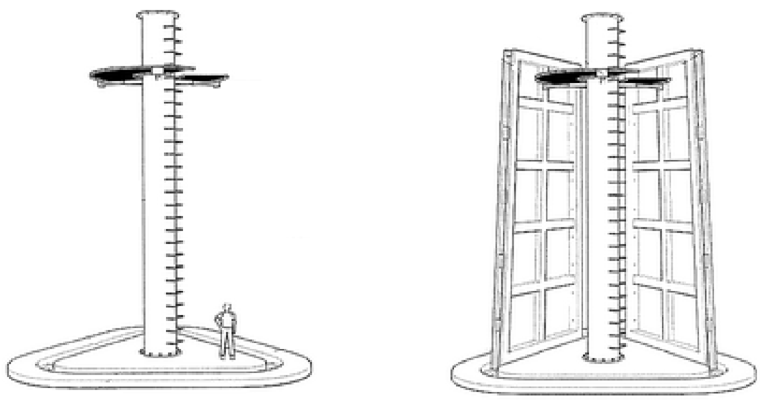


Figure 2.18: Metallic sectionalised erecting column installed to the foundation, providing support to the flat walls, taken from Cortina-Cordero (2010)

As first step the lowest section of the column is attached to the foundation and then the first three flat elements are installed, tilted to their position as the column provides a steady support. After that the semi-circular corner elements are leaned on two of the neighbouring flat segments, forming the first segment of the tower (Fig. 2.19). Each time a full level is complete, the pre-stressing cables are passed through the ducts, are being tensioned and then concrete is poured into the ducts to hold the cables. Ultimately the flat wall elements get so narrow that they are eliminated, and only a tubular ring - consisting of the three semi-circular corner elements - frames the upper sections. When the tower is finished the sectionalised inner column can be removed.

The structural integrity of the finished structure is reached by pre-stressing cables, attaching the semi-circular segments to the foundation, and also to the flat elements. The prefabricated elements have internal ducts to house the cables.

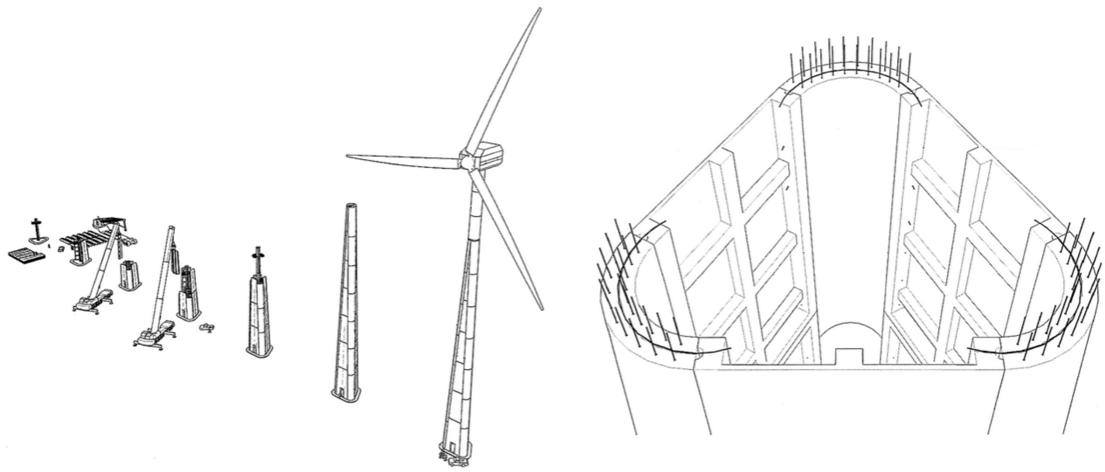


Figure 2.19: Erecting procedure of a prefab tower, taken from Cortina-Cordero (2010)

2.4.2 Prefabricated modular tower (US 7770343 B2)

A modular concrete tower made of thin prefabricated elements is the object of the Fragüet und Bernat (2010) patent. The wall plates have horizontal and vertical internal stiffeners, in which pre-stressing cables can be found. The idea is to have as few as possible different moulds and components to reach a hundred meter high altitude. Thus the tower has three $30 - 35$ m high and 4.5 m wide tapered segments, the lower and the middle ones are made out of five, while the top one is put together by three segment-wise identical components. This means that only 13 elements and 3 different moulds are required.

Once the three segments are standing, post-tensioning strands are passed through the horizontal duct in the ribs of every element, providing the solidity of the structure (Fig. 2.20). When these cables have been pre-stressed the vertical joints between each pair of modular elements are closed. The joints are sealed from the outside and inside by liquid cement.

After the horizontal cables are pre-stressed, the vertical post-tensioning strands are placed in the ducts of the vertical wall ribs. These tubes are subsequently filled with mortar, that secures and integrates the cables inside the tube and hence inside the walls.

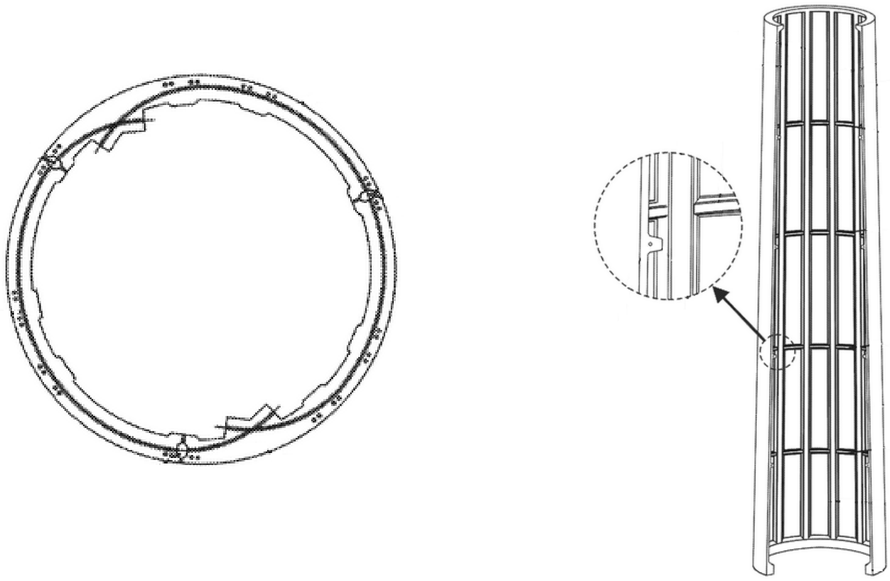


Figure 2.20: Position of the post-tensioning strands in the segments of the tower concept, taken from Fragüet und Bernat (2010)

The authors state that the method has several advantages, such as the short construction time resulting from the usage of a reduced amount of elements. Furthermore the elements are light due to the thin wall thickness that results in the easy transportation and handling on the construction site. The most important factor however is that the whole structure is post-tensioned, what shall provide a certain tower stiffness.

2.4.3 Concrete tower (US 8220212 B2)

The use of concrete with varying compressive strength over the height of the tower is discussed in the patent of Stiesdal (2012). As the wind turbine structures have a conical form in general, the upper diameters are significantly smaller than the lower ones. If the tower is equipped with post-tension strands - anchored only to the foundation and to the top of the tower without any other anchor points in between - an approximately constant normal force over the entire height of the system is given. In order to be able to withstand these forces with lesser concrete areas, a higher compressive strength has to be used (Fig. 2.21). In the patent it is stated, that with the use of materials of varying quality, the production costs of the structure can be optimised.

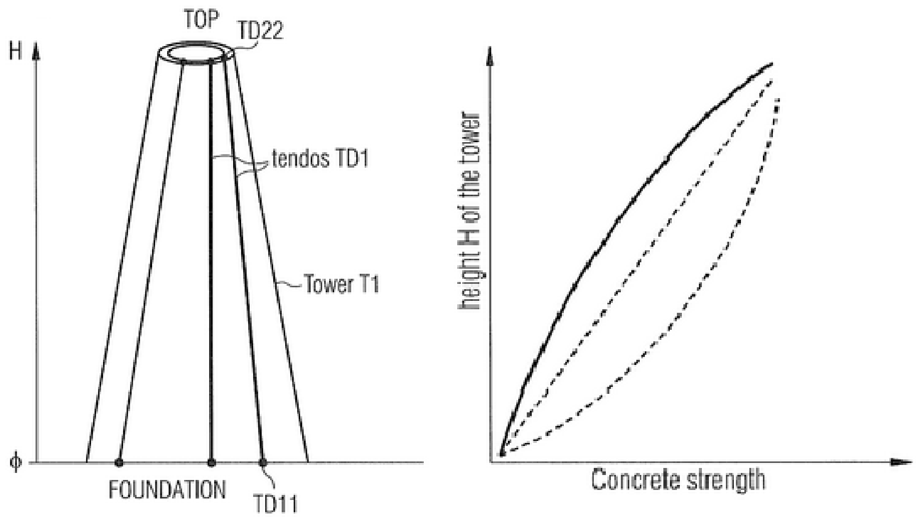


Figure 2.21: Required concrete strength over the tower height for a wind turbine tower according to Stiesdal (2012)

2.4.4 Wind turbine (EP 1474579 B1)

Another method for the erection of towers out of prefabricated elements is given in the patent of De Roest (2006). In this patent horizontal polygonal rings are formed using precast concrete parts. In the vertical direction several rings are placed on top of each other, connected by mortar or post-tensioning strands. Whereby various suggestions are made for the design of the segments.

The first suggestion is forming a regular polygonal cross section, with an even number of angles, using identical elements. The elements have a trapezoid shape, and are placed on each other with an offset, preventing long vertical joints. This approach has the advantage that the whole tower can be made with a single mold, as the lateral formwork can be adjusted to the desired element.

At the second suggestion a regular polygonal cross section, with asymmetric elements is used. In this case each side of the polygonal ring consists out of two different elements, preferably a short and a long side, which together form the tapered symmetric trapezoid. The shorter part can be a parallelogram with a constant width, while the other forms a tapered trapezoid shape. Similar to the first variation, the required mold can be geometrically adjusted to the form, and hence the production of the elements becomes easier.

The third suggestion is an irregular polygonal cross section, with an even number of angles. The side surfaces of the tower alternatively consist out of rectangular and symmetrical tapered trapezoid shapes. The rectangular shape is preferably alternating on the left and right side of the elements, thus preventing the formation of long vertical joints. The advantage of this design is that there are always some elements with a constant width, that enables the easier use of post-tension cables through the whole tower.

The last suggestion is a circular cross section, with the shape of a hollow cylinder. To

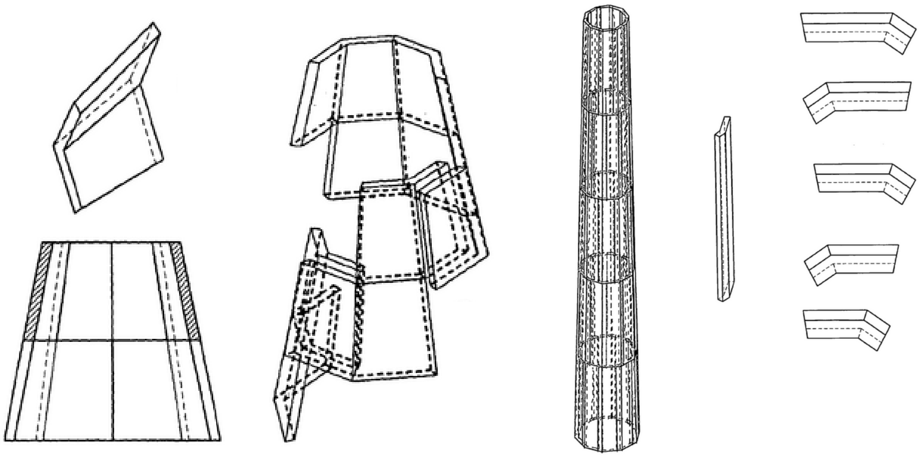


Figure 2.22: Segments out of symmetrical and asymmetrical elements, taken from De Roest (2006)

ensure that the required resistance and mass is reached, the elements can be produced with different wall thickness and/or diameter for each segment. As the radius change for each section, the walls should be placed on top of each other in a way that it is still possible to position the post-tension cables.

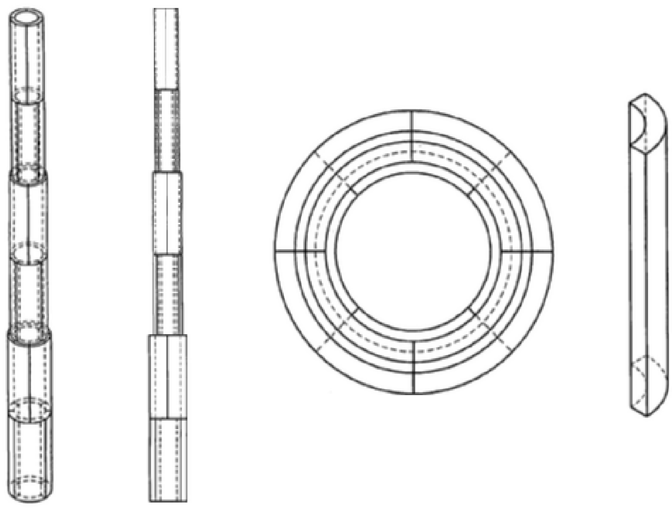


Figure 2.23: Cylindrical shaped tower with varying segment diameter, taken from De Roest (2006)

Chapter 3

New construction method for wind turbine towers developed at the TU Wien

A new method for constructing wind turbine towers out of precast concrete elements is being developed at the Vienna University of Technology. To evaluate the feasibility of this innovation a prototype was built. In this chapter the tower construction concept, the double wall production, the erection of the prototype as well as the recorded double wall production tolerances are discussed.

3.1 Tower construction concept

The here presented concept combines the advantages of prefabricated and cast-in-place concrete structures in order to gain a cost-efficient and fast tower erection method. In other words, this is a concept where semi-finished parts, double walls, are used to erect a tower. Double walls are 50 mm to 70 mm thin concrete slabs connected by spacers. This is a well known technology, which is used in the building construction, and there are many manufacturing lines all over the world producing them. Due to their light weight, they can be transported cheaply by common lorries to any construction site. On site they are positioned, connected to segments, stacked over each other and the hollow space between the double walls is filled with concrete. The advantage of such a construction method is that the time consuming on-site erection of framework is spared. Moreover the thin double wall concrete slabs function as bearing structures for the final state of the tower.

In our case the double walls are arranged and connected to tapered or cylindrical polygonal ring segments. In this process each joint is equipped with reinforcement. This reinforcement guarantees a continuous force transmission through the whole tower structure in it's final state. Additionally, during the construction phase a special welded connection between the double walls is used to gain load bearing segments.

The segments have to be stacked one over another and filled continuously with concrete in order to gain a tower structure . Inevitably all joints resulting from this kind of tower erection have to be sealed. Whereby different sealing types have to be used depending on the joint's orientation (horizontal or vertical). The lifting and stacking can only be fulfilled if the segments provide embedments that can transmit the self weight of at least two segments. Three of these embedments have to be equally distributed over the circumference of the polygonal cross-section at the top and the bottom of the segments. They can also be used to position the segments correctly at their foreseen position in the tower. For example these embedments could be equipped with sockets that can receive steel rods which could be used to guide the segments at the positioning process. After each segment is positioned the underlying one is filled with concrete. The way the concrete is brought into the double walls depends on the height of the segments.

The segment height is approximately limited to 13 m because the double walls are currently produced and transported with this maximal dimension. Such big heights make it nearly impossible to introduce and compact the wet concrete from above. Therefore, self compacting concrete (SCC) is fed into the hollow segment core from an at the bottom of the segment situated opening. This concreting procedure results in a very high concrete pressure, but a dense and a blowhole free filling with no need for additional manual compacting is gained. This continuous process of lifting and concreting allows to raise a fully reinforced structure without joints in the core. Such a structure can be designed with no need for post-tensioning.

The relevance of the proposed building method can be shown if it is compared to other concrete tower building methods. Plane double walls weigh approximately one third of fully bodied precast concrete ring or partial ring segments, which are used to erect towers. Therefore, they are easy to store, transport and manipulate by construction machines with a smaller lift capacity or if the same machines are used three times bigger segments can be manipulated.

The next very important subject is the load bearing capacity. Towers out of full precast elements need to be post-tensioned and therefore high strength concrete is necessary to bear the high pressure loads. In contrary semi-precast building methods with continuous reinforcement show nearly the same load bearing behaviour as cast-in-place structures with the same material usage, whereby post-tensioning can be replaced by normal reinforcement.

This positive effect is compensated by the additional workload for the assembly and sealing of the segments. This material saving design goes hand in hand with a climate dependent concealing procedure, which still can be carried out fast with high pouring rates and no need for additional formwork. Concluding, it is obvious that the herein described semi-precast tower erection method is very promising.

3.2 Prototype

To proof, that the concept is working and feasible, a prototype tower section as shown in Fig. 3.1 was built. The prototype has a nonagonal cross-section and consists out of six segments, which stacked one on top of another result in a 16.15 m high structure. The walls of the five bottom segments were produced in a way so that the segments taper with one degree in relation to the vertical axis. Only the segment on top was produced without any inclination. Therefore, the outer circumference of the nonagon at the bottom amounting to 4.15 m narrows to 3.80 m at the top.

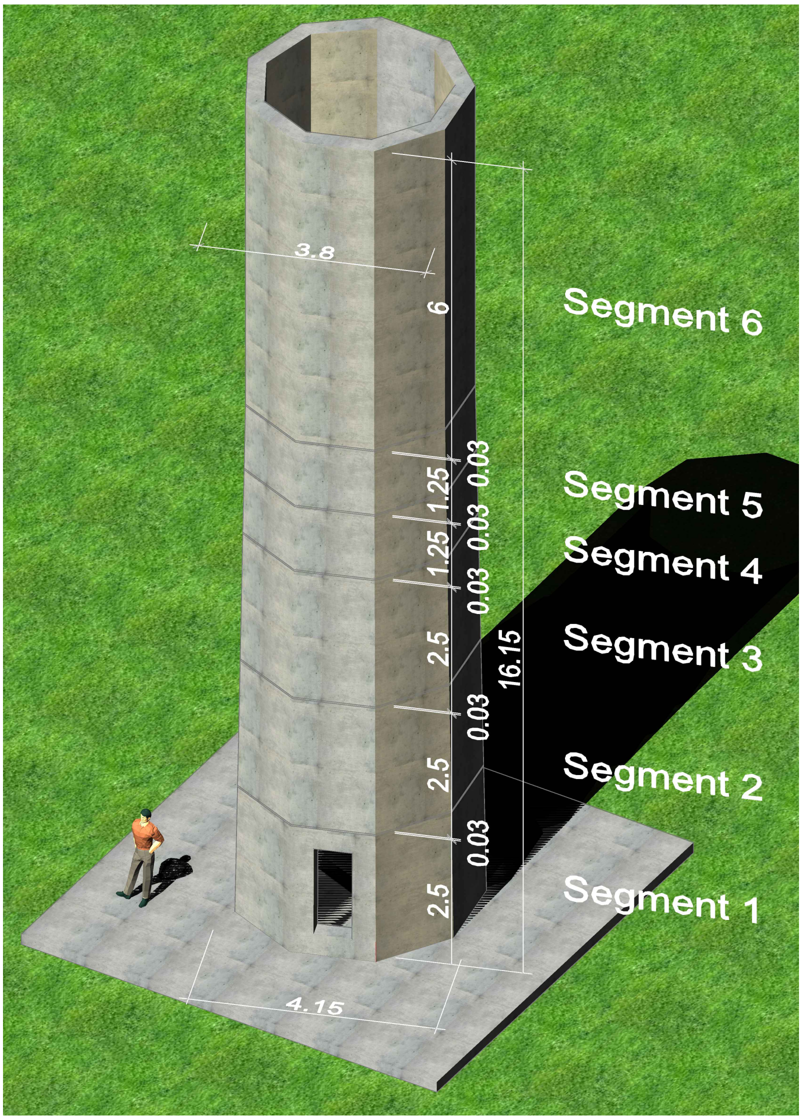


Figure 3.1: A rendering of the prototype tower section with it's main dimensions in [m]

Segments with three different heights were planned allowing to try out and test structural details with minimal effort and the influence of the height on the double wall production as well as on the tower erection process. In order to observe the production deviances which can be achieved in the precast plant, and in order to test the performance of the designed details the first three segments were chosen with a rather small height of 2.50 m. The next two segments had a height of 1.25 m, which allowed to try out the filling of two segments with concrete and to test the horizontal and vertical joint sealing. The last segment was chosen to be 6.00 m high in order to show that also high segments can be assembled using the same design as used before for the smaller segments. An overview of the geometrical dimensions of the segments according to Fig. 3.1 is given in Table 3.1.

Table 3.1: Geometrical overview of the prototype tower section segments, according to Fig. 3.1

	Top outer polygon circumference [m]	Bottom outer polygon circumference [m]	Height [m]
Segment 1	4.12	4.21	2.50
Segment 2	4.03	4.12	2.50
Segment 3	3.94	4.03	2.50
Segment 4	3.89	3.94	1.25
Segment 5	3.85	3.89	1.25
Segment 6	3.85	3.85	6.00

The six segments and a 30 mm high joint between these result in the 16.15 m high prototype. The height of the joints is chosen according to the recommendation for horizontal double wall joints where compression forces can be transmitted (EN 14992 (2012)). This joint size is big enough so that concrete can fullfill it without any relevant disturbance for an aggregate size up to 16 mm. In addition this joint size allows to set up the segments so that the tower axis is vertical.

3.2.1 Double wall production

The double walls for the prototype tower section have been produced by the company Oberndorfer, in one of their fabrication plants located in Herzogenburg, Austria. All walls consisted out of two 50 mm thick concrete slabs with the strength of C30/37, connected by spacers so called Kappema waves according to DIBt (2011), whereby the overall wall thickness amounts to 300 mm. The slabs had a trapezoidal shape which tapered from the bottom to the top, whereby the outer slab - which will later face the outside - has to be wider than the inner one, see Fig. 3.2. In addition all outer shells were equipped with an embedment providing two welding bases which were positioned every two meters, and other standard double wall embedments like lifting anchors and plastic threaded sleeves for the temporary fixing and positioning of the double wall elements on site.

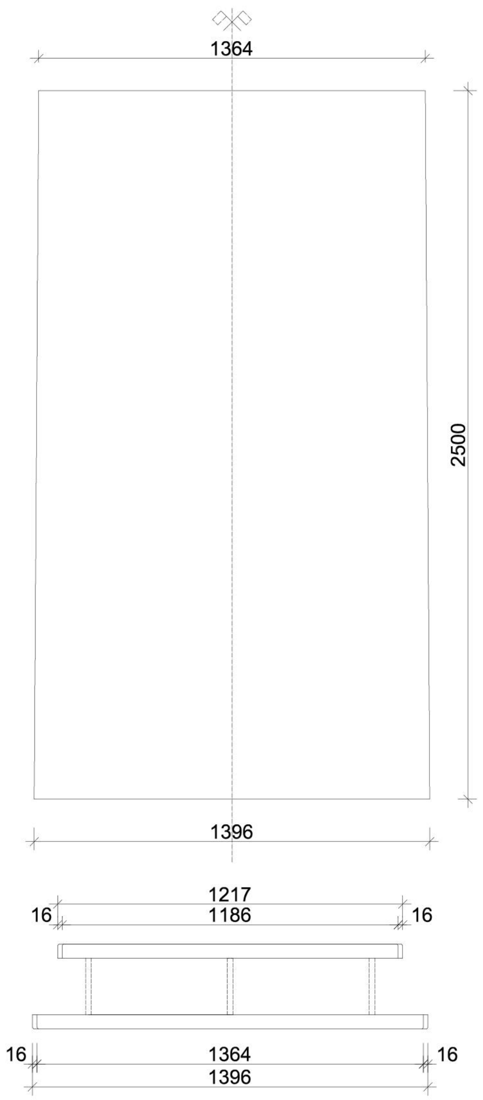


Figure 3.2: Standard double wall for Segment 1; front and ground view

The previously described basic double wall configuration was altered according to additionally needed features. As the segments needed to be lifted and positioned three over the circumference of the segments equally distributed embedments were positioned at the top and bottom of the double walls. In our case, where we had nonagonal segments, every third double wall had two of these embedments, situated centric. The second feature was a special opening in one of the slabs, which had four threaded sleeves surrounding it. This opening was used to mount a hose to it where concrete was pumped through. The last of these features were a crimped steel reinforcement bar which should strike out in the hollow core of the double walls. In the embedded state the crimped bars shall stitch out in a way, that they form loops which later can be equipped with steel cables.

The production is mostly automated, only a few workers are needed for the whole process. Due to the high level of automation the production provides a certain accuracy. The factory uses special steel pallets, that are transported through by roller conveyors. The production circulation is divided into working stations. Once the work is finished at a station, the steel pallet is moved to the next one. As next an overview of the production process is given whereby the focus is laid on the specialities of the double walls used for the prototype. A detailed description is given in the work of Janjic (2014).

The production line begins with the cleaning of a steel pallet. To provide a clean and flat surface, any stain that may have remained from the previous production has to be removed. As the steel pallet is fully cleared and dried, it is transported to the next area. At this stage a fully automatised robot marks the areas for special embedments and places the lateral steel formwork elements onto the pallet, one at a time (Fig. 3.3). The placement is done by CAD/CAM data. The steel formwork is then fixed onto the pallet by means of magnet bolts. The robot works only with elements of standard lengths. Therefore if a specific geometry is desired, the gaps in the geometry have to be filled manually with styrofoam profiles at the next station.

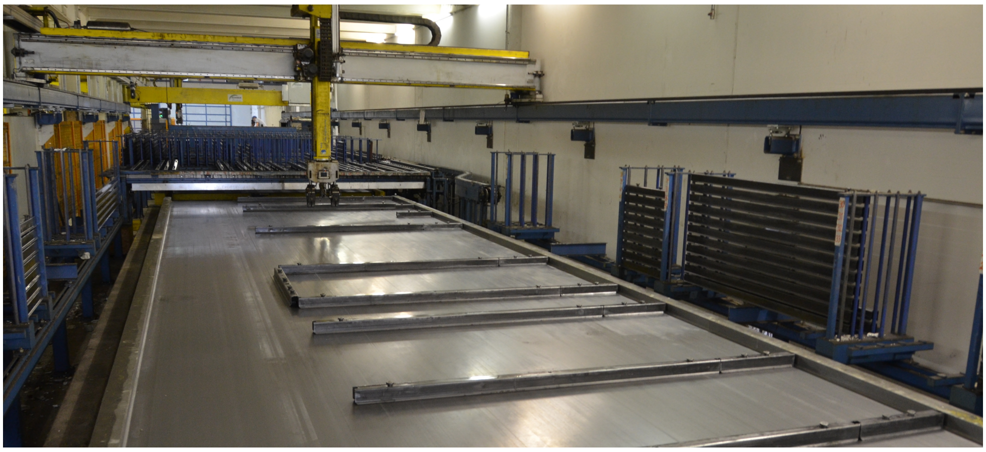


Figure 3.3: Robot in the second station of the double wall production circulation, fixing lateral formwork onto a pallet

At the next station an optical laser device points the lines of the geometry and the place of special embedments on the pallet. Workers manually cut and glue styrofoam parts into the empty areas along the laser's lines to complete the framework. The special embedments and various opening exclusions are placed as well. After the formwork is finished, the workers apply an oil coating on the surface of the pallets to prevent the precast elements from sticking to the formwork. Lastly plastic distance holders are placed. These elements assure that the necessary concrete cover is given (Fig. 3.4).

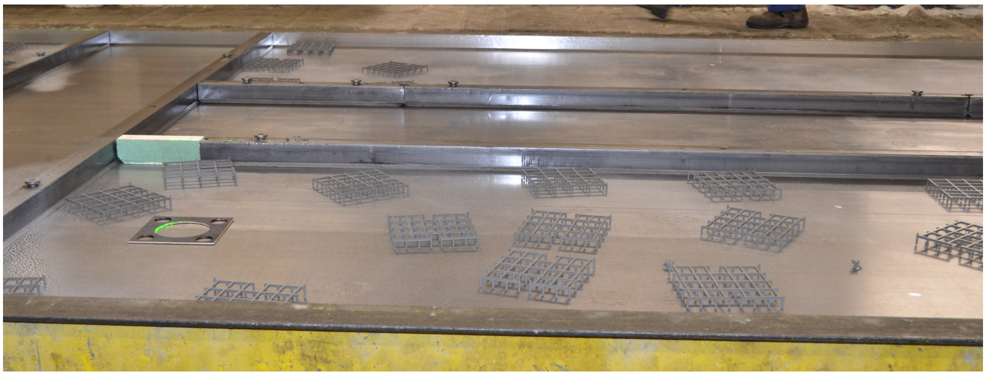


Figure 3.4: Styrofoam filling the formwork gaps, and plastic distancers securing the desired concrete cover

At the third station a reinforcement mat and all other embedments are placed and fixed in between the formwork. The mat is produced automated by a welding robot previously to the placement. The robot can produce completely randomized reinforcement grids with different openings, bar diameter and bar spacings. In some cases mistakes can occur; some steel bars are too long, or the intended openings in the mat are not cut out properly (Fig. 3.5). In such cases the workers have to manually correct the reinforcement with bolt cutters.

(a) (b)

Figure 3.5: Reinforcement mat (a) for the double wall with a recess with too long bars that have to be adjusted manually and (b) a geometrically correct mat leaving space for the pump opening embedment

The Kappema wave elements (double wall distancers) and any additional reinforcement has to be added manually. Due to their setup, the Kappema secure the right distance between the concrete slabs of the double wall and they provide the needed load resistance during all stages of construction, keeping the slabs together. They consist of wave shaped steel sheets with three on-welded steel bars that have hooks which stick into the concrete (Fig. 3.6). If needed they can be fixed to the reinforcement mat so that they stay at their position during the production.

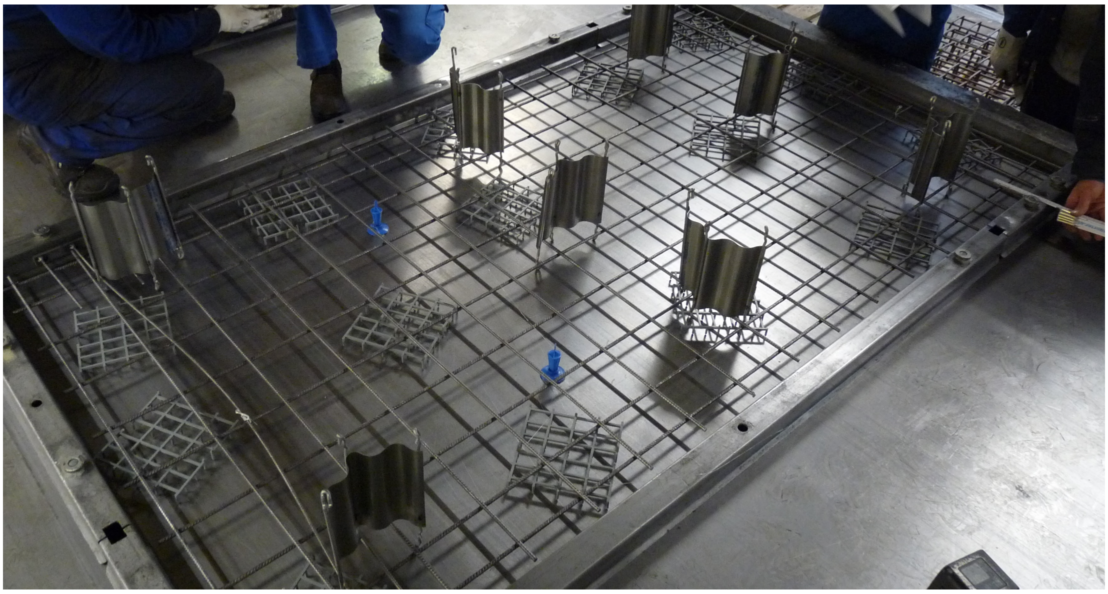


Figure 3.6: Kappema steel waves in their final position, fixed to the reinforcement net

At this station also lifting anchors are mounted to the Kappema (Fig. 3.7(a)). The number needed depends on the weight of the elements e.g. the double walls for Segment 6 were equipped with four instead of two hooks, like double walls for the other segments. Then the weld embedments are placed every two meters over the height. These embedments consist of two 180 mm long and 25 mm wide steel plates connected by a 6 mm in diameter reinforcement bar and anchored by two further 8 mm in diameter reinforcement bars (per plate) into the concrete slab. The connecting bar was designed to be longer than the width of the elements so that it could be jammed between the lateral formwork and therefore fixed in their position, see herefore Fig. 3.7(b). Whereby only the outer slabs are equipped with these welding embedments.

(a) (b)

Figure 3.7: Embedments like (a) lifting anchors and (b) weld bases are attached to the reinforcement mat

Then plastic threaded sleeves are glued on the steel pallet. These sleeves are used to fix bracings to the double walls during the assembly of the segments. Two of them are positioned at one third of the height at the top of the outer shell. As next every third double wall is equipped with lifting and positioning embedments. Two different designs for these embedments were used. the first one was a concrete block with a cylindrical hole in the centre anchored by glass fibre bars in to both double wall slabs (Fig. 3.8(a)). The second one was a steel part out of three steel sheets with the shape of the greek letter "π" (Fig. 3.8(b)). The two vertical sheets had "teeth" which anchored into both double wall slabs, while the horizontal sheet provided a round hole which could be used to attach a lifting or positioning equipment. The hole embedment was coated in order to prevent corrosion. The concrete "block" was the chronologically first idea to transmit a big force through the thin double wall slabs. It turned out that there was no fast and easy way to fix them for the following concreting and vibrating processes, therefore the steel variant was developed allowing to be easily kept in place by magnets.

(a) (b)

Figure 3.8: Lifting and positioning embedment in the shape of (a) concrete blocks fixed manually and (b) a "π" shaped steel part fixed by magnets

The last embedmenst which were positioned were reinforcement loops placed only in three of the nine double walls of each segment (Fig. 3.9). These loops shall make it possible to attach flexible reinforcement to the double walls which is necessary in two of the vertical segment joints.

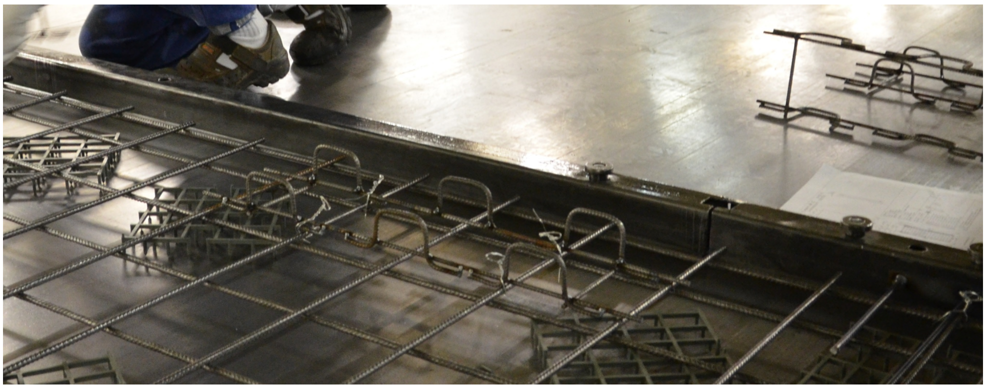


Figure 3.9: Reinforcement loops allow to attach flexible reinforcement in two vertical segment joints

Once the reinforcements and embedments were placed, the work at the third station was finished and the pallet moved to the next station where a crane bucket for concreting was situated (Fig. 3.10). This crane bucket filled the formwork automated with the information about the geometry of the formwork and the desired thickness of the concrete slabs. Only special spots required a bit more concrete e.g. the lifting anchors. Therefore some more concrete was placed manually, resulting sometimes in thicker and heavier elements.

Figure 3.10: Crane bucket filling the formwork with concrete

In order to provide the required quality, strength and the even distribution of the concrete, the pallets were transported to the vibration station. In addition workers spread if needed the concrete to ensure that no undesired uneven spots occur. During the vibration the lifting and positioning embedments were held in their place manually (Fig. 3.11).

(a) (b)

Figure 3.11: Double wall compacting by vibrating (a) whereby the concrete variant of the lifting and positioning embedment was held manually in position and (b) the result of the compacting process with a maybe too smooth surface of the inner side of the concrete slab

As last step of the production the slabs are stored for 24 hours in a special climate chamber. In there the temperature and humidity is set to the perfect curing conditions. This makes it possible to ship them out after one day of curing.

The next day the second double wall slab was produced in nearly the same way the first one was. There were just two differences. First of all most of the embedments were situated in the first slab so that at the third station only the reinforcement mat and the embedment for the flexible reinforcement had to be positioned. The second difference had been that the first slab was turned and pushed in the wet concrete of the second one, so that both had been merged. After that both slabs went into the curing chamber for at least another 24 hours. At the end the finished double walls were gained.

(a) (b) (c)

Figure 3.12: Double wall slab merging process (a) where the pallet with the slabs from the first day of production are fixed into a hydraulic rotating machine (b) the rotation of the pallet and (c) the merged shells after the vibration

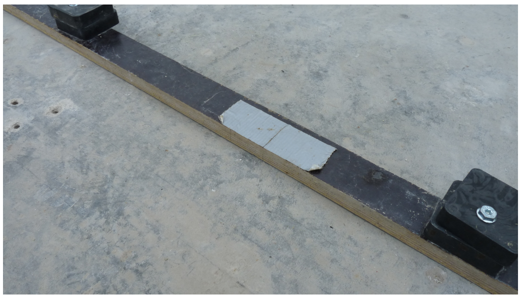
3.2.2 Segment assembly

The segments have to be assembled on a completely flat surface to ensure that any undesired slops or bumps and humps don't hinder the correct double wall positioning. For that purpose a not reinforced concrete pre-assembly field of 8 x 8 m was built, as described in Janjic (2014). This field was later used as foundation for the whole prototype tower section. As a reference point for the geometry, a nail was shot in the center of the pre assembly field. This point was the reference for the whole assembly process.

As first assembly step the ground section of each segment had to be marked on the ground. Therefore a wooden plank was used as compass to draw the outer circumference of the ground section polygon of the segments (Fig. 3.13). The center of the pre-assembly field was the center of the polygons and therefore a nail was placed there. Once the wooden plank was attached to the center nail any circle could be drawn to the ground where by maximum deviations of 1 mm were observed which was within the tolerances. This large scale compass was also used to mark the edges of the ground polygon along the outer circumference. It turned out that an accurate measuring equipment is crucial for this process because an error of 1 mm per polygon side cumulates to 9 mm for the whole nonagon. However, the polygon edges were connected with the polygon centre resulting in rays which could be used to produce analogical polygons as intersection of the polygon rays and any polygon diameter. After that the respective polygon sides and the middle of each side was marked (Fig. 3.14). The middle of the sides was used as aligning point for the double walls. The alignment according to the double wall centre ensured that any errors are equally divided between the two neighbouring vertical joints.

Figure 3.13: A wooden plank is used to draw the outer bottom circumference of the segments with the help of a reference point in the middle

(a)



(b)

Figure 3.14: Marked polygon sides with (a) the mark for the polygon side midpoint and (b) the planks attached to the polygon sides

After marking, the polygon sides were equipped with a guiding construction for the double walls. The simplest guiding construction of which one could think were wooden boards attached to the ground (Fig. 3.15). They were placed at the outside whereby the middle of the polygon sides was marked on them. As next the double walls were prepared for the segment assembly. Herefore the middle of the outer double wall side was marked, so that the elements could be aligned according to this mark. Then the reinforcement loops described in Section 3.2.1 were equipped with flexible steel rope reinforcement and hidden in the elements. In addition small wires were attached to the rope reinforcement and fixed at the outside of the double walls, so that they could be pulled out later and therefore brought into their final position.

Right before the double walls were positioned the pre-assembly field horizontality was measured and two plastic plates per double wall slab were placed establishing the required planarity of the pre-assembly field. As it can be seen in the puddle of water in Fig. 3.15 the uneven shrinkage of the concrete pre-assembly field as well as the not uniform subsoil resulted in a very uneven assembly field with many bumps. Therefore a dumpy level was used to measure out the field and plastic plates with different heights were stacked to piles providing the same level of height for each double wall.

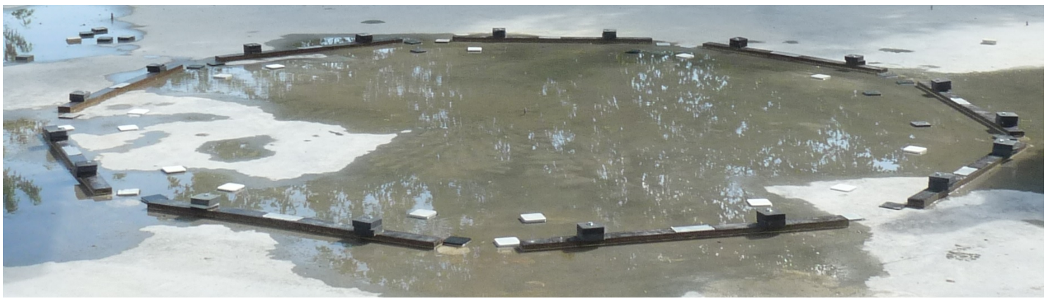
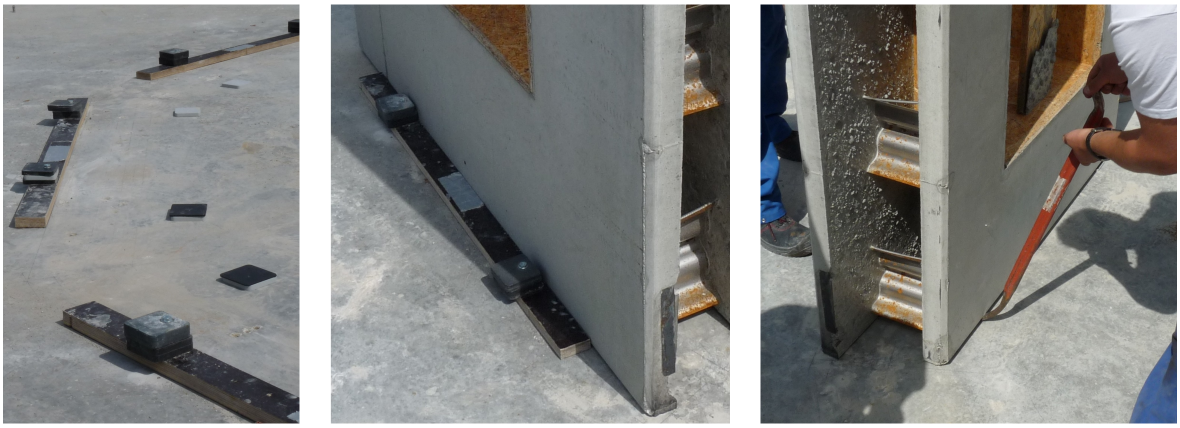


Figure 3.15: Prepared pre-assembly field with alignment boards and height adjusting plastic plates

As next the double walls were arranged forming a segment. This arrangement process consisted of the lifting, the aligning and the fixing of the double walls as well as the placement of reinforcement in the vertical joints. After that the loose elements had to be connected to form a load bearing segment.

In the first step of arrangement the double walls were aligned according to the mark at the middle of the double walls in their foreseen position of the segment. The wooden planks on the pre-assembly field functioned as aligning aid and therefore ease the double wall arrangement, making it faster and more accurate (Fig. 3.16(a), 3.16(b)). The elements were moved by a gantry crane which was within reach of the assembly field (Fig. 3.17). The fine adjustment of the position however was performed manually with a crowbar (Fig. 3.16(c)).



(a) (b) (c)

Figure 3.16: Double wall positioning process with (a) wooden alignment boards and plastic distance plates providing a certain height level, (b) an aligned double wall and (c) the fine adjustment of an element with a crowbar

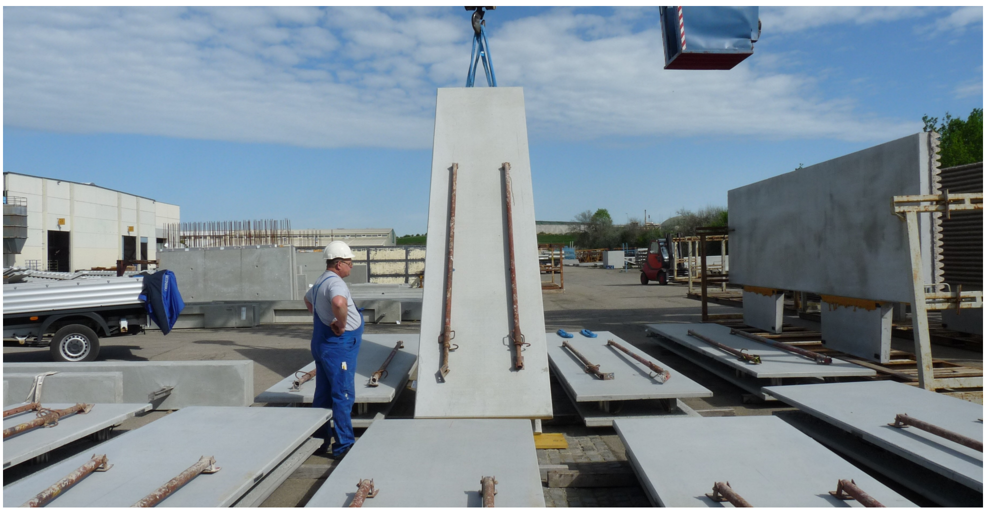


Figure 3.17: Due to the height of Segment 6 the push-pull-props were attached to the elements previous to the lifting with the gantry crane

The elements' desired inclination of 1° was levelled using a digital level. The accuracy of commercial spirit levels is limited if not used for angles of 0 or 90 degrees, nevertheless it was possible to establish the desired double wall inclination. An additional verification of the inclination was that the lateral double wall narrow sides had to be absolutely vertical.

Once the elements were placed, push-pull-props were attached to the double wall and the foundation. The outer double wall slabs had two embedded plastic threads enabling to mount props to it. There was only one exception to this procedure. The double walls for Segment 6 were so high that the props have been mounted to them in advance (Fig. 3.17). On the other end of the props - at the pre-assembly field - threaded expansions bolts, which were anchored in the concrete ground were used to keep the props in place. The special design of the push-pull-props allowed to adjust their length very fine which enabled to establish the desired double wall inclination (Fig. 3.18).

Figure 3.18: Final adjustment of the push-pull-props in order to establish the desired inclination of 1° in relation to the vertical axis using a digital spirit-level to measure the inclination

The double wall inclination in combination with the double wall geometry caused that the whole double wall stood on one edge of the inner concrete slab. Undesired deviations sometimes even let the elements stand only on one point, causing them to rotate a bit around this point. Therefore in some cases it was necessary to prevent the undesired rotation by fixing the inner slab of the element. Steel bars were anchored inside the segment and plastic plates were used as wedges in order to clamp the element between the outer wooden aligning board and the steel bars (Fig. 3.19).

Figure 3.19: View from the inside of a segment showing steel bars with plastic plates used as wedges to clamp the double walls against the outer aligning board, therefore keeping the elements in the right position

After the placement of each element a stiff reinforcement cage (Fig. 3.20) was put in the lateral hollow space between the double wall slabs. This cage provided the horizontal reinforcement for the vertical segment joints, connected by vertical and horizontal reinforcement bars whereby the vertical bars had hook on top allowing to fix them on the top Kappema waves (Fig. 3.20(a)). The joint reinforcement provided the lap for the reinforcement in the double wall slabs, whereby the lap reinforcement could only be placed between the Kappema waves.

(a) (b) (c)

Figure 3.20: Horizontal reinforcement cage (a) before it is placed, (b) placed in the lateral hollow double wall space and (c) fixed by hooks at the top Kappema wave

After the reinforcing the next double wall was threaded sideways over the protruding vertical joint reinforcement. All other steps like fixing, inclining and reinforcing were similar to the first placed element. These steps were continued for all elements except for the last one.

(a)

(b)

(c)

Figure 3.21: Double walls (a) of Segment 1 (2, 5 m high) and double walls of Segment 6 (6, 0 m high) from (b) the inside and (c) the outside fixed by skew bracings during segment assembly

The last two vertical joints could not be reinforced in the same way as the other eight before. The stiff reinforcement would have stuck out of both double wall elements neighbouring the last one and make it geometrically impossible to position the last one. Therefore in advance these two vertical joints were equipped with flexible steel rope reinforcement (Fig. 3.22).

The idea is, that each side of the last two vertical joints is equipped with flexible steel rope loops. Once all elements are at their foreseen position the loops can be pulled out horizontally by wires which were attached to the ropes in advance. In the fixed state the loops from both sides overlapped each other and a vertical bar was placed in this

Figure 3.22: Positioning of the last double wall without horizontal reinforcement, showing the steel cable reinforcement in the neighbouring panels

overlapping area securing the correct positioning while the bar itself was fixed at the top Kappema waves (Fig. 3.23).

However the placement of the last double wall was the practical test showing if all the previous marking and positioning work was done with a sufficient accuracy. If it was insufficient then there would have been not enough space for the last double wall. This case never occurred and all segments could be assembled as planned.

(a) (b) (c)

Figure 3.23: Steel cable loops as joint reinforcement where (a) wires fixed at the outside were used to pull the (b) loops into their final position (c) secured with a reinforcement bar hooked to two Kappema waves

As next the loose elements were connected forming one load bearing segment. Weld bases embedded in the outer shell and positioned approximately every two meters over the height were used to connect the elements. Between the weld bases reinforcement bars, which were used as filler, were placed and welded to both adjacent bases (Fig. 3.24). The bases were designed with a length of 180 mm and possible misplacements and shifts of their position was taken into consideration. Once all connections were established the skew bracings could have been removed.

(a) (b)

Figure 3.24: Double wall connection by (a) weld base embedments with filling bars in-between (b) welded to the adjacent bases

As last step of the segment assembly the inner and the outer joints were sealed. Three different sealing types had been used. While the outer joints were sealed by a cementitious load bearing material the inner ones were sealed with synthetic materials. The outer joints were filled with mortar, which provided the same optical appearance as the neighbouring concrete and is also capable of transmitting shear and compression forces between the double walls. However the right mortar selection and application as well as curing are vital for the optical appearance (Fig. 3.25). If not enough care is taken cracks can occur. The inner joints were sealed either by on-glued foils or round ethylene-propylene-diene monomer (EPDM) tubes (Fig. 3.26).

Due to production deviations the joint widths hadn't been uniform and therefore different EPDM tube diameters were necessary in order to fill the gaps properly. While the on-glued foil was fixed with a special but temperature depending glue the EPDM tubes were kept in place by wires. Both sealing forms worked well, nevertheless the tubes were easier to handle and therefore most of the joints were sealed with them.

(a) (b) (c)

Figure 3.25: Outer vertical joints filled with (a) wet mortar can depending on the material selection, application and curing either form (b) cracks or show (c) a nice surface

(a) (b)

Figure 3.26: Inner joint sealed by (a) on-glued foils or (b) elastomer tubes

After the mortar sealing hardened the segment was finished and ready to be moved from the pre-assembly field. The lifting was performed using a special spreader with whom the segments were lifted vertically at the three lifting and positioning embedments at the top of the segments. This special spreader consisted out of four U220 steel beam profiles with the quality of S355, whereby the profiles were grouped and connected in pairs forming two "I" profiles with a gap. Both compound I profiles were put together one on top of the other forming a cross connected by one threaded M36 bar with long nuts and thick load distributing steel plates (Fig. 3.27). The gaps had been used to place steel plates in order to lift the segments and the spreader itself (Fig. 3.28).

Figure 3.27: Assembly of the spreader, using four U220 profiles

Figure 3.28: The finished spreader, with all connections and lifting steel plates

3.2.3 Prototype erection

During the erection of the prototype different construction processes and structural details had to to be tested. This was done as cost saving as possible. Therefore, the prototype was not completely filled with concrete, respectively not every segment was connected load bearing with the one above respectively beneath. Even without the load bearing connection the erection of the full 16 m height was necessary to show that the segment geometry can be established enabling to arrange the segments correctly on top of each other.

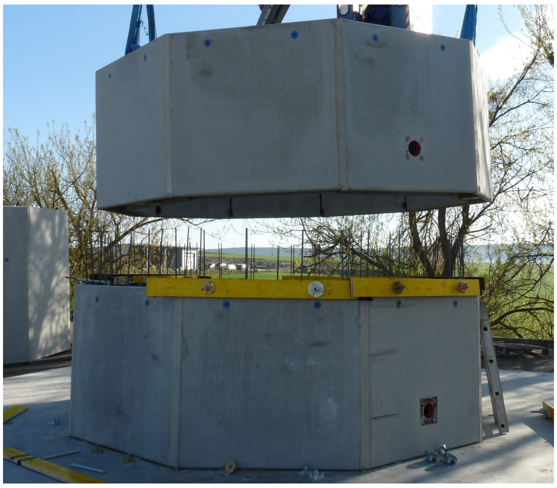
The third segment according to Fig. 3.1 was the first one that was assembled as described in Section 3.2.2. This segment served as preliminary test for the prototype, see also Janjic (2014). Therefore it was filled with concrete to test the performance of the double walls, the weld connections and the sealings. In this first try normal concrete was pumped in from the bottom.

As next the concreting of two segments and all accompanying challenges had to be tested. Therefore the segments 4 and 5 were assembled. Both segments had a height of 1.25 m. This small height was chosen in order to ease the sealing work of the horizontal joint between the two segments. After the assembly a reinforcement had to be provided in the horizontal joint, both segments had to be placed correctly over each other and the resulting joint had to be sealed.

Stiff reinforcement cages were placed from the top in the hollow space between the concrete slabs of segment 4, see Fig. 3.29. This reinforcement enables the force transmission between the vertical reinforcement of the double wall slabs once the double wall filling concrete is hardened. The length of the bars had to be chosen according to the needed bonding length.

Figure 3.29: Vertical reinforcement cages placed into segment 4

The segment 5 was threaded through the vertical joint reinforcement and positioned on segment 4 (Fig. 3.30). The upper segment was resting on three points. These point were concrete blocks embedded in every third double wall of the nonagonal segments and therefore they were equally distributed over the circumference. This distribution is on the one hand necessary to lift the segments easily and on the other it provides the equal self weight transmission.



(a)

(b)

Figure 3.30: Placement of Segment 5 on Segment 4, (a) with the out-sticking reinforcement and the partially pre-assembled horizontal joint formwork, and (b) view of the horizontal joint without formwork

In order to arrange the segments correctly, what is accomplished when the final tower axis is vertical, certain working steps are necessary. First of all the bottom segment 4 had to be standing on a completely horizontal plane. Thou any double wall production and segment assembly errors influencing the vertical tower axis were compensated by placing equalizing plates on top of the blocks thus establishing a horizontal plane. This plane was calibrated using a digital spirit level to measure to horizontality. In the horizontal joint between segment 4 and 5 cheap plastic spacers (Fig. 3.31) were used to provide the planarity. For the other horizontal joints where bigger segment were used, load bearing steel sheets had to be used as equalizing plates.

Figure 3.31: Plastic spacers on a concrete block providing the horizontal joint size of 30 mm and the needed horizontal plane

The horizontal joint must be at least 30 mm high according to EN 14992. This is needed so that the filling concrete with a maximum aggregate size of 16 mm fills the whole joint, providing the compression force transmission over the whole double wall thickness. A joint with this size have to be sealed using a formwork.

Wooden boards with an on-glued ethylene-propylene-diene monomer (EPDM) layer of 5 mm were used as formwork. The layer of EPDM should keep the joints tight even with small deviations in the desired joint geometry. The wooden boards were arranged along the circumference of the segment (inside and outside), whereby the boards are kept in place by anchor bars and plates, see Fig. 3.32 and 3.33. The anchor bars were covered by plastic tubes so that the bars could be retrieved after concreting. The edge execution of the formwork was not sophisticated, therefore the edge sealings had to be manually improved using additional EPDM stripes or plastic plates to seal them, see Fig. 3.34. At some points also polyurethane foam was used to seal the joint, see Fig. 3.35. Care was taken that no foam get in between the hollow core of the double walls. This additional foam sealing was a belt and braces approach and never used again.

Figure 3.32: Formwork for horizontal joint of one double wall consisting of wooden boards with an onglued 5 mm thick EPDM layer connected by anchor bars and plates whereby the bars between the wooden boards are covered by plastic tubes

Figure 3.33: Horizontal joint formwork inside a segment

(a) (b)

Figure 3.34: Sealing the outer edges of the formwork using (a) EPDM stripes and (b) plastic plates

(a) (b)

Figure 3.35: PU-foam was applied to any possible leaky spots, such as (a) the outer segment edges and (b) the embedded blocks at the bottom of the elements

Once the horizontal joint was sealed the concrete was filled into both segments. The concrete was brought in through an opening at the bottom of the lower segment, where a pump hose was attached by a climp collar to an adapter pipe with a valve, see Fig. 3.36. Self compacting concrete (SCC) with a quality of C40/50 was used. The filling was brought in until a height of 2.4 m. This height is 1.15 m over the level of the horizontal joint and shall test the sealings under extreme conditions, because the horizontal sealing should in practice only be loaded by a concrete pressure resulting from 0.5 m height of wet SCC concrete.

Figure 3.36: Concreting hose connected by a climp collar to an adapter tube (with a valve) bolted to a double wall

The sealings kept tight and the segments' desired filling height was reached within 50 min. There had been only a few spots where cement lime came out, like the not so perfectly sealed outer edges. These imperfections could also later be observed after the concrete was hardened and the formwork was removed, see Fig. 3.37.

Figure 3.37: Imperfections of the concreted edge geometry

The joints along the elements were sealed well, as it can be seen in Fig. 3.38. Howsoever, the optical irregularities had no influence on the bearing resistance and both segments could be removed from the pre-assembly field, see Fig. 3.39. This was the final test for the tower erection.

Figure 3.38: Concreted horizontal joint along the double wall element displaying that the EPDM equalizing layer fulfilled it's task

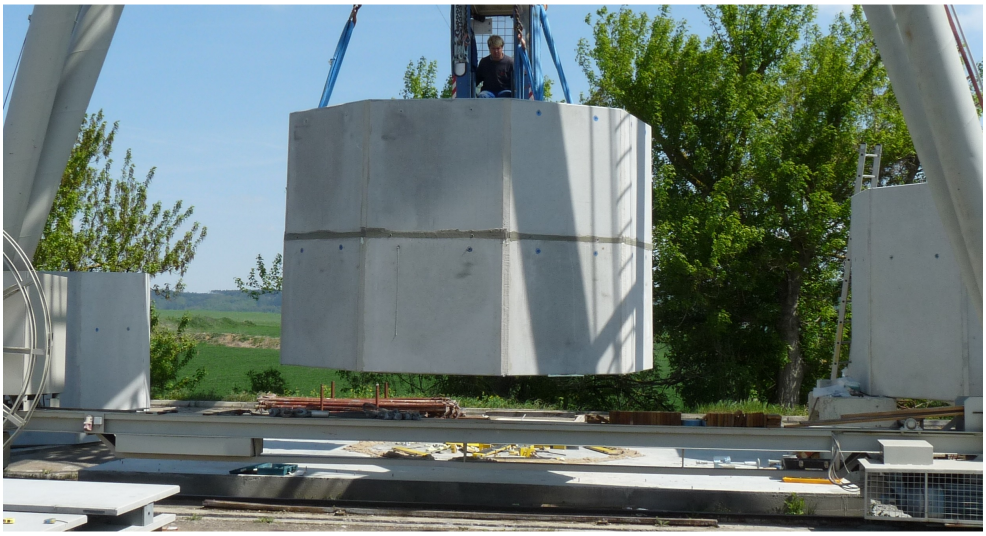


Figure 3.39: Lifting of the segments 4 and 5 after they had been concreted together

In the next step, the segments 6, 2 and 1 were assembled, whereby the segment 1 was the last in order because it was situated at the bottom of the tower prototype, see also Fig. 3.40. The assembly of the segments was done as described in section 3.2.2. The supreme segment 6 was 6 m high. Interestingly the assembly of this segment needed not more time than the assembly of the smaller ones. While the segment 2 had a standard design the lowermost segment 1 had a door opening which allowed to enter the prototype. This opening had to be equipped with formwork and an adequate bracing structure, see also Fig. 3.41(a).

Figure 3.40: From left to right: Segment 2, the lastly finished 1, the 6 m high Segment 6, Segment 3, and to the rightmost the monolithic Segment 4 and 5

(a) (b)

Figure 3.41: Segment 1 with (a) the brand door opening and (b) the horizontal joint reinforcement

The segments 1 and 2 were stacked one over another as it was done with 4 and 5. The lower segment 1 was firstly equipped with reinforcement for the horizontal joint, see Fig. 3.41(b). Then equalizing plates were positioned on top of the three lifting and positioning embedments (in segment 1 the "π" shaped steel variant was used). So that the hollow segment 2 could be lifted and placed on segment 1 using the U220 spreader, see also Fig. 3.42. In contrast to segment 5 which was small and rather light the segment 2 couldn't be set up manually in the right position. The segment was too heavy and inertial to get it by hand into the corrected position, therefore a guiding steel part was used to lead the segment to slide into the correct position.

(a) (b)

Figure 3.42: The hollow Segment 2 is (a) lifted using the U220 spreader and (b) placed on top of Segment 1

These guiding parts were two steel sheets welded perpendicular forming a "T", see Fig. 3.43 and 3.44. One sheet was facing the concrete and the perpendicular one served as stiffener. The one facing the concrete was bent with an angle of 1.5° pointing away from the walls, so that two of these parts arranged vis-à-vis serve as one guiding for the upper segment. The parts were on the one hand connected by two bolts into the concrete and on the other they were connected by two threaded bars to each other (Fig. 3.44). At least three pairs of these steel guiding parts are necessary to place the segment correctly.

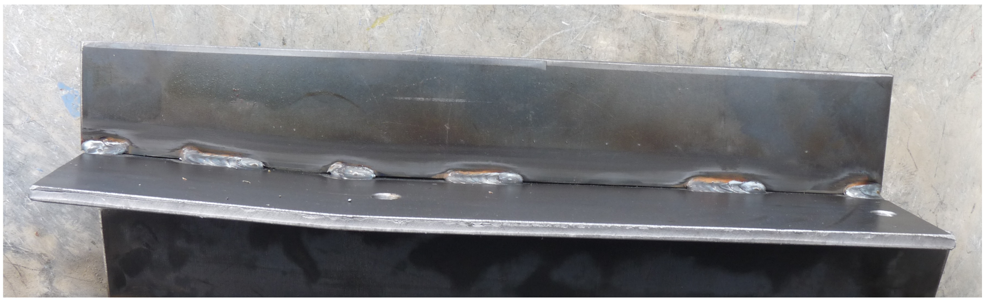


Figure 3.43: Side view of the "T"-shaped steel parts showing the 1.5° bending of the steel which will guide the upper segment

Figure 3.44: Guiding parts mounted to a segment during the positioning process

It is obvious that a guidance aid is needed to place the segments correctly. The former described one is not the optimum and intentionally it was planned to use three thorns which can be embedded in the lifting and positioning blocks. Unfortunately the somewhat inaccurate placement of the concrete blocks in some segments hindered this design. For any future segments a better placement procedure of the lifting and positioning embedments out of concrete has to be foreseen respectively only the steel variant should be used.

After the segment placement the horizontal joint formwork was mounted and both segments were filled with concrete, see Fig. 3.45. Again SCC concrete was pumped in from the bottom. A total concreting height of 4.8 m had been reached, whereby the first 3 m were pumped into the segment within 10 min. Then the concrete had 13 min to harden until the remaining 1.8 m were pumped in (Fig. 3.46). This sequence can be seen in the recordings of the deformation measurements while concreting in Fig. 3.64 and 3.65.

Figure 3.45: The two lowermost segments with the wooden formwork ready for concreting

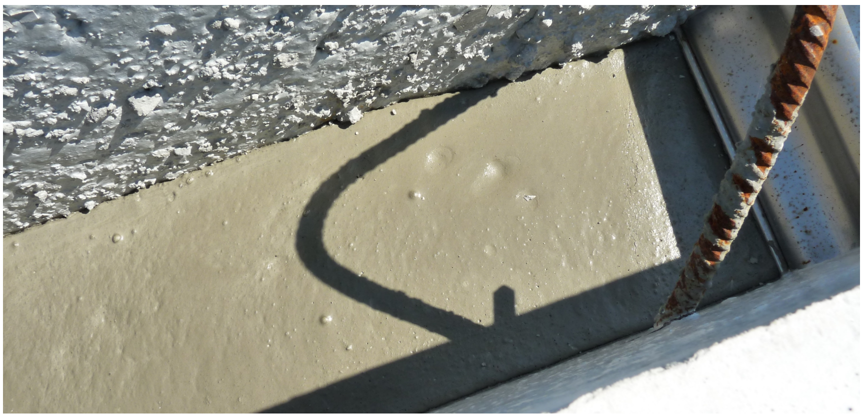


Figure 3.46: The upper surface of the self compacting concrete after the concreting process

In order to achieve a faster and easier erecting process for the following segments, additional steel guiding parts were fixed to the top of the segments 2, 3 and 5 (Fig. 3.48). Per horizontal joint four pairs of guiding parts were used. Two were placed at each side of one edge and the other two were right at the centre of a side, see also Fig. 3.47 and 3.48. This was the last step before the erection of the whole tower started.

Figure 3.47: Distribution of the steel guidance parts at a horizontal joint

Figure 3.48: All segments on ground with the guiding parts easing segment placement

A mobile crane was necessary to position the segments. As described earlier, a digital spirit level was used to set up the three sets of distance plates in every horizontal joint providing the needed horizontality. In addition a 10 mm thick elastomer plate was positioned on top of the distance plates ensuring that the force transmission between the block is as equal as possible and therefore minimising stress peaks. This was done from a crane basket (Fig. 3.49). The crane was positioned next to the pre-assembly field ensuring that the segments are inside the crane's load radius. The positioning and especially the fine adjusting was performed using a cherry picker, see also Fig. 3.50 and 3.51. The tower erection was finished with the placement of the 6 m high segment 6. It was lifted with the spreader to ensure a nearly vertical loading during positioning because it was not filled with concrete, see Fig. 3.52. Once the segment was in place the guiding parts were removed and the erection of the tower was complete, see Fig. 3.53.

Figure 3.49: Placing distance plates to set up the horizontal plain, determined with a spirit level from a crane basket

Figure 3.50: Placement of the segment 3 performed from a cherry picker

(a) (b)

Figure 3.51: Segment 4 and 5 are (a) lifted and (b) positioned

(a) (b) (c)

Figure 3.52: Segment 6 during (a) lifting, (b) positioning and at it's (c) final place

Figure 3.53: The finished 16.15 m high prototype

3.3 Double wall element manufacturing deviations

The decentralised produced double walls are the basic component of the herein proposed construction method. This implies that one has to deal with the typical production deviations occurring in a common pallet circulation plant for double walls. Therefore, each of the 54 double wall elements which were produced for the prototype have been surveyed. In the first step it has to be proven that the commercial double walls can be used and that the joints are wide enough. Thou the qualitative influence of the deviation on the erection costs are estimated, so that in the last step improvements for the production in the plant and the erection method itself can be proposed. Therefore each slab geometry like height, width and thickness, as well as double wall overall thickness and the mutual slab offsets as well as the element weight was recorded, see also Fig. 3.54. The nominal dimensions are given in Table 3.2, whereby the tables in the following subsections only provide the recorded relative deviations.

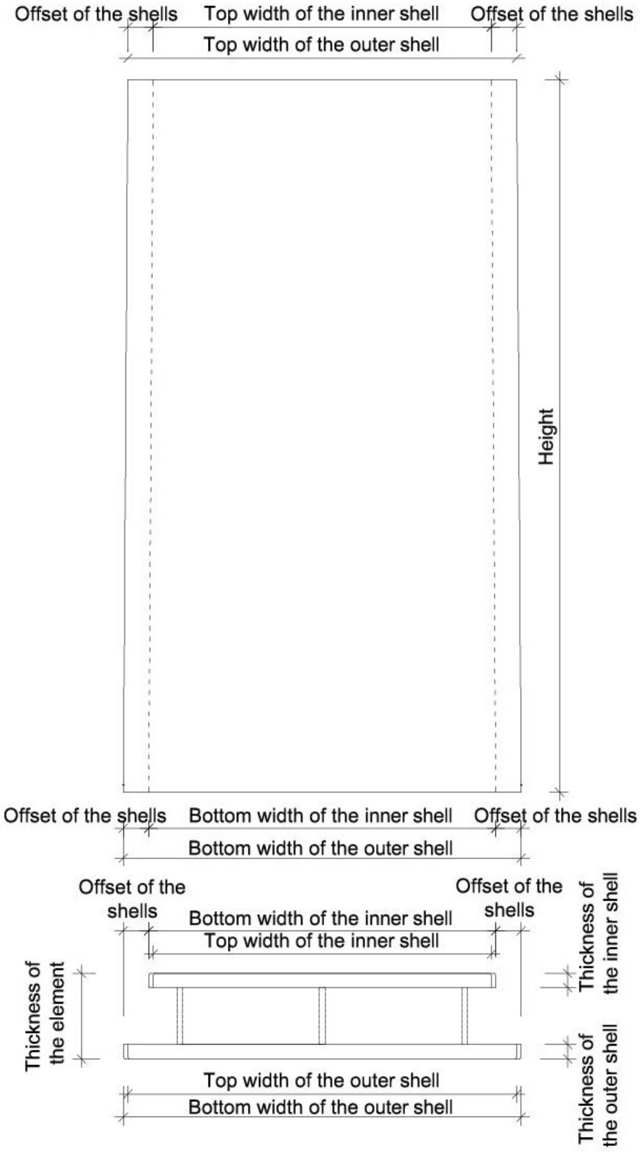


Figure 3.54: Geometrical values that have been surveyed for each of the 54 double walls of the prototype

Table 3.2: Geometrical overview of the prototype tower segments, according to Fig. 3.54

Segment	Height [m]	Width [mm]				Shell thickness [mm]	Element thickness [mm]	Offset of the shells [mm]
		Inner shell		Outer shell				
		Top	Bottom	Top	Bottom			
1	2.50	1186.0	1217.0	1364.0	1396.0	50.0	300.0	89.0
2	2.50	1151.0	1183.0	1332.0	1364.0	50.0	300.0	89.0
3	2.50	1124.0	1153.0	1302.0	1331.0	50.0	300.0	89.0
4	1.25	1108.0	1124.0	1286.0	1302.0	50.0	300.0	89.0
5	1.25	1091.0	1107.0	1270.0	1286.0	50.0	300.0	89.0
6	6.00	1091.0	1091.0	1269.0	1269.0	50.0	300.0	89.0

3.3.1 Height of the elements

The actual height of the double wall slabs enables to evaluate the accuracy of the automated lateral formwork placement. Deviations of the slab height play a rather subordinate role because the minimum horizontal joint size amounts to 30 mm. This means that the maximum coverable deviation could amount to ± 30 mm what has to be considered for the design of the horizontal joint formwork. Therefore the height of the inner and outer double wall slabs was measured at both ends and from there an average value was determined, see Table 3.3.

Table 3.3: Element height deviations

		Segment 1		Segment 2		Segment 3		Segment 4		Segment 5		Segment 6	
		Deviation		Deviation		Deviation		Deviation		Deviation		Deviation	
Element	Slab	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)
1	inner	-7.0	-0.3	3.5	0.1	-2.0	-0.1	-2.5	-0.2	-1.5	-0.1	-1.0	0.0
	outer	3.0	0.1	2.0	0.1	-1.0	0.0	2.5	0.2	1.0	0.1	-0.5	0.0
2	inner	0.5	0.0	-3.5	-0.1	-3.0	-0.1	-0.5	0.0	-3.5	-0.3	-4.0	-0.1
	outer	0.5	0.0	2.5	0.1	0.0	0.0	2.5	0.2	0.5	0.0	-1.5	0.0
3	inner	-2.0	-0.1	-1.5	-0.1	0.0	0.0	-1.0	-0.1	-3.0	-0.2	-2.5	0.0
	outer	2.0	0.1	-2.0	-0.1	-1.0	0.0	-3.0	-0.2	-2.5	-0.2	-2.0	0.0
4	inner	0.5	0.0	6.0	0.2	-2.0	-0.1	-3.0	-0.2	-2.5	-0.2	-6.5	-0.1
	outer	2.5	0.1	0.0	0.0	-1.0	0.0	-1.0	-0.1	-1.5	-0.1	-3.0	-0.1
5	inner	0.0	0.0	1.5	0.1	-1.0	0.0	-2.0	-0.2	-3.5	-0.3	-5.5	-0.1
	outer	-1.0	0.0	2.5	0.1	-1.0	0.0	-2.5	-0.2	0.0	0.0	-1.5	0.0
6	inner	1.0	0.0	-1.5	-0.1	-1.0	0.0	-3.0	-0.2	-2.5	-0.2	-4.0	-0.1
	outer	2.0	0.1	1.5	0.1	-1.0	0.0	-4.0	-0.3	-4.0	-0.3	-2.0	0.0
7	inner	2.0	0.1	1.0	0.0	0.0	0.0	-2.0	-0.2	-2.5	-0.2	-1.0	0.0
	outer	-0.5	0.0	1.0	0.0	-1.0	0.0	-3.0	-0.2	-1.0	-0.1	-2.0	0.0
8	inner	-0.5	0.0	-2.5	-0.1	0.0	0.0	-1.0	-0.1	-1.5	-0.1	-2.0	0.0
	outer	1.5	0.1	-0.5	0.0	-5.0	-0.2	-3.0	-0.2	-2.5	-0.2	-2.0	0.0
9	inner	1.5	0.1	-1.5	-0.1	-1.0	0.0	-1.5	-0.1	-1.0	-0.1	-1.0	0.0
	outer	1.5	0.1	0.5	0.0	-1.0	0.0	-3.5	-0.3	-3.5	-0.3	-3.0	-0.1
Total average	inner	-0.4	0.0	0.2	0.0	-1.1	0.0	-1.8	-0.1	-2.4	-0.2	-3.1	-0.1
	outer	1.3	0.1	0.8	0.0	-1.3	-0.1	-1.7	-0.1	-1.5	-0.1	-1.9	0.0

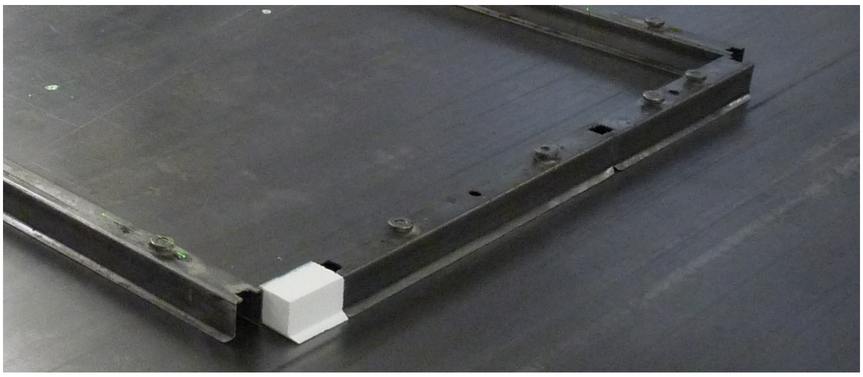


Figure 3.55: An inaccurate placement of the formwork and the manually on-glued styrofoam parts can result in manufacture deviations of the double wall slab geometry

Interestingly the deviations are very small and rather insignificant, indicating that the automated formwork placement is working well. The biggest difference was measured for the inner slab of segment 1, which was 7 mm too short. What can be led back to a manual placement error of a styrofoam formwork piece as it can be seen in (Fig. 3.55). Therefore it is recommended that these manually placed formwork pieces are used as few as possible.

3.3.2 Width of the elements

The width of the elements is one of the factors governing the needed vertical joint width, whereby a small joint width is desired because it governs the design of the joint sealing type and has an influence on the connections of the double walls. In the present case the maximum deviation of the slab width can amount to the joint width divided with tangent of the half inner polygon angle, resulting in an acceptable error of approximately 10.6 mm. Whereby this value is de- or increased depending on the deviation of the horizontal shift between both double wall slabs which is further discussed in subsection 3.3.5.

The observed deviation of the width were in the same magnitude as the ones of the height, what attributes to the fact that the same working processes are causing them, see Table 3.4 and 3.5. However the highest value that was observed amounted to 11 mm at the outer shell of element 3 and 6 of segment 4. This exceeds the aforementioned value about 0.4 mm, nonetheless it was possible to arrange the segment 4. The deviation is in fact higher than the deviation of the height, what is due to the longer lateral formwork profiles - compared to the ones at the top or bottom - where the fixing magnets sometimes were not strong enough to keep the formwork in place, see Fig. 3.56

Figure 3.56: The weak formwork fixing magnets cause manufacture errors in the element's outer shape

This observation is also visible if the average deviations of height (0.4 mm) are compared with the ones of the width (2.5 mm). One of the most noticeable deviations is the bottom width of the outer shell of element 3 of segment 4 which is shorter by 68mm. This huge deviation occurred because one edge of the slab was damaged during transport.

Table 3.4: Element width deviations

		Segment 1				Segment 2				Segment 3			
Element	Section	Inner slab Deviation		Outer slab Deviation		Inner slab Deviation		Outer slab Deviation		Inner slab Deviation		Outer slab Deviation	
		(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)
1	bottom	0.0	0.0	4.0	0.3	4.0	0.3	4.0	0.3	7.0	0.6	-6.0	-0.5
	top	1.0	0.1	1.0	0.1	-1.0	-0.1	4.0	0.3	-1.0	-0.1	-5.0	-0.4
2	bottom	3.0	0.2	4.0	0.3	1.0	0.1	4.0	0.3	2.0	0.2	-1.0	-0.1
	top	3.0	0.3	5.0	0.4	-2.0	-0.2	2.0	0.2	2.0	0.2	4.0	0.3
3	bottom	4.0	0.3	3.0	0.2	5.0	0.4	-1.0	-0.1	2.0	0.2	3.0	0.2
	top	2.0	0.2	1.0	0.1	0.0	0.0	0.0	0.0	1.0	0.1	2.0	0.2
4	bottom	5.0	0.4	7.0	0.5	5.0	0.4	0.0	0.0	4.0	0.3	-3.0	-0.2
	top	3.0	0.3	5.0	0.4	-3.0	-0.3	5.0	0.4	2.0	0.2	-2.0	-0.2
5	bottom	5.0	0.4	7.0	0.5	3.0	0.3	7.0	0.5	4.0	0.3	-1.0	-0.1
	top	2.0	0.2	2.0	0.1	-2.0	-0.2	1.0	0.1	3.0	0.3	-4.0	-0.3
6	bottom	2.0	0.2	6.0	0.4	-2.0	-0.2	1.0	0.1	5.0	0.4	-3.0	-0.2
	top	1.0	0.1	5.0	0.4	-2.0	-0.2	0.0	0.0	1.0	0.1	-3.0	-0.2
7	bottom	3.0	0.2	6.0	0.4	7.0	0.6	5.0	0.4	3.0	0.3	-1.0	-0.1
	top	2.0	0.2	3.0	0.2	0.0	0.0	4.0	0.3	1.0	0.1	-4.0	-0.3
8	bottom	1.0	0.1	5.0	0.4	3.0	0.3	8.0	0.6	-3.0	-0.3	-6.0	-0.5
	top	3.0	0.3	2.0	0.1	1.0	0.1	7.0	0.5	-2.0	-0.2	1.0	0.1
9	bottom	4.0	0.3	3.0	0.2	2.0	0.2	5.0	0.4	-2.0	-0.2	-6.0	-0.5
	top	0.0	0.0	2.0	0.1	-1.0	-0.1	1.0	0.1	-2.0	-0.2	-2.0	-0.2
Total average	bottom	3.0	0.2	5.0	0.4	3.1	0.3	3.7	0.3	2.4	0.2	-2.7	-0.2
	top	1.9	0.2	2.9	0.2	-1.1	-0.1	2.7	0.2	0.6	0.0	-1.4	-0.1

Table 3.5: Element width deviations

		Segment 4				Segment 5				Segment 6			
Element	Section	Inner slab Deviation		Outer slab Deviation		Inner slab Deviation		Outer slab Deviation		Inner slab Deviation		Outer slab Deviation	
		(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)
1	bottom	-8.0	-0.7	3.0	0.2	2.0	0.2	4.0	0.3	-2.0	-0.2	2.0	0.2
	top	1.0	0.1	8.0	0.6	1.0	0.1	1.0	0.1	-1.0	-0.1	0.0	0.0
2	bottom	1.0	0.1	2.0	0.2	-3.0	-0.3	4.0	0.3	-1.0	-0.1	-2.0	-0.2
	top	9.0	0.8	2.0	0.2	-1.0	-0.1	4.0	0.3	0.0	0.0	-3.0	-0.2
3	bottom	1.0	0.1	-68.0	-5.2	-1.0	-0.1	8.0	0.6	-3.0	-0.3	1.0	0.1
	top	8.0	0.7	11.0	0.9	0.0	0.0	4.0	0.3	-2.0	-0.2	2.0	0.2
4	bottom	0.0	0.0	3.0	0.2	-7.0	-0.6	6.0	0.5	0.0	0.0	-4.0	-0.3
	top	4.0	0.4	3.0	0.2	-2.0	-0.2	5.0	0.4	1.0	0.1	-1.0	-0.1
5	bottom	-3.0	-0.3	9.0	0.7	-1.0	-0.1	6.0	0.5	1.0	0.1	3.0	0.2
	top	5.0	0.5	5.0	0.4	2.0	0.2	2.0	0.2	4.0	0.4	1.0	0.1
6	bottom	-2.0	-0.2	7.0	0.5	-3.0	-0.3	4.0	0.3	4.0	0.4	-3.0	-0.2
	top	-2.0	-0.2	11.0	0.9	-4.0	-0.4	2.0	0.2	4.0	0.4	-4.0	-0.3
7	bottom	-3.0	-0.3	5.0	0.4	1.0	0.1	6.0	0.5	0.0	0.0	1.0	0.1
	top	-2.0	-0.2	1.0	0.1	7.0	0.6	3.0	0.2	1.0	0.1	2.0	0.2
8	bottom	-3.0	-0.3	-3.0	-0.2	-1.0	-0.1	5.0	0.4	-4.0	-0.4	-5.0	-0.4
	top	5.0	0.5	7.0	0.5	-1.0	-0.1	7.0	0.6	4.0	0.4	-5.0	-0.4
9	bottom	-2.0	-0.2	1.0	0.1	-5.0	-0.5	0.0	0.0	-1.0	-0.1	5.0	0.4
	top	3.0	0.3	3.0	0.2	-4.0	-0.4	1.0	0.1	-2.0	-0.2	0.0	0.0
Total average	bottom	-2.1	-0.2	-4.6	-0.3	-2.0	-0.2	4.8	0.4	-0.7	-0.1	-0.2	0.0
	top	3.4	0.3	5.7	0.4	-0.2	0.0	3.2	0.3	1.0	0.1	-0.9	-0.1

Some recommendations regarding the accuracy of the width can be made. First of all the number of manually placed styrofoam pieces have to be reduced, what can be achieved if longer standard profiles are used. Secondly it has to be checked, if the formwork fixing magnets could be improved or alternatively any additional ones can be placed. As last it is recommended that a calibre is used to check the geometry after concreting and vibrating so that corrections can be made.

3.3.3 Thickness of the slabs

The weight of the double walls and therefore the slab thickness shall be as low as possible while a certain bearing resistance shall be provided. Therefore the slab thickness was designed to be 50 mm. Interestingly the biggest deviations were encountered for the slab thickness, reaching a maximum of 19.9 mm (39.8 %) respectively an overall average deviation of 7.5 mm (14.9 %).

This huge deviation has it's origin in the way the concrete is poured into the formwork. The concrete can be placed automated or manually. The automated process is pretty precise in terms of the concrete volume that is placed but it does not take care that all embedments like the lifting hooks are fully covered in concrete. Therefore, at these vital spots concrete has to be manually poured in, causing thicker concrete slabs, see Fig. 3.57. If the workers do not take care and pour too much concrete at the vital spots, it will result in too thick slabs. This issue was encountered at segment 1 (maximal average deviation of 17.1 mm) and therefore care was taken at the production of segment 2 (maximal average deviation 3.8 mm).

Figure 3.57: Additional concrete is poured onto the lifting hooks in order to provide a better anchoring, thus increasing the thickness of the slabs

As the influencing factor is the labour work, the final accuracy depends on the motivation of the workers themselves. This fact is reflected in the deviations of the segments produced after segment 2, which never achieved the mostly optimal values of segment 2. Whereby it has to be mentioned that some elements for segment 2 had at some spots a thickness lesser than 50 mm what shall not happen at vital spots.

Table 3.6: Slab thickness deviations

		Segment 1				Segment 2				Segment 3			
Element	Slab	Deviation At the top (mm)	(%)	Deviation At the bottom (mm)	(%)	Deviation At the top (mm)	(%)	Deviation At the bottom (mm)	(%)	Deviation At the top (mm)	(%)	Deviation At the bottom (mm)	(%)
1*	inner	18.2	36.3	13.4	26.8	1.5	3.0	-0.5	-1.0	9.0	18.0	0.8	1.5
	outer	10.9	21.8	14.2	28.3	7.0	14.0	3.5	7.0	5.8	11.5	6.3	12.5
2	inner	16.6	33.2	11.2	22.4	2.5	5.0	-2.0	-4.0	11.5	23.0	7.0	14.0
	outer	3.5	6.9	7.9	15.7	6.0	12.0	6.0	12.0	4.5	9.0	7.5	15.0
3	inner	15.1	30.1	8.6	17.2	0.5	1.0	-1.0	-2.0	7.5	15.0	3.0	6.0
	outer	4.5	9.1	8.2	16.4	6.5	13.0	0.5	1.0	0.5	1.0	-1.0	-2.0
4*	inner	19.9	39.8	14.3	28.7	-1.5	-3.0	-1.5	-3.0	7.5	15.0	4.0	8.0
	outer	11.0	21.9	6.7	13.3	3.0	6.0	1.5	3.0	-1.5	-3.0	0.5	1.0
5	inner	16.2	32.3	11.4	22.8	-2.0	-4.0	0.5	1.0	9.5	19.0	3.5	7.0
	outer	6.9	13.8	5.9	11.8	2.0	4.0	3.5	7.0	5.5	11.0	7.0	14.0
6	inner	17.2	34.4	10.0	20.0	4.5	9.0	7.0	14.0	13.0	26.0	5.5	11.0
	outer	8.9	17.7	6.8	13.5	0.5	1.0	1.5	3.0	6.0	12.0	5.0	10.0
7*	inner	17.9	35.7	17.7	35.4	7.0	14.0	7.5	15.0	14.5	29.0	7.0	14.0
	outer	9.1	18.2	9.8	19.7	3.5	7.0	0.5	1.0	1.0	2.0	7.5	15.0
8	inner	15.5	30.9	15.5	31.0	3.0	6.0	6.0	12.0	9.0	18.0	4.0	8.0
	outer	5.3	10.6	6.6	13.1	2.0	4.0	0.0	0.0	1.5	3.0	0.5	1.0
9	inner	17.5	35.1	12.8	25.7	5.5	11.0	4.0	8.0	5.5	11.0	3.5	7.0
	outer	7.1	14.2	9.4	18.8	4.0	8.0	3.0	6.0	7.5	15.0	0.0	0.0
Total average	inner	17.1	34.2	12.8	25.5	2.3	4.7	2.2	4.4	9.7	19.3	4.3	8.5
	outer	7.5	14.9	8.4	16.7	3.8	7.7	2.2	4.4	3.4	6.8	3.7	7.4

Table 3.7: Slab thickness deviations

		Segment 4				Segment 5				Segment 6					
Element	Slab	Deviation At the top (mm)	(%)	Deviation At the bottom (mm)	(%)	Deviation At the top (mm)	(%)	Deviation At the bottom (mm)	(%)	Deviation At the top (mm)	(%)	Deviation At the middle (mm)	(%)	Deviation At the bottom (mm)	(%)
1*	inner	14.4	28.8	11.0	22.0	8.4	16.8	13.8	27.5	-5.4	10.8	10.0	20.0	9.2	18.4
	outer	3.6	7.1	6.2	12.5	6.1	12.1	7.4	14.8	4.7	9.3	3.9	7.7	8.9	17.8
2	inner	14.4	28.8	12.7	25.3	12.4	24.7	10.9	21.8	8.3	16.5	7.0	14.0	8.8	17.6
	outer	2.1	4.3	1.8	3.5	6.0	12.0	5.7	11.4	7.5	15.0	5.9	11.7	1.5	3.1
3	inner	16.2	32.4	13.5	27.1	14.0	28.0	15.8	31.6	9.6	19.2	10.0	20.0	9.4	18.9
	outer	9.8	19.7	8.7	17.3	8.9	17.7	0.6	1.2	1.5	2.9	4.0	7.9	6.0	11.9
4*	inner	12.8	25.7	14.4	28.8	18.7	37.3	18.0	35.9	3.4	6.9	8.0	16.0	9.8	19.5
	outer	8.9	17.9	9.4	18.8	4.7	9.4	5.1	10.2	3.3	6.7	-0.1	-0.2	10.2	20.3
5	inner	16.5	33.1	17.6	35.1	13.3	26.5	12.6	25.1	7.5	15.0	8.5	17.0	6.8	13.6
	outer	1.9	3.9	4.9	9.9	6.9	13.7	7.8	15.5	-0.6	1.2	3.4	6.7	10.9	21.8
6	inner	18.9	37.7	16.1	32.2	12.2	24.3	10.2	20.3	16.6	33.3	7.5	15.0	17.2	34.4
	outer	3.5	6.9	5.9	11.8	6.3	12.5	8.0	16.0	15.7	31.4	11.0	22.1	-1.2	-2.4
7*	inner	14.1	28.1	16.1	32.2	15.2	30.4	15.3	30.5	4.7	9.3	3.5	7.0	8.4	16.9
	outer	1.5	3.0	5.9	11.9	4.1	8.2	8.8	17.5	6.4	12.9	1.9	3.8	3.8	7.6
8	inner	12.3	24.5	16.4	32.8	12.3	24.5	13.5	27.0	12.2	24.3	9.5	19.0	12.0	24.0
	outer	6.0	12.0	5.2	10.5	6.1	12.1	1.4	2.8	9.3	18.5	1.5	3.0	-19.2	-38.4
9	inner	16.3	32.6	14.9	29.8	13.9	27.8	13.3	26.5	7.4	14.9	6.0	12.0	4.2	8.3
	outer	8.2	16.5	5.9	11.9	9.8	19.7	6.0	12.0	4.5	8.9	3.5	6.9	8.9	17.8
Total average	inner	15.1	30.2	14.7	29.5	13.3	26.7	13.7	27.4	7.1	16.7	7.8	15.6	9.5	19.1
	outer	5.1	10.1	6.0	12.0	6.5	13.0	5.6	11.3	5.8	11.9	3.9	7.7	3.3	6.6

3.3.4 Weight of the elements

The weight of the elements is important for the transportation and handling onsite. This can directly influence the needed crane size on the job-site and therefore also the erection costs. The weight was measured using four pressure load cells placed on steel plates and a load distributing wooden plate on which the double walls were placed, see Fig. 3.58. The load cells were connected to a converter sending it's data to a computer.

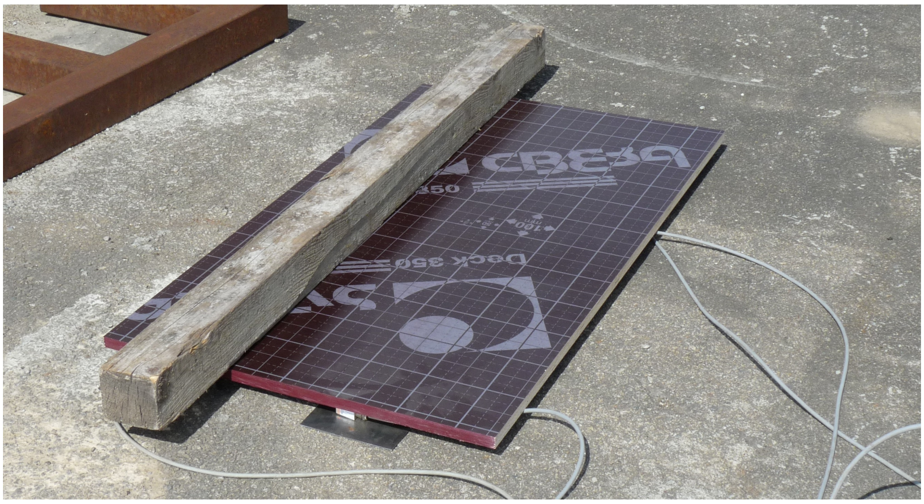


Figure 3.58: Weight measuring setup consisting of a wooden plate distributing the load on 4 pressure load cells

As former mentioned in subsection 3.3.3 the double wall slab thickness showed the biggest deviations to the nominal values, therefore resulting in heavier double walls, see Table 3.9. The nominal values for each double wall of the prototype tower are given in Table 3.8. Whereby each segment had in terms of the weight two different types of elements. The ones with lifting and positioning embedments (heavier element) and the ones without, whereby the pump opening was neglected in this listing. In the most extreme case of segment 1 element 1 the planned weight was exceeded by 245.4 kg (28.7 %), see Table 3.9. This is a very extreme value but still within range of the safety factor for the dead load ($\gamma_s = 1.35$). Therefore it is recommended to gather more weight data and in the future recommendations for a lower safety factor for industrialised produced precast elements can be made. The increase of the accuracy can be achieved if the recommendations from subsection 3.3.3 are considered.

Table 3.8: Planned element weights

Weight of the elements [kg]						
Element	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Segment 6
1 / 4 / 7	853.8	860.0	841.1	441.0	444.0	1830.5
2 / 3 / 5 / 6 / 8 / 9	806.7	786.0	767.5	368.4	371.0	1770.0

Table 3.9: Element weight deviations

Element	Segment 1 Deviation (kg)	Segment 1 Deviation (%)	Segment 2 Deviation (kg)	Segment 2 Deviation (%)	Segment 3 Deviation (kg)	Segment 3 Deviation (%)	Segment 4 Deviation (kg)	Segment 4 Deviation (%)	Segment 5 Deviation (kg)	Segment 5 Deviation (%)	Segment 6 Deviation (kg)	Segment 6 Deviation (%)
1*	245.4	28.7	16.4	1.9	81.2	9.7	56.0	12.7	51.0	11.5	220.5	12.0
2	183.2	22.7	65.0	8.3	139.0	18.1	72.6	19.7	76.0	20.5	156.4	8.8
3	188.3	23.3	35.5	4.5	42.0	5.5	104.6	28.4	77.0	20.8	253.0	14.3
4*	227.9	26.7	-30.7	-3.6	64.9	7.7	75.0	17.0	54.0	12.2	168.0	9.2
5	167.1	20.7	30.7	3.9	123.0	16.0	94.6	25.7	82.0	22.1	245.9	13.9
6	161.1	20.0	67.5	8.6	178.5	23.3	97.6	26.5	77.0	20.8	104.4	5.9
7*	218.9	25.6	54.8	6.4	138.9	16.5	66.0	15.0	54.0	12.2	170.5	9.3
8	146.1	27.5	66.7	8.5	53.5	7.0	87.6	23.8	71.0	19.1	252.0	14.2
9	210.5	26.1	51.8	6.6	71.5	9.3	95.6	26.0	88.0	23.7	184.8	10.4
Total average	194.3	24.6	39.7	5.0	99.2	12.6	83.3	21.6	70.0	18.1	195.1	10.9
Total weight	1748.5	24.5	357.7	4.9	892.5	12.5	749.6	21.2	630.0	17.7	1755.6	10.9

*double walls with lifting and positioning embedments

3.3.5 Offset of the concrete slabs

The deviation of the slab offset is important for the design of the vertical joint and as mentioned in subsection 3.3.2 it's impact has to be rated together with the deviation of the element width. The deviation of the offset is depending on the merging process of both double wall slabs. Both, translations and rotations regarding the nominal geometry have been observed. Some of the errors occurred due to embedments, which connect both slabs like Kappema waves and lifting embedments. They sometimes collided with the reinforcement of the lower plate in which the upper was pushed in. This embedment then shifted the upper slab.

It is again clearly visible that the segment 1, which was produced at first showed an 11.5 mm bigger offset as planned, see Table 3.10 and 3.11. This was clearly improved at segment 2. The deviations can on the one hand be minimised by taking care that the probability of the collision between embedments and reinforcement of both slabs are minimised. On the other hand it is possible to measure the offset while one of the slabs is still filled with wet concrete (see Fig. 3.59) and therefore one special distance adjusting equipment could be designed. This adjusting equipment could be fixed to the steel pallet using magnets and therefore helping to adjust the upper slab into the correct position.

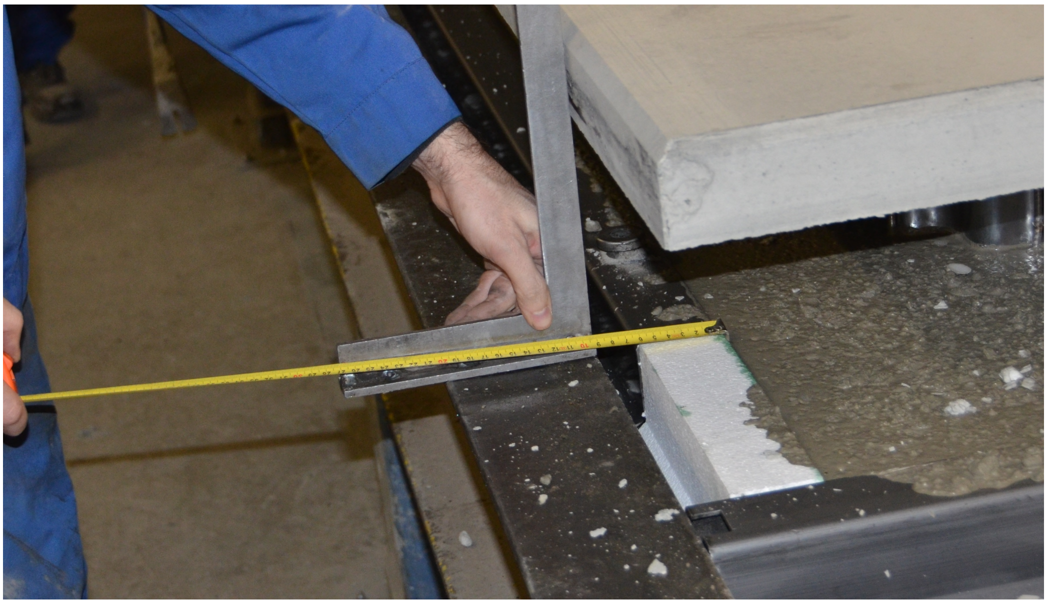


Figure 3.59: Measuring the offset of the elements at the production line while the lower slab is filled with not yet hardened concrete

Table 3.10: Deviations of slab offset for segments 1,2,3

		Segment 1				Segment 2				Segment 3			
		Deviation				Deviation				Deviation			
Element	Section	left		right		left		right		left		right	
		(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)
1	bottom	6.5	7.3	-4.5	-5.0	2.0	2.2	-1.0	-1.1	-3.5	-3.9	-4.0	-4.5
	top	10.0	11.2	-10.0	-11.2	1.0	1.1	1.0	1.1	2.0	2.2	-4.0	-4.5
2	bottom	3.5	3.9	-2.5	-2.8	3.0	3.3	0.0	0.0	-5.0	-5.6	0.0	0.0
	top	-2.0	-2.2	1.0	1.1	5.0	5.5	-2.0	-2.2	-3.0	-3.4	2.0	2.2
3	bottom	4.5	5.0	-5.5	-6.1	2.0	2.2	-5.0	-5.5	-8.0	-9.0	-3.5	-3.9
	top	3.0	3.4	-2.0	-2.2	-1.0	-1.1	0.0	0.0	-1.0	-1.1	-4.5	-5.1
4	bottom	1.5	1.7	-9.5	-10.6	0.0	0.0	3.0	3.3	-4.0	-4.5	-4.0	-4.5
	top	1.0	1.1	1.0	1.1	3.0	3.3	-6.0	-6.6	-2.0	-2.2	-4.0	-4.5
5	bottom	2.5	2.8	-6.5	-7.3	4.0	4.4	-3.0	-3.3	-9.0	-10.1	-4.0	-4.5
	top	3.0	3.4	-5.0	-5.6	-1.0	-1.1	1.0	1.1	-7.0	-7.9	0.0	0.0
6	bottom	-4.5	-5.0	-5.5	-6.1	6.0	6.6	-4.0	-4.4	-3.0	-3.4	-4.0	-4.5
	top	-5.0	-5.6	2.0	2.2	-1.0	-1.1	1.0	1.1	-5.0	-5.6	-4.0	-4.5
7	bottom	6.5	7.3	-9.5	-10.6	3.0	3.3	-2.0	-2.2	-2.0	-2.2	-4.0	-4.5
	top	5.0	5.6	-6.0	-6.7	1.0	1.1	2.0	2.2	-6.0	-6.7	-2.0	-2.2
8	bottom	-2.5	-2.8	-6.5	-7.3	5.0	5.5	-1.0	-1.1	1.0	1.1	-4.0	-4.5
	top	1.0	1.1	-7.0	-7.9	1.0	1.1	0.0	0.0	3.0	3.4	1.0	1.1
9	bottom	4.5	5.0	-11.5	-12.8	3.0	3.3	-1.0	-1.1	-4.0	-4.5	-4.0	-4.5
	top	2.0	2.2	-3.0	-3.4	-1.0	-1.1	1.0	1.1	-4.0	-4.5	1.0	1.1
Total average	bottom	2.5	2.8	-6.8	-7.6	3.1	3.4	-1.6	-1.7	-4.2	-4.7	-3.5	-3.9
	top	2.0	2.2	-3.2	-3.6	0.8	0.9	-0.2	-0.2	-2.6	-2.9	-1.6	-1.8

Table 3.11: Deviations of slab offset for segments 4,5,6

		Segment 4				Segment 5				Segment 6			
		Deviation				Deviation				Deviation			
Element	Section	left		right		left		right		left		right	
		(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)
1	bottom	5.0	5.5	-9.0	-9.9	-1.0	-1.1	-4.0	-4.4	-1.0	-1.1	-9.0	-10.1
	top	7.0	7.7	-4.0	-4.4	0.0	0.0	-4.0	-4.4	1.0	1.1	-1.0	-1.1
2	bottom	3.0	3.3	-3.0	-3.3	-4.0	-4.4	1.0	1.1	1.0	1.1	1.0	1.1
	top	-5.0	-5.5	-4.0	-4.4	-3.0	-3.3	4.0	4.4	-3.0	-3.4	0.0	0.0
3	bottom	-4.0	-4.4	-5.0	-5.5	-3.0	-3.3	-1.0	-1.1	3.0	3.4	9.0	10.1
	top	-11.0	-12.1	-4.0	-4.4	-2.0	-2.2	3.0	3.3	-1.0	-1.1	5.0	5.6
4	bottom	5.0	5.5	-6.0	-6.6	6.0	6.6	-3.0	-3.3	-3.0	-3.4	-1.0	-1.1
	top	7.0	7.7	-3.0	-3.3	-3.0	-3.3	3.0	3.3	1.0	1.1	-1.0	-1.1
5	bottom	4.0	4.4	-5.0	-5.5	-4.0	-4.4	1.0	1.1	4.0	4.5	-1.0	-1.1
	top	-3.0	-3.3	1.0	1.1	-2.0	-2.2	0.0	0.0	3.0	3.4	-1.0	-1.1
6	bottom	6.0	6.6	-7.0	-7.7	0.0	0.0	-5.0	-5.5	1.0	1.1	-5.0	-5.6
	top	-2.0	-2.2	-1.0	-1.1	0.0	0.0	-4.0	-4.4	8.0	9.0	-1.0	-1.1
7	bottom	3.0	3.3	-5.0	-5.5	-2.0	-2.2	-5.0	-5.5	-2.0	-2.2	4.0	4.5
	top	0.0	0.0	2.0	2.2	-5.0	-5.5	-2.0	-2.2	-5.0	-5.6	1.0	1.1
8	bottom	-9.0	-9.9	-2.0	-2.2	2.0	2.2	-7.0	-7.7	-6.0	-6.7	-3.0	-3.4
	top	-7.0	-7.7	8.0	8.8	-1.0	-1.1	-1.0	-1.1	2.0	2.2	-2.0	-2.2
9	bottom	-7.0	-7.7	1.0	1.1	-5.0	-5.5	-4.0	-4.4	1.0	1.1	2.0	2.2
	top	-10.0	-11.0	-1.0	-1.1	-7.0	-7.7	8.0	8.8	-2.0	-2.2	2.0	2.2
Total average	bottom	0.7	0.7	-4.6	-5.0	-1.2	-1.3	-3.0	-3.3	-0.2	-0.2	-0.3	-0.4
	top	-2.7	-2.9	-0.7	-0.7	-2.6	-2.8	0.8	0.9	0.4	0.5	0.2	0.2

3.3.6 Thickness of the elements

The thickness of the elements is provided in most cases by the interconnecting Kappema embedments (Fig. 3.60). These 30 cm long steel elements can be bent or moved during the merging process of the two slabs. These deformations caused a reduced thickness in the case of segments 1, 2 and 6. However each third element received either steel or concrete lifting and positioning embedments, which are more solid than the remaining connecting elements and therefore they also should maintain the planned thickness of the element as well. In some cases (segment 3) the elements were thicker than planned (Table 3.12). This phenomenon could be caused either by a concrete aggregate stuck between the formwork pallet and the embedded blocks, or the interconnecting elements were stuck on the reinforcement. Therefore in the future placements the area of the blocks was left unfilled, to ensure nothing can stick under the embedments. For segments 4 and 5 the average deviations show that both shells are not parallel to each other. At the bottom of the elements the thickness is thinner, and at the top it's thicker, compared to the nominal dimensions.

Furthermore the segments are assembled according to their outer shell, in order to provide a perfect outer segment geometry. This means, that the occurring deviations have to be covered by geometrical tolerances inside the tower. A significantly thinner or thicker element could make the sealing of the inner horizontal joints more difficult. Although the deviations are mostly in the acceptable range of the tolerances (1 %), it is advised to pay more attention during the merging process to avoid any unforeseen difficulties on the construction site. Furthermore the interconnecting elements have to be planned and positioned in a way, that they don't collide with the reinforcement mat in the concrete slabs.

Table 3.12: Overall double wall element thickness for all six segments

		Segment 1		Segment 2		Segment 3		Segment 4		Segment 5		Segment 6	
Element	Section	Deviation		Deviation		Deviation		Deviation		Deviation		Deviation	
		(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)
1*	bottom	-0.5	-0.2	-2.5	-0.8	1.0	0.3	0.5	0.2	-2.5	-0.8	1.5	0.5
	middle											-2.5	-0.8
	top	-2.0	-0.7	-7.5	-2.5	0.3	0.1	1.0	0.3	0.5	0.2	-1.5	-0.5
2	bottom	-1.0	-0.3	-3.5	-1.2	3.5	1.2	0.0	0.0	-1.0	-0.3	-2.0	-0.7
	middle											-1.5	-0.5
	top	-0.5	-0.2	-8.5	-2.8	-0.5	-0.2	1.0	0.3	0.0	0.0	-1.0	-0.3
3	bottom	-2.5	-0.8	-5.5	-1.8	1.0	0.3	0.0	0.0	-2.0	-0.7	-1.5	-0.5
	middle											0.0	0.0
	top	-0.5	-0.2	-9.5	-3.2	1.5	0.5	0.5	0.2	3.5	1.2	0.9	0.3
4*	bottom	0.5	0.2	-1.0	-0.3	2.5	0.8	-1.0	-0.3	4.5	1.5	0.5	0.2
	middle											-1.0	-0.3
	top	0.5	0.2	-1.5	-0.5	1.0	0.3	7.5	2.5	3.5	1.2	1.5	0.5
5	bottom	-1.5	-0.5	-4.0	-1.3	1.5	0.5	-2.5	-0.8	-4.0	-1.3	-1.5	-0.5
	middle											-1.5	-0.5
	top	1.0	0.3	-4.5	-1.5	0.0	0.0	5.0	1.7	7.5	2.5	-1.0	-0.3
6	bottom	0.0	0.0	-3.0	-1.0	0.5	0.2	-2.5	-0.8	-2.0	-0.7	-2.5	-0.8
	middle											1.0	0.3
	top	-0.5	-0.2	0.0	0.0	-0.5	-0.2	4.5	1.5	8.0	2.7	-0.5	-0.2
7*	bottom	-0.5	-0.2	-3.5	-1.2	6.0	2.0	0.5	0.2	6.5	2.2	0.5	0.2
	middle											-0.5	-0.2
	top	-1.0	-0.3	0.0	0.0	7.5	2.5	3.5	1.2	8.0	2.7	-1.5	-0.5
8	bottom	0.0	0.0	-3.5	-1.2	1.5	0.5	-2.0	-0.7	1.0	0.3	0.0	0.0
	middle											-1.5	-0.5
	top	0.0	0.0	-2.0	-0.7	0.0	0.0	-0.5	-0.2	2.0	0.7	0.5	0.2
9	bottom	-2.5	-0.8	-3.5	-1.2	4.5	1.5	-4.5	-1.5	1.0	0.3	-1.0	-0.3
	middle											-0.5	-0.2
	top	1.0	0.3	-1.0	-0.3	1.5	0.5	4.5	1.5	3.0	1.0	1.0	0.3
Total average	bottom	-0.9	0.3	-3.3	1.1	2.4	0.8	-1.3	-0.4	0.2	0.1	-0.7	0.4
	middle											-0.9	0.4
	top	-0.2	-0.1	-3.8	1.3	1.2	0.4	3.0	1.0	4.0	1.3	-0.2	0.4

Figure 3.60: Kappema elements - standing on pallet - that are responsible for the interconnection of the shells and the thickness of the element

3.4 Segment deformation during concreting of the segments

Displacement transducers were mounted to certain segments in order to estimate the effects of the pressure caused by the concreting process. For this examination so called LVDT (Linear Variable Differential Transformer) sensors were placed over the height of the segment at the vertical joints. These LVDT sensors are electromechanical transducers, that convert a given linear strain into a corresponding electrical signal. The position of these sensors over the vertical joint can be seen in Fig. 3.61. The measurement was done during the concreting process of segment 1 and 2. Two vertical joints of segment 1 were examined; one between elements 3 and 4, and another between elements 6 and 7 (Fig. 3.61). Six sensors per joint were placed with an even distribution.

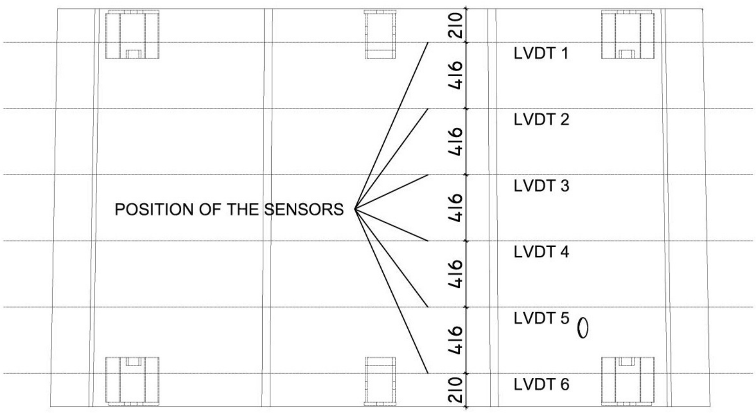


Figure 3.61: Location of the LVDT sensors over the segments' height [mm]

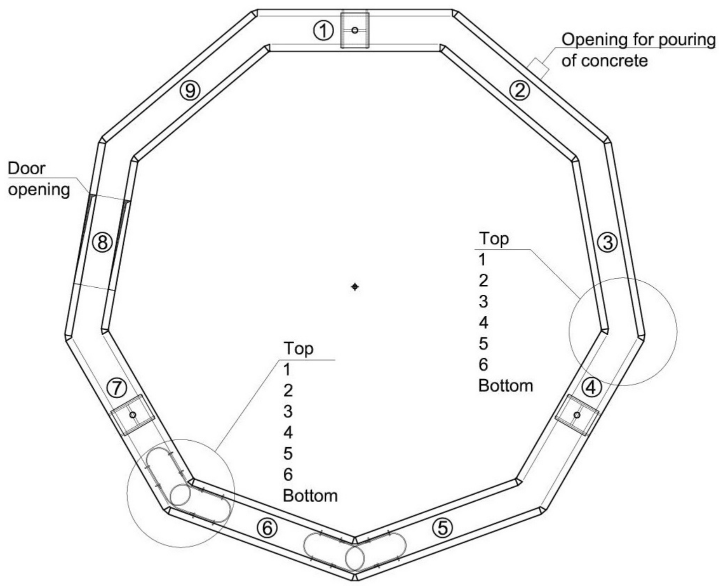


Figure 3.62: Location of the examined joints marked in the segments' ground view

The concreting started from the bottom, through the opening in element number 2 of the lower segment (Fig. 3.36). In under 13 min a 3 m height was filled with self-compacting concrete (SCC), as described in Subsection 3.2.3. After the concrete reached the desired height - covered the embedded base blocks of the upper segment - the pump was removed. Until the next mixer truck had arrived, the already poured concrete started to harden. 12 min later an additional 1.80 m high section of the upper segment was filled through the opening in the upper segment. After the procedure was finished, the joints were examined further, up to a total time of 2 h.

Figure 3.63: The position of the LVDT sensors on the vertical joint between elements 3 and 4

The joint deformation over time is pictured in the charts of Fig. 3.64 and 3.65, where the positive values mean a shrinkage and negative ones an expansion of the examined areas. The deformation of the segment correspond to the concreting process. As the concrete was continually poured into the hollow area the hydraulic pressure started to build up on the lower section of the segment. The top and the bottom sensors however show a significantly lesser deformation compared to the other four sensors in both joints. The reason for that is not only the lack of hydraulic pressure like for the top one, but also their placement at the height of the welded connections. At these points the stiff weld connection hinder the deformation of the segment. However a small expansion during the examination time is noticed at these points amounting to a maximum of 0.12 mm. The other four sensors show a similar deformation history, and as it could be expected, the two measurement points in the middle show the biggest expansion of all. At these areas the hydraulic pressure might be lower, but there is no tension bearing connection. This phenomenon was identified at both examined joints.

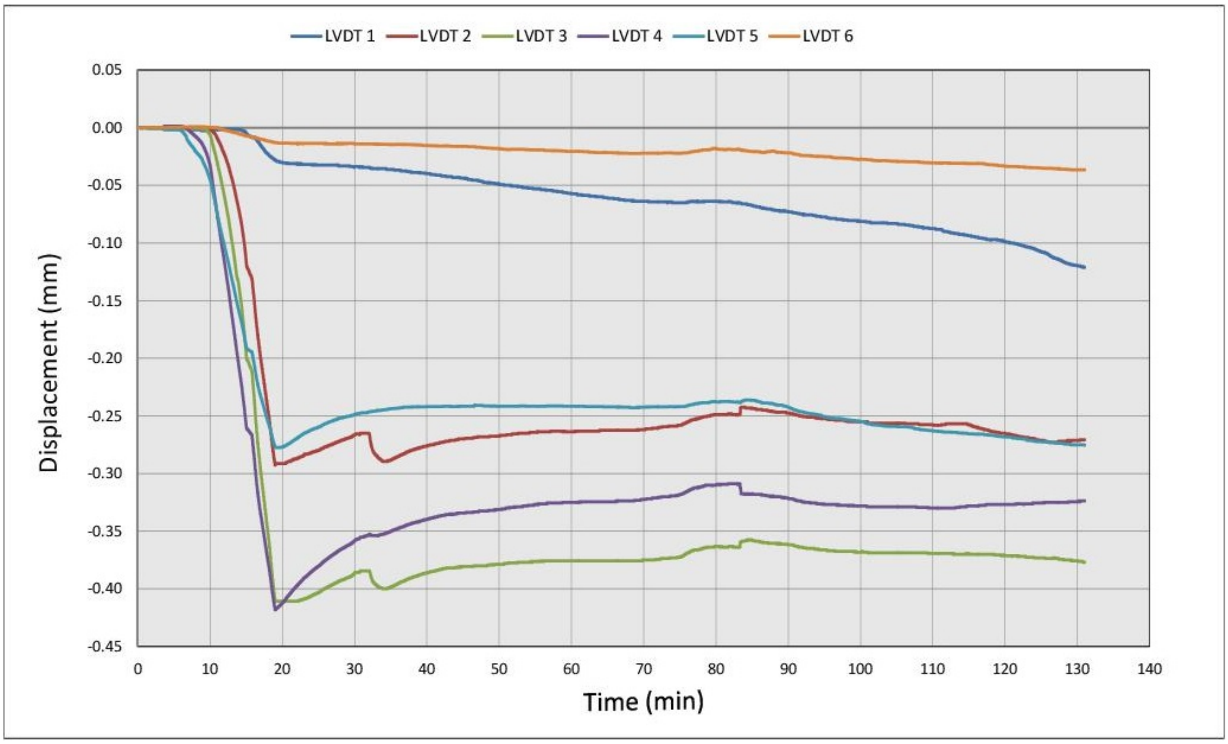


Figure 3.64: Displacement of the vertical joints between elements 3 and 4 of segment 1 according to the sensor location in Fig. 3.61

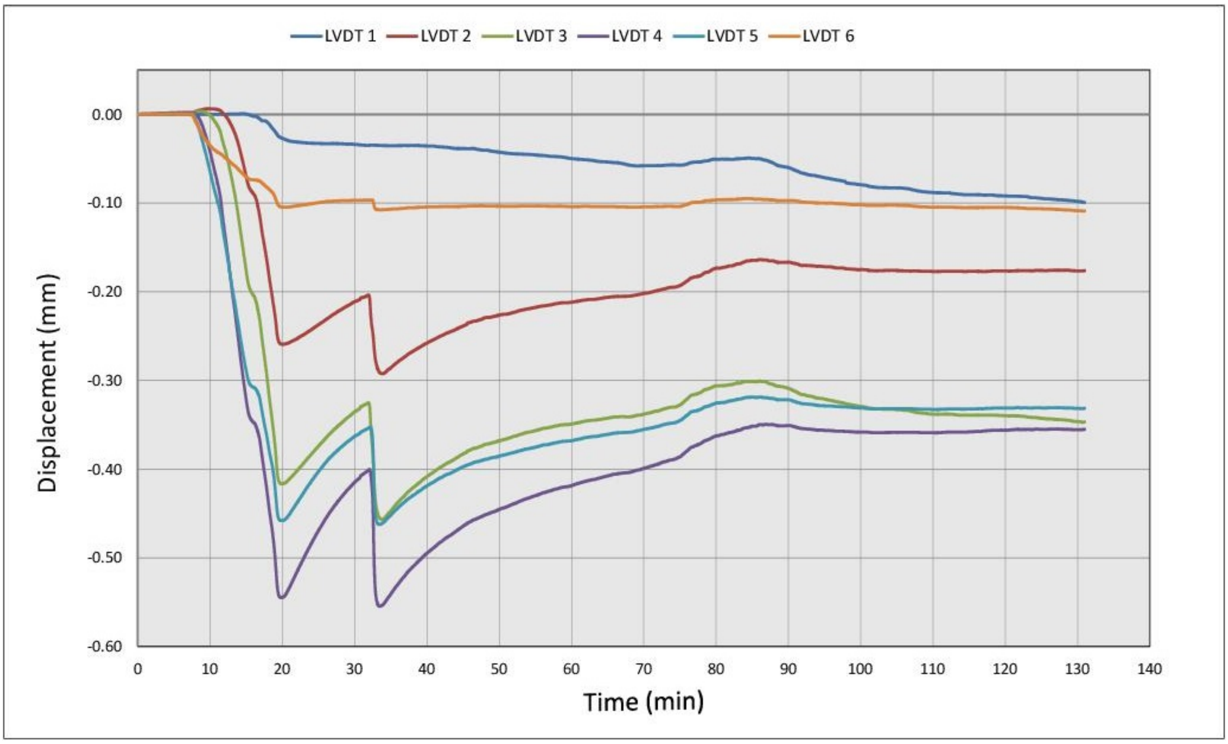


Figure 3.65: Displacement of the vertical joints between elements 6 and 7 of segment 1 according to the sensor location in Fig. 3.61

At the beginning of the concreting process (from minute 8 until 12 of the examination) a small shrinkage can be noticed in the uppermost sensor. This could have been due to the relative fast filling of the hollow area, what caused the elements to expand at the bottom and contract at the top of the segment, resulting in a minor compressive stress in the double wall connection.

In the joint between element 3 and 4 the maximal expansion during the concreting amounted to 0.43 mm while the one between 6 and 7 had an expansion of 0.55 mm. The possible reason is that the door opening in element number 8 causes a geometrical discontinuity, therefore the concrete stacks up along the walls of the door's formwork, causing a higher hydraulic pressure at these points. However the final deformation in general amounts to 0.35 mm, which can be stated to be similar to the deformation Janjic (2014) observed. Furthermore the usage of SCC (self-compacting concrete) validated his expectations as well. At his mock-up segment normal concrete was used, and the hydraulic pressure was built up with a slower speed farther away from the concrete pumping opening (Fig. 3.68). Using SCC resulted a nearly even distributed deformation at the same time in the whole segment, and therefore loaded the segment uniform. The maximal deformation occurred at minute 19 in the joint between elements 3 and 4, and a minute later (20) at the other side of the segment in the second examined vertical joint.

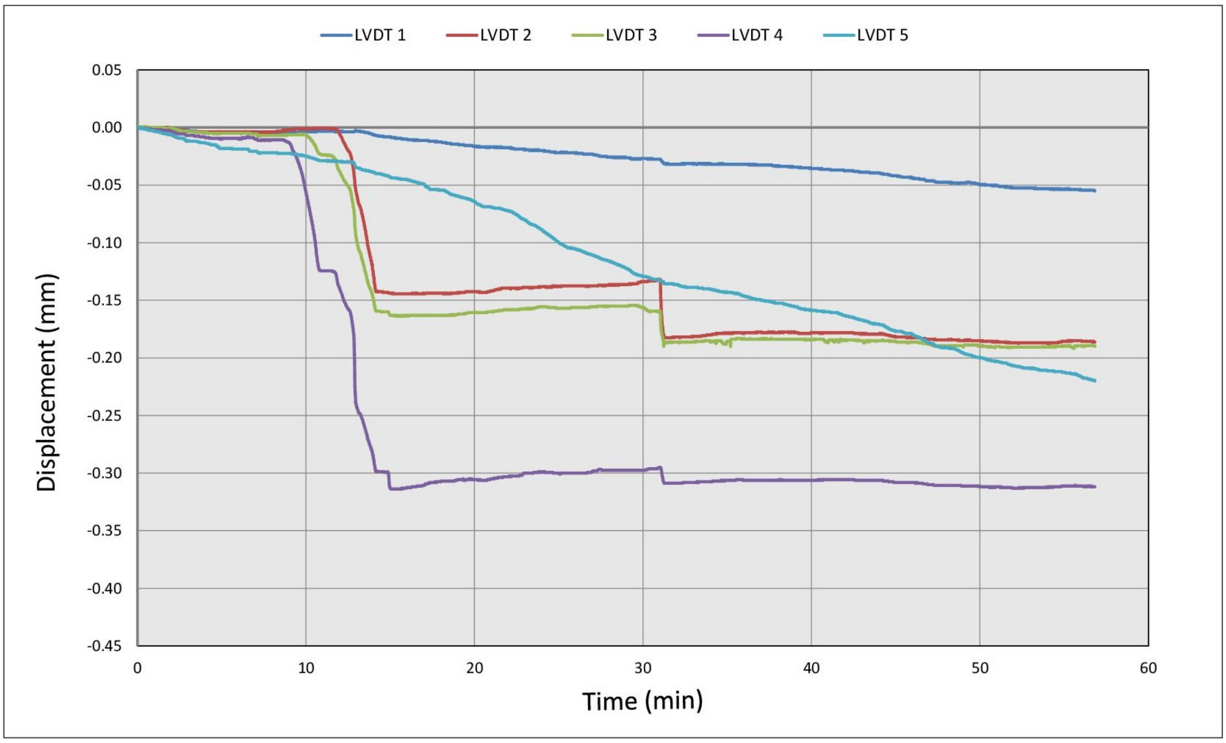


Figure 3.66: Displacement of the vertical joints between elements 1 and 9 of the prototype segment 3, taken from Janjic (2014)

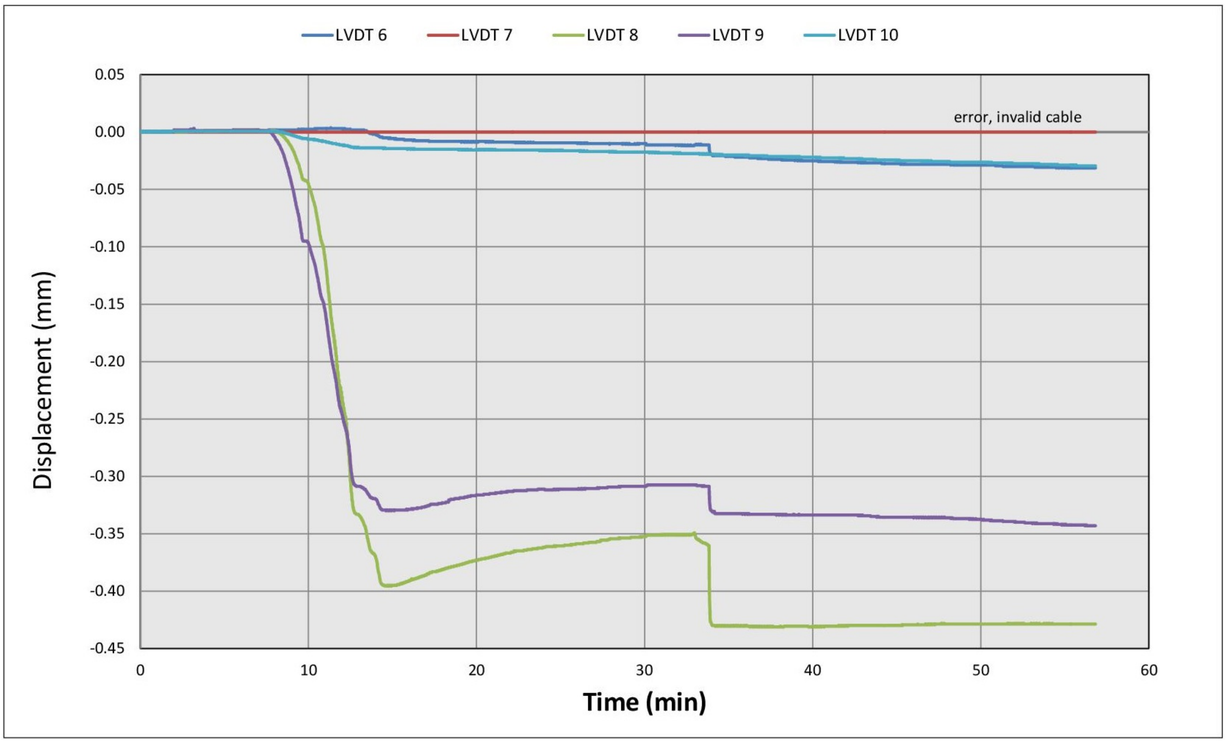


Figure 3.67: Displacement of the vertical joints between elements 3 and 4 of the prototype segment 3, taken from Janjic (2014)

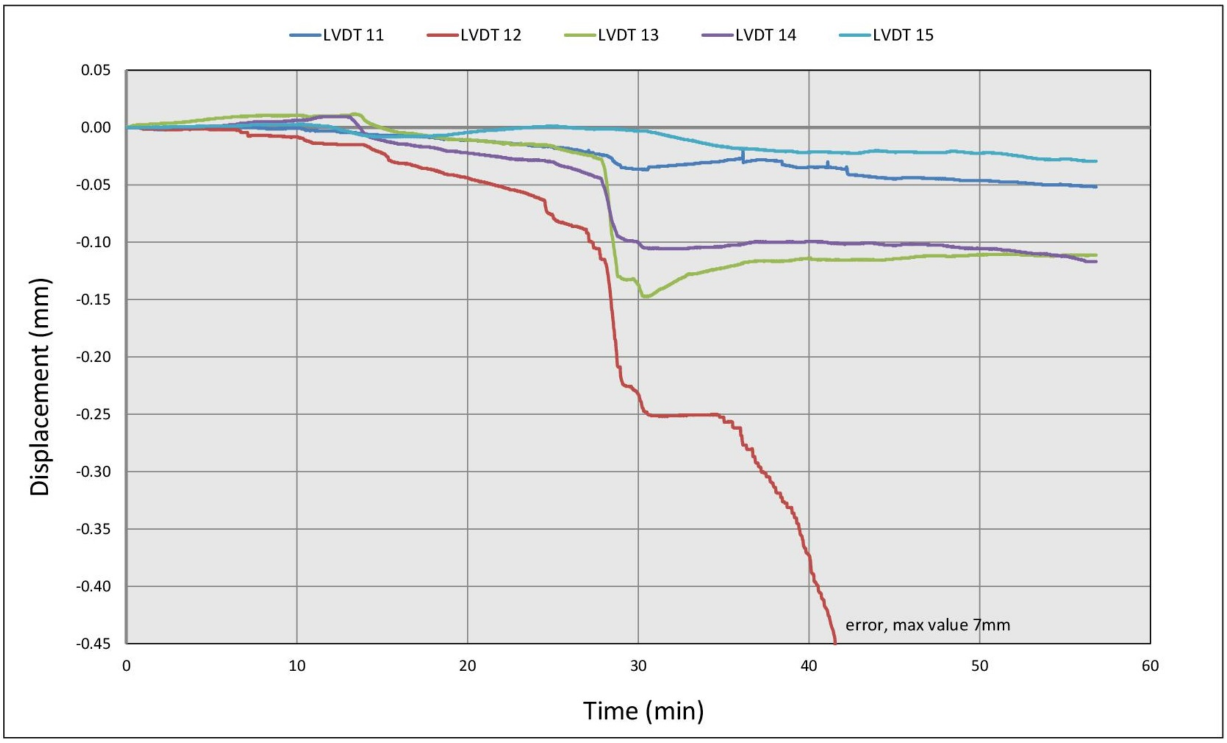


Figure 3.68: Displacement of the vertical joints between elements 6 and 7 of the prototype segment 3, taken from Janjic (2014)

The prototype segment 3 (Janjic (2014)) was filled with concrete similarly. In the first 10-15 minutes the concrete was pumped under pressure through the opening hole in the corresponding element, after that it was poured from the top (where the second jump occurs in Fig. 3.66, 3.67, and 3.68 at the 30 minute mark). One of the most important differences between the two tests is the use of SCC. For segment 1 the displacements were evenly distributed among the measured joints, while segment 3 of Janjic (2014) showed in the joint that was the farthest away from the opening significantly lesser displacements than in the other two.

Furthermore a small reduction of pressure over time can be seen in the diagrams 3.66, 3.67 and 3.68. The general reason for that can be explained with the faster hardening of the SCC. The stiffer it becomes, the less hydraulic pressure is applied on the walls, thus reducing the tension in the joints. However, for the joint between element 3 and 4 a significantly higher reduction can be noticed. After 90 min of the examination the displacements reached their final value (Fig. 3.69).

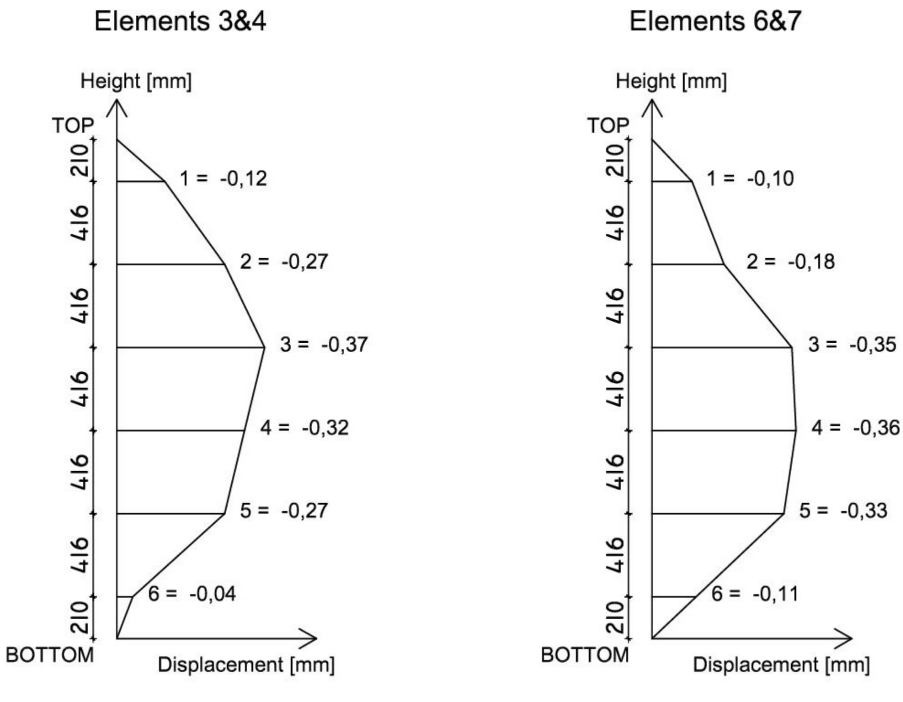


Figure 3.69: The final displacement of the vertical joints 3-4 and 6-7 of segment 1

During the concreting procedure of the prototype tower the speed of the filling was tested. Although the whole height of 4.80 m was filled in under 35 min, an additional bonding time would be necessary before continuing the concreting process. To determine the exact speed, the periods of the filling and hardening times are examined. The first period lasts from minute 20 (the beginning of the pouring) until 35, when the first 3 m were filled, and already started to harden. The second period is from minute 35, starting with the pouring of the additional 1.80 m, and lasts until around minute 60, where the deformation shrinks back to the same value as at the end of the first period. This provides a filling velocity of 4.80 m/h for the prototype. It's difficult to make any further conclusions regarding higher elements, but it can be stated that this velocity is a valid value. Whether the speed can be maintained in the case of 13 m high segments, further examinations have to be made.

Chapter 4

Summary, conclusions, and future outlook

This thesis serves as a study of the newest erection methods for wind turbine towers containing a detailed discussion of the manufacturing and erection processes of a prototype based on a new method proposed by the Institute of Structural Engineering at the TU Wien. The recent development of the wind turbine industry shows that the requirements for higher towers can be best met with precast concrete constructions, however other materials are also used. The main disadvantages of the most methods are the prefab dimension limitations due to transportation and often a small bearing resistance in relation to the used material qualities. The herein introduced wind tower design however aims to combine the favourable properties of the easily transportable lightweight double wall elements and the high load bearing resistance cast-in-place concrete structures. The method suggests a simpler and more economical tower structure compared to fully precast concrete element structures. To determine the feasibility of the concept, a more than 16 m high prototype out of double walls was built.

The thesis furthermore reflects on the work of Janjic (2014), who participated in the early development of a mock-up segment, including the comparison and evaluation of the original ideas and the altered solutions used for the prototype. The aim is to introduce the new method with all it's advantages and flaws, suggesting future development ideas in order to improve the efficiency and accuracy for a possible serial production. Even though the experimenting with different solutions was limited during the manufacturing, the implemented alterations in varying segments allow for drawing conclusions whether they improve the process or other solutions are to be found.

4.1 Evaluation of the double wall production process

The production of the double wall elements is one of the most important processes in the whole procedure of the tower construction. Accuracy, production rate, cost efficiency and reliability play an important role. The double wall elements are produced in a highly automated plant, providing the same manufacturing conditions for the various parts.

- Every third element of each segment received a lifting and positioning embedment either made out of steel or concrete. These blocks would serve as an auxiliary socket for the accurate positioning of the segments during erection. Both materials were used and tested during the production. Given their designed outer dimensions, the blocks have an identical volume, and weight approximately the same. Although the production of the steel blocks might have been more complicated and more expensive, they proved to be easier to handle. They could also be fixed more accurate to their foreseen position, using industrial magnets.

- The production of the double wall elements has a great influence on the erection of the wind tower on the construction site. For that reason all 54 of the produced elements were examined. In the process the height, width, and thickness of the slabs, furthermore the weight, the relative offset of the slabs and the total thickness of the elements were measured. Depending on the workflow of the elements, the inaccuracies could be identified and quantified. The placement of the formwork is automated up to 95 %, what results in minor - therefore acceptable - maximal deviations regarding the height and width of the elements (0.3 % to 0.8 %). The only exception was the element 3 of segment 4 with a deviation of 5.2 %. This could originate in the manual placement of a small formwork part with inadequate fixing (Fischer (2015)). It is recommended to minimise the manual formwork work by the automated placed one as often as possible.

- The formwork can be filled with concrete automatically and manually as well. In the case of manual control a measured maximal deviation of 39.8 % appears in the thickness of the slabs; inner slab of element 3 of segment 1 (Fig. 3.6). This results in the increased maximal weight of the elements, up to 28.7 % in case of element 1 of segment 1. Such deviations in the weight of the elements have to be either taken into account or minimized within the limits of economy, as they have an influence on the transportation and the required lifting equipment on the construction site. For that reason it would be advised to fill the formworks automatically and only use manual placement to improve vital areas (Fischer (2015)).

- In the last step, the merging of the two slabs governs the offset and the total thickness of the elements. The deviations of both influence the needed size of the vertical segment joints. A maximal deviation of 11.2 % appears in the horizontal offset of both slabs at the top of element 1 of segment 1. This amount is however still within the tolerance limits, as such failures were calculated in the designed joint width.

In contrast the total element thickness had a maximal deviation of a minor 3.2 % at element 3 of segment 2. The deviations resulting from the merging process can be minimised if possible collisions between the reinforcement and slab connecting embedments are minimised. Moreover if the occurring deviations are determined during the production, a special designed device could be used to bring the slabs in the right position as long as one of both slabs is not hardened.

All in all some simple modification in the double wall production as well as the prior made CAD planning can improve the accuracy of the double walls. This can be easily implemented in any pallet circulate carousel. As marginal note it can be stated that the relative deviation gets smaller the huger the elements get, what means that the possible errors are absolute values and independent of the element size. This is a good message for the production of segments with heights up to 13 m. Further it is recommended that a quality management recording geometrical data is implemented in order to identify reject goods so that elements with too big deviations of the geometry does not hinder the tower erection (Fischer (2015)).

4.2 Evaluation of the erection process

The erection of the prototype worked most of the time as intended, however numerous challenges arose and had to be dealt with. Starting from the assembly of the segments to the positioning and filling the polygonal rings with concrete, the process was confronted by following challenges:

- During the assembly of a segment one of the most crucial tasks was the accurate positioning of the elements with the desired inclination, within strict accuracy tolerances. The main difficulty here was the uneven surface of the concrete pre-assembly field. A horizontal plane is required to avoid any collisions between the elements in order to ensure an identically accurate inclination. Plastic distance plates were placed under the elements providing the same height. An alternative could be the use of a precise steel construction as a pre-assembly sub-structure, as it would provide a much higher accuracy and could be already equipped with measuring and height adjusting equipment.

- As all segments share the same tapered form, another solution to aid the assembly could be the use of an auxiliary construction similar to the one discussed in Subsection 2.4.1. The system could have different designs. Either a central pillar with adjustable steel arms to provide the angle for any desired radius, or having one construction for each segment with the required diameter. With the aid of such a construction the assembly process would require significantly less time and the possible failure rate would be reduced due to the replacement of manual measurements. Although the development of a system like that is accompanied by initial investment costs, it pays off with the serial production of the towers.

- After two segments were positioned over each other, the horizontal joint had to be sealed prior to the concreting process. The used formwork consisted out of nine pairs of wooden boards. The proposed formwork design served well along the polygon sides but it was difficult to seal the edges. The design have to be improved e.g. using edge pieces out of steel.

4.3 Future outlook

The prototype showed that the concept is feasible, but it only gave a glimpse on the quantitative advantages of the proposed erection method. In the next step a wind turbine with a certain hub height has to be chosen so that a tower can be designed enabling not only a qualitative but also quantitative assessment of the tower erection method. A currently ongoing research indicate that a wind turbine with a hub height of 140 m with a hybrid design would consist out of a 80 m high concrete part and a bit less than 60 m high steel part (because of the turbine on top). The concrete part would be put together out of 10 segments with a height of approximately 8 m and a dodecagonal (twelve-edged) shape. The chosen cross-section reflects the fact that the outer polygon diameter would be around 10 m resulting in an extent of the polygon of 31.06 m further resulting in an element width of 2.59 m. Some of the herein presented structural details will have to be altered to meet the requirements of the bigger segments but it is possible. All in all, again the big advantages of a semi-precast concrete tower being the fast tower erection using decentralised produced and cheap standard double walls while providing the bearing capacity of a cast-in-place structure will establish the proposed construction method for high towers at the markets worldwide.

List of Figures

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