3 2017

vakblad over betonconstructies

Cement

fib Symposium 2017 Maastricht

Partners van *Cement*, kennisplatform betonconstructies

Cement is een platform van én voor constructeurs. Het platform legt kennis vast over construeren met beton, en verspreidt deze onder vakgenoten. Om het belang hiervan te onderstrepen kan een bedrijf kennispartner van *Cement* worden.

Een partner geniet een aantal voordelen, zoals een flinke korting op het abonnement en een profielpagina op *Cementonline*. Het partnerschap is voorbehouden aan bedrijven voor wie de kennis daadwerkelijk is bedoeld. Hebt u ook interesse om partner te worden, neem dan contact op met Jacques Linssen, j.linssen@aeneas.nl.

abt

Alferink van Schieveen INGENIEURSBUREAU

AMB INGENIEURS

bam

BARTELS INGENIEURS VOOR BOUW & INFRA

B|A|S Research & Technology

BEKAERT better together

BETON VERENIGING

BuildSoft structurally loved by engineers

Cement&BetonCentrum

Centraal overleg Bouwconstructies

CONSTRUSOFT

geelhoed engineering b.v.

goudstikker | de vries

Adviesbureau ir. J.G. Hageman B.V.

IMd Raadgevende Ingenieurs

Pieters BOUWTECHNIEK

VAN ROSSUM RAADGEVENDE INGENIEURS

SBR CUR net

SCIA A NEMETSCHEK COMPANY

Stufib

Verhoeven en Leenders ingenieurs in bouwconstructies

VeriCon ingenieurs voor de bouw

VNconstructeurs

YouCon

Meer informatie over deze bedrijven en over het partnerschap staat op www.cementonline.nl/partners.

4 – 7
'Architecture is a misconception'

Chris Poulissen has created a number of striking bridge designs. Yet he prefers not to call himself an architect. An interview with the keynote speaker at the *fib* Symposium 2017.

8 – 13
Africa's largest suspension bridge

To improve the transportation network between the capital city Maputo and the south of Mozambique, a suspension bridge with a total length of 1225 m is being constructed.

64 – 67
World's largest sea lock

The Noordersluis in IJmuiden (The Netherlands) is being replaced by a new gigantic concrete sea lock, made from 290 000 m^3 of concrete, that will provide defence against the rising sea levels.

Cover photo:
Almonte River Viaduct, Spain
credits: Rúbrica, Castellón (Spain)

Beste *Cement*-lezer, dear *Cement*-reader,

For probably everybody this edition of *Cement* will be special and therefore some explanation seems to be appropriate. For the Dutch readers, besides content (more foreign projects) and the large number of pages (almost twice the usual size), the language (English) is also special. For foreign readers the journal on itself will already be special.

For the foreign readers: *Cement* is the Dutch journal on concrete for structural engineers for already 69 years. The eight editions per year consist of papers on structural issues and challenging projects, in which the various aspects, like architecture, structural design, material technology and execution, are illustrated.

For the Dutch readers: because of the international 2017 *fib* Symposium being hosted in the Netherlands (Maastricht, June 12-14, 2017), it was decided to combine special symposium sessions with this English edition of *Cement* and to also distribute it among the participants of the symposium.

The theme of the 2017 *fib* Symposium is 'High Tech Concrete: Where technology and engineering meet'. In order to contribute to a *fib* symposium you are supposed to supply a scientific paper, that will be peer reviewed. Generally, this is a barrier for people from the engineering practice to participate. On the other hand, the largest projects worldwide are done by engineers that have very wide knowledge and broad experience worthwhile sharing with others. So, there was this idea to give them the opportunity to present projects by only supplying general information and interesting photos and figures. Since this is not acceptable for the proceedings of the symposium, and, in fact, is similar to the content of *Cement*, the idea for a special English edition of *Cement*, a meeting point for Dutch and worldwide engineering, was born.

Reviewing, gathering the required information and bringing everything in the desired format, resulting in this special *fib Cement* issue, was thanks to the efforts of:

Paul Lagendijk
Delft University of Technology

Mladena Luković
Delft University of Technology

Hanneke Schaap
Aeneas Media

We also want to thank the authors for their very valuable input and cooperation.

Dick Hordijk
Editor-in-chief *Cement* and Chair Organizing Committee 2017 *fib* Symposium
Jacques Linssen
Editor and publisher *Cement*

Cement's editorial staff talk to Chris Poulissen, keynote speaker at the *fib* Symposium 2017

'Architecture is a misconception'

Chris Poulissen, from the Flemish part of Belgium, has created a number of striking bridge designs together with his partner Laurent Ney, including in the Netherlands. Yet he prefers not to call himself an architect. "Forget that word. Architecture isn't important. There are so many more serious matters in the world, so many bigger challenges."

1 De Oversteek, Nijmegen, The Netherlands
2 Ghatkopar Koparkhairne Bridge, Mumbai
credits: Ney & Partners

Career

Chris Poulissen began his career at AWG (Bob van Reeth's Architect Work Group). He soon met Laurent Ney (during the renovation of the Koning Boudewijnstadium, for example), who was working as an engineer at engineering firm Bureau Greisch. In 1995 Poulissen launched his own firm: Architectenbureau C. Poulissen, which later became Poulissen & Partners. Poulissen and Ney have always cooperated closely. Increasingly, they have been focusing on designing bridges. When they received the assignment to design the Oosterweel Link in Antwerp, they founded the firm Ney-Poulissen Architects & Engineers, which was renamed NP-Bridging in 2011. They now work all over the world, including in the Netherlands, India and Japan. Well-known Dutch projects include De Oversteek and De Lentloper, both recently built in the city of Nijmegen.

Ask Poulissen where he gets his ideas for his designs and he will tell you he doesn't know. "The best designs evolve gradually, in a process with multiple people. You have to consider all of the interests at play. Things like ecology, flora, fauna, noise pollution, contamination. It's barely about the form. For example, when Laurent and I were working on the bridge project 'De Oversteek' in Nijmegen (photo 1), we didn't anticipate beforehand that those arches would be there. That idea materialised during the project, in part because of the limited budget. That forced us to find clever solutions. And the solution with the arches turned out to cost a lot less, and it doesn't require much maintenance. There are no joints, no bearings. The design for the bridge 'De Lentloper' (photos 3 and 6) didn't fall out of the sky either. We turned the reference design, which was based on prefab girders, completely inside out. In the end it resulted in a design that cost 30% less than the budget and also generated 25% more surface area (fig. 4)."

2

Design competitions

The most important challenge in a design process is not what something will eventually look like, but what the real needs are. That's why architecture is a misconception according to Poulissen. "It's not about architecture, it's about the bigger picture, where you stand in life." Poulissen and Ney had to fight for two to three years to get a footpath onto their design for two megabridges in Mumbai (fig. 2). The overwhelming majority of the city's 20 million inhabitants doesn't have a car. "In my view they also had a right to move from one side of the river to the other. For me, that's the essence of bridges. De Oversteek was about a footpath too. It was supposed to be 1.5 km long, but according to the municipality that's why it would never be used. But as it turns out, it's a huge success, almost too huge if you look at how busy it is there sometimes."
That explains why Poulissen is not a fan of the design competition phenomenon. During a contest for Groenplaats, the historic square in Antwerp, Poulissen went so far that he barely even showed his design. "I did have a design on me, but I said beforehand that I didn't know if that would be it and that I didn't know what would be it either. I also said that if you want to get yourself into a huge mess, you should make a design and say 'this is it'. I advised them to talk to the people who live and work near the square to find out what their interests are. We had no choice in my opinion. We had to ask these people before putting even a single line onto paper."
Another major disadvantage of design competitions is the waste according to Poulissen. "We have to stop making each other miserable. Take De Oversteek.

"I can't stand misery in the world"

3

Do you really think that if one of the other seven candidates had won it would be a much worse bridge? Our competitors put a lot of time and energy into a design that didn't win and therefor wasted valuable money. And money is probably not even the most important problem. What do you think happens to young people who miss out on a project, and miss out again, and again? That's how you destroy ambitions and dreams. Of course, I understand why these competitions are held. Clients have to be able to justify their decisions. But I think we can do better. And why shouldn't a losing team put their ideas at the disposal of the winner, so he can make his design even better? Now that knowledge is completely lost."

Cooperation

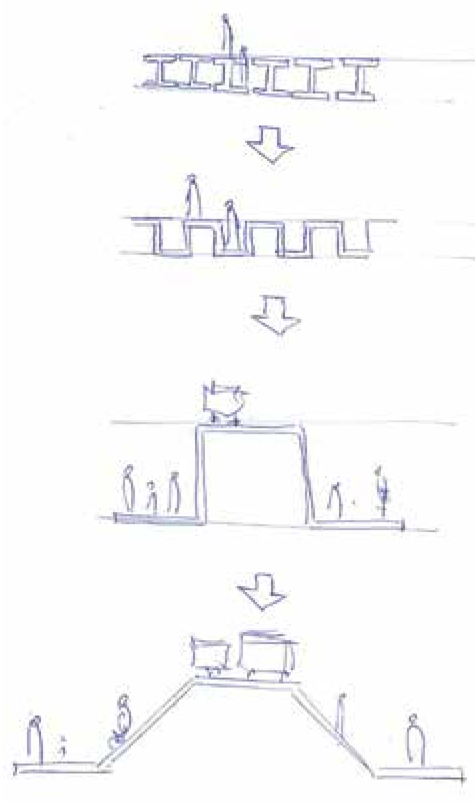
The process is still too scattered according to Poulissen. In practice, it often comes down to an architect conceiving of something based on form, and so he is creating a mechanical problem that an engineer will have to solve. The engineer will use all kinds of complex sums and complicated programs to show that the construction meets a standard. And once he has calculated everything, the contractor has to find a way to build it. But the contractor isn't always aware of where the design came from. Because we are not allowed to provide him with information during the tendering stage. Sure, sometimes there's this competitive dialogue, but that's in writing. That's not a dialogue! His lack of information will cause the contractor to do everything he can to reduce risk. So he will think of yet more ways to adapt the construction. That's not how it should be done. You have to develop things together. Determine together how a project should take shape, when and how. It's about the nature of the construction, the nature of the raw materials, the nature of the needs. You should be guided by the materials. That's what will tell you what form it should have, so to speak."

Indeed, cooperation is important to Poulissen. It's taken for granted in the Netherlands. "It's really part of your DNA. It's for good reason that you're pioneers in public-private partnerships. But in Belgium people don't trust you if you suggest working together. It makes them wonder, 'what does he want from me?'"

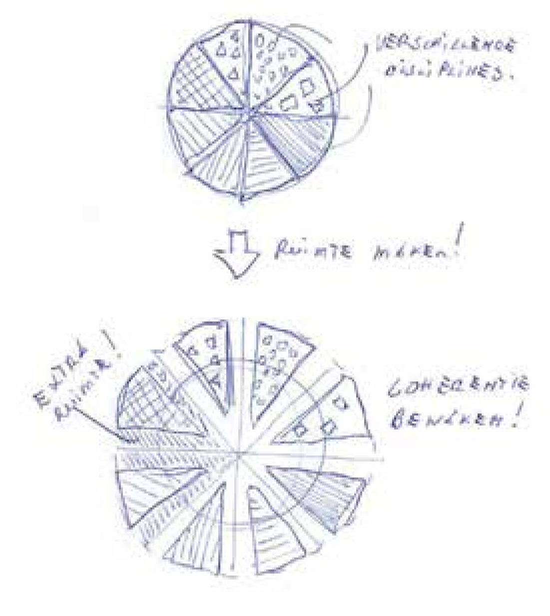
In practice, not much always comes out of an integrated approach. Architects design something that they think will please the client. "It makes it easy to determine what kind of a risk someone should take. That idea is out of date. Why not have a designer bear part of the financial risk of a project for once? I guarantee you that the world would look much different. Undoubtedly more exciting, interesting, intense, serious and responsible."

"So it's much more about what's important to people than it is about the form. We always try to discover what the individual interests are. In everything, in every project, in every conversation. For an investor, that means a return on your investment. The bridge or the building is not the point at all. He couldn't care less about that. Me either, for that matter. What I want is to make the world better. I can't stand all this misery. I really can't stand it. My body reacts to it. That's why I wanted to make a small contribution in India to reducing the enormous gap

"It's not about architecture but about the bigger picture, where you stand in life"



4



5

3 De Lentloper, Nijmegen, The Netherlands
credits: Ney & Partners / Thea van den Heuvel

4 The design of De Oversteek originated from a reference design based on prefab girders and the idea of making the best possible use of the pedestrian surface of the cross section

5 Making the pie bigger gives the players more space

6 De Lentloper
credits: Ney & Partners / Thea van den Heuvel

between the poor and the rich. The footpath that I mentioned earlier is an example of that. Believe me, what the bridge looks like doesn't interest me much."

Developer

"People sometimes say that I have a lot of luck. That's true. But I did create situations that make it possible to have good luck. I bought that ticket to India. I took the risk 18 years ago of developing that old warehouse on the 'Eilandje' Island in Antwerp. I once said that I am a developer. Not in the conventional sense of the word. But I do feel like a developer. I have brought people together and instilled enthusiasm in them. I try to set processes into motion. My role in doing so is to let people put their heads together and come up with a design. I try to ask the right questions. For as long as it takes until I understand what people are saying. And then I grab a marker and draw it on a flip-over. By talking *and* drawing you're using two channels simultaneously. That's a great help in understanding what's meant. But take a look at a large engineering firm. There won't be a flip-over anywhere in sight! Just a TV screen hanging somewhere for presentations. But only one person out of ten probably dares to go near the screen. People don't participate nearly enough."

Intelligent

"I try to be the oil in the machine. It's what gets everything running smoothly and effectively. I sometimes explain my role using a pie as an analogy, in which everyone involved in a project is a piece of that pie. What I do is try to make the pie bigger so that everyone has more space, more chance to be themselves and therefore function better (fig. 5). That makes people happy. In the end, I make sure that it's a coherent entity again, that the pieces of pie come together again. You need good people for that. I once heard that you can tell if someone's intelligent because they will surround themselves with more intelligent people. I firmly believe that. The people working in my office are all smarter than me. Otherwise they wouldn't be here, because we could do what they do ourselves. And luckily there's Laurent. He's seven times wiser as me." ☒

> "Why not have a designer bear part of the financial risk of a project? The world would look much different"

Jacques Linssen and Dick Hordijk

6

Africa's largest supension bridge

Mozambique, located on the east coast of southern Africa with a coastline of 2800 km (fig. 2), is rich in natural resources and is rated as the 4th fastest growing economy in Africa. Part of the Mozambique's National Development Master Plan is to improve the transportation network between the capital city Maputo and the south of the country. To achieve this aim, a bridge is being constructed across the Maputo Bay as a connection to South Africa. After its completion in 2018, it will be the longest suspension bridge in Africa with a main span of 680 m and a total length of 1225 m.

Construction of the Maputo-Katembe Bridge (photo 1 and fig. 3) started late 2014 with a total project value, including the southern link roads, of approx. US$ 700 million. Design and execution is being carried out by China Road & Bridge Corporation (CRBC) and is based on FIDIC's Silver book EPC contract. German consultant GAUFF Engineering is responsible for quality supervision as well as design verification according to the Eurocode.

Concept of the bridges

The bridge consists of reinforced concrete approach viaducts on the North and South banks of the crossing, which connect to the main span, a suspension bridge constructed of steel box girder sections, with two large subsoil gravity anchor blocks that are filled with sand and concrete. The bridge will carry four traffic lanes, two in each direction, with a design speed of 80 km/h.

Jörn Seitz
GAUFF GmbH & Co. Engineering KG,
Nuremberg (Germany)
Dean Gary Swanepoel
GAUFF GmbH & Co. Engineering KG,
Maputo (Mozambique)
B. Pengyu
China Road and Bridge Corporation
(CRBC), Bejing (China)

1 Maputo construction site: North Approach Bridge with two of the eight piers for the free cantilever bridge that will lead over the anchor block
credits photos: GAUFF Engineering
2 Map of Mozambique
3 Visualisation of the finished Maputo Katembe Bridge

2

The North and South Approach bridges are being constructed using two different design and construction methods influenced by the local urban development. In the north, located in the middle of a very congested central business district and harbour, the approach bridge will be a balanced cantilever 853 m long construction rising up towards the main pylon. The main distances between the piers are 119 m. The Southern Approach Bridge, situated in a totally rural area without any obstructions, will be built using prefabricated post-tensioned T-beams to form its total length of 1234 m (photo 4 and 5). The approach bridges connect on either side to a single-span double-hinged suspension bridge with a centre span of 680 m, supported by hangers attached to two cables, which are draped over the main cable saddles of the towers and connected to the anchor blocks on each side of the river. Side spans are 260 m and 285 m long.

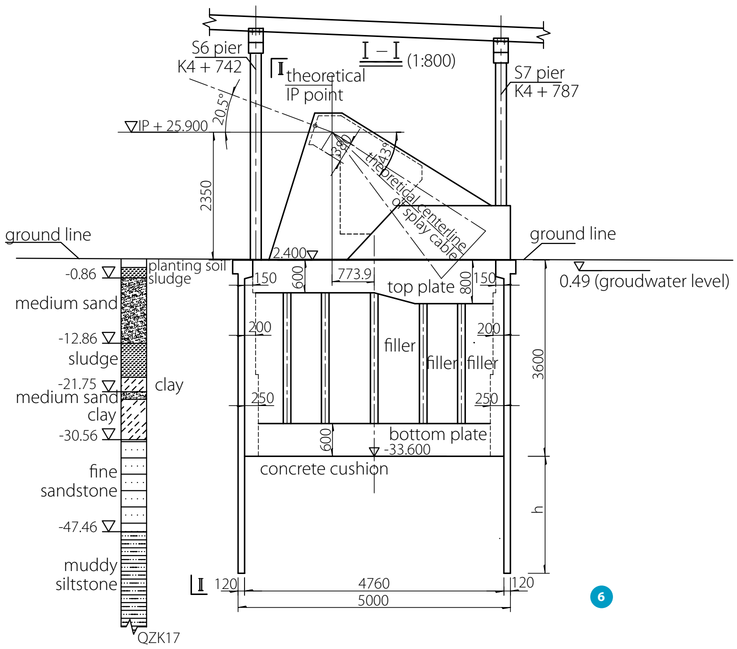
The bridge concept was designed according to Chinese standards with the overall design verified against Eurocode specifications and specifications according to the South Africa Transport and Communications Commission (SATCC). Especially for the pile foundation changes in the amount of reinforcement were required considering considering the requirements by the different codes.

Each gravity anchorage is made up of the foundation, splay-saddle buttress, and anchorage chambers. Some of these chambers are empty, and some are filled with concrete and sand requiring a specific density, all adding to the total weight of the structure. Each shaft has an external diameter of approximately 50 m, a wall thickness of 1.20 m and a wall panel depth of up to 56 m. The deepest anchorage structure was the one on the south side of the crossing; with 37 m it is believed to be one of the world deepest constructed during the last years. Figure 6 gives the structural detail of the shaft and photo 7 shows the final construction stage for the completion of the anchorage block inside of the shaft. Photo 8 illustrates the completion of

3

4

5



6

the massive concrete construction built on the shaft to hold the main cables.

Piling and diaphragm walls for the anchorage shafts

As there was no comparable project in Mozambique for the design of the bridge foundation piles, the design was based on the findings of a geotechnical investigation, which started two years ahead of the actual construction work. Pile construction for the towers and foreshore bridge piers began in tandem with the anchorage excavation. Before pile production could begin, their bearing capacity was verified using static test loads. Based on this all 331 piles were optimised in diameter and length. A total of 283 piles were constructed for the approach bridges, each with a diameter of 1.50 m and an average depth of 50 m, and 48 piles were installed for the towers, 24 at each tower, and each with a diameter of 2.20 m and length of 110 m at the south tower and 95 m at the north.

The excavation for the piling followed the international reverse-circulation-drilling method. The quality and integrity of the concrete in all piles was verified over their total length using the Crosshole Sonic Logging (CSL) testing method by an independent third party after 28 days. Concrete cubes were manufactured even for 365-day compressive strength tests.

Pylon, cables and steel box girders

The main structure of the tower is formed of rectangular hollow box sections, with a length of 7.00 m and a width of 5.00 m. The wall thickness of the upper tower is 1.00 m, and this increases to 1.20 m towards the bottom, resulting in a total thickness of 1.80 m at the base. The final height of the tower on the north, Maputo side, will be 135 m and on the south, Katembe side, just one meter higher (fig. 10 and photo 11).

Prefabricated parallel wire strands will be used for the main cables, which are made up of 91 galvanised high-strength steel wires, 5 mm in diameter with a nominal tensile strength of

4 Pylon M2 (south) and piers of the approach bridge
5 Second of 34 piers of the Southern Approach Bridge (December 2016)
6 Cross section of southern shaft with charateristic soil layers
7 Construction stage of the southern anchor block
8 Massive concrete construction of the northern anchor block

1670 MPa, resulting in an outside diameter of 509 mm and a total strand length of 1317 m. The main cables are connected on each side of the bay into the gravity anchorage blocks (photo 9 and 12). The cables are specially protected against corrosion in a permanent airtight system.

For the hangers, galvanised high strength steel wires will be used. The transverse distance between the main cables hangers is 21.90 m and the standard distance between the hangers along the bridges main span orientation is 12 m, with the length of hangers ranging from 73 m at the towers to 3 m at midspan.

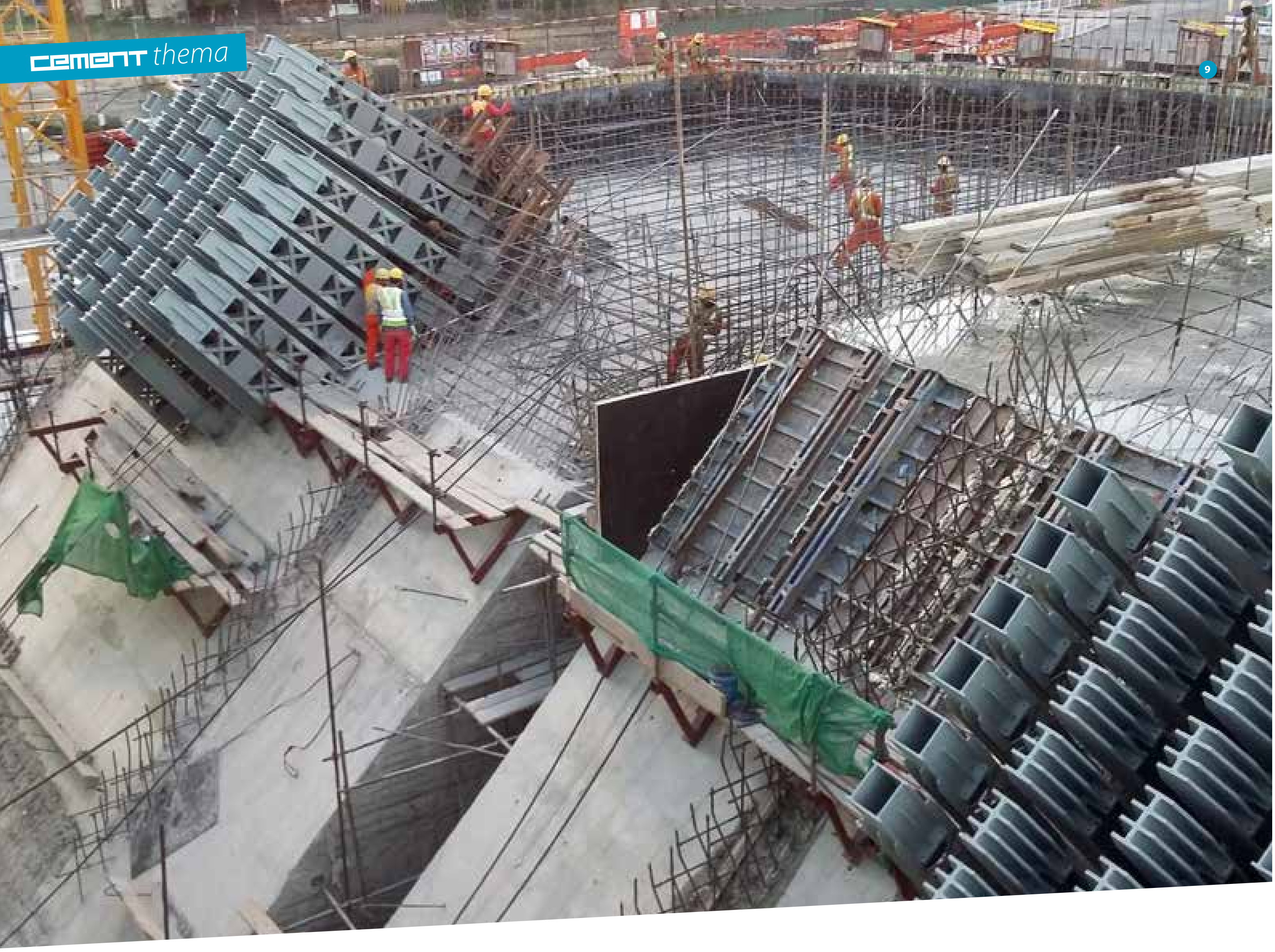
The steel box girders are being manufactured in Nantong near Shanghai in China and will be delivered to Maputo by ship by manufacturer ZPMC.

Special concrete

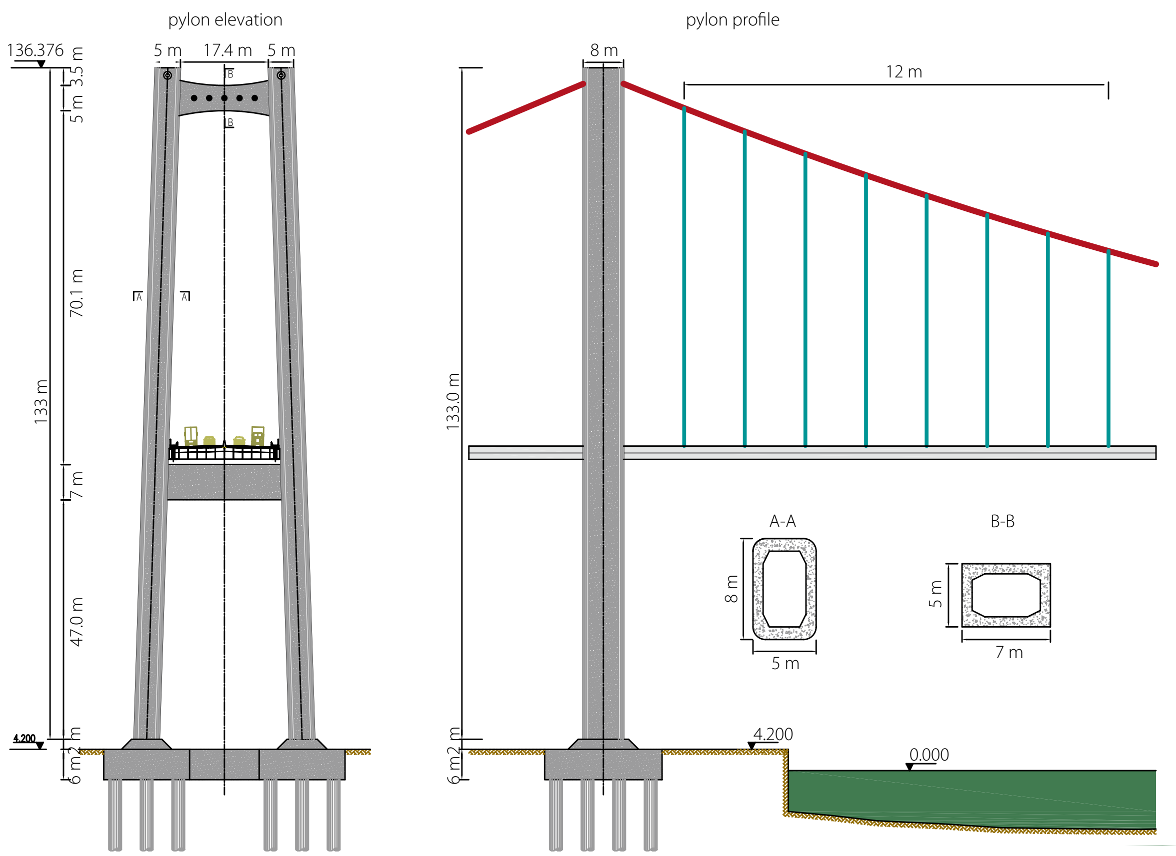
One of the unusual aspects of the concrete on this project was the addition of up to 40% fly ash. This not only offers immediate cost savings but also long term benefits. The fly ash is produced in and delivered from South Africa and gives the concrete an extremely high durability, a fact which was confirmed by the University of Cape Town's Concrete Materials & Structural Integrity Unit which performed Durability Indexes testing on the samples cored from the bottom slab of the anchorage. The tests performed were the OPI- oxygen permeability, WSI-Water sorptivity Index and the CCI-chloride conductivity index. The results obtained were confirmed as the best test results ever obtained from a site concrete. High workability was of vital

8

7



9 Installation of the steel constructions for the anchorage of the main cables (north side)
10 Details of the pylon
11 Construction of southern Pylon M2 (January 2017)
12 Construction of the southern anchor block



10

importance to the project during the casting of the anchorage base slabs and pumping the concrete up to heights of 140 m. Laboratory testing confirmed that the concrete was still workable up to 16 hours after the initial mixing. This is directly related to the high quantity of fly ash and a retarder from China specifically formulated for this project. Concrete cube crushing strengths at 28 and 90 days have confirmed a remarkable strength gain. C40 concrete had results of 51.9 MPa and 69.5 MPa at 28 and 90 days respectively.

Producing sustainable concrete and most importantly a sustainable project is particularly important to CRBC and the client. Achieving a balance of social, environmental and economical factors is part of the contractor's quality management system which was developed by GAUFF Engineering following the 'Triple Bottom Line' concept from the United Nations' Bruntland Report. Through this CRBC aspires to produce a sustainable structure as a whole and to promote sustainability across the board. Reduction of its carbon footprint by reducing CO_2 emissions is part of the company's mix-design philosophy and is achieved through the use of fly ash as an extender; it has resulted in dramatically lowering the cementitious CO_2 emissions of the concrete from an estimated 352.5 kg CO_2/t to 229.5 kg CO_2/t, a reduction of 35%.

Summary

For this project, CRBC and GAUFF together with the client have developed a comprehensive quality management monitoring system, which covers all aspects of construction in Maputo and also the extensive production of the complex steel components being manufactured in China.
The calculations using Chinese standards and their verification against Eurocodes were completed in June 2016, alongside the production of piles and diaphragm walls. In the coming 18 months the construction work will focus on steel fabrication for the suspension bridge, erection of the main cables, lifting of the 57 steel box girder segments, and the respective quality monitoring of the production in China.
At the same time the construction of the highly demanding balanced cantilever post tensioned North Approach Bridge will commence (photo 1) as well the installation of the T-beams for the Southern Approach Bridge. Handover of the new bridge to the Mozambique Nation is scheduled to take place early 2018. ☒

11

12

A sleek and open bridge design

Queen Máxima

In 2016 Mobilis TBI completed the Queen Máxima bridge, an energy-neutral tail bridge named after the Dutch queen. It was commissioned by the city of Alphen aan den Rijn and crosses the river Oude Rijn. The implementation of energy-neutral requirements was a unique selling point in the offer of Mobilis TBI. It is achieved by the installation of a field of solar panels that can generate the energy for the moving of the steel bridge parts, lighting and all other equipment.

The Queen Máxima bridge (photo 1) has three adjacent bridge decks, in which two parallel tail bridges are situated. The tail bridge at the east side is connected to one of the decks for the road traffic and to the deck for the cycling/pedestrian lane. The tail bridge at the west side is only connected to a deck for road traffic (fig. 4).

The concrete land span bridges are 140 and 50 m long and the steel tail bridge decks are approximately 19 m. The tails contain the counterweights that ensure the smooth opening and closing of the bridge with a minimum of energy consumption. The project is characterized by its unique design, which tended to be as sleek and as open as possible.

Architectural design

The architectural design was very important to the client. Therefore, Mobilis TBI closely collaborated with the architect, Syb van Breda & Co Architects and designed a bridge that fully exploited the structural possibilities of the applied materials and met the wishes of the client to be elegant.
At first a single deck bridge was specified, so traffic lanes could be assigned freely. This would have resulted in a width of the deck of about 30 metres and a very dark area under the bridge. In the last stage of the tender the architect proposed creating three separate decks. This resulted in a more open construction and, very importantly, in the opportunity to build tail bridges.

If the deck had been about 30 metres in width it would have been impossible to build tail bridges. Instead, it would have been a bascule bridge. The difference between a tail bridge and a bascule bridge is, that the counterweights of a tail bridge are moving in the open air, besides the foreland bridges and, in closed position, they are visible above the deck (photo 2), while a bascule bridge has its counterweight under the bridge, mostly in a large cellar. As a result, a bascule bridge has higher lifting capacity than a tail bridge but due to the large cellar is less aesthetically appealing.

Another wish of the architect was to make a slender deck construction with no visible cross beams. So the concrete decks of the land span bridges have a continuous height of about 1.2 m in combination with spans of 27.5 m. On the locations of the intermediate supports, the bridges for the road traffic are supported by two conical columns. The bridge for cycling/pedestrians is supported there by one column. All columns have a circular cross section, with a diameter varying from 1.5 m at the base to 1.2 m at the top.

The column under the bridge for bicycles and pedestrians is placed eccentrically in the transversal direction. To make the construction stable this bridge is connected to the adjacent bridge for road traffic, using concrete beams with a circular cross section (photo 3 and fig. 6).

The design of the steel bridges was given a lot of attention. The steel river spans have the same deck height as the land span bridges and the tails are beautifully shaped. The use of tail bridges resulted in open structures for the concrete piers adjacent to the river.
Details also had the attention of the architect. This resulted in a

André Bouman, Nick Nass
Mobilis TBI

1 Queen Máxima bridge, Alphen aan den Rijn
credits photo 1 and 2: Syb van Breda & Co Architects
2 Tails with counterweights, visible above the deck
3 The stairs for pedestrians to reach the deck and the circular beam to connect the decks
4 Top view on the bridge

bridge

1

2

3

beautiful shape of the handrail, the lighting masts and the signs for the road traffic and the shipping traffic. Also the lighting on the bridge and under the bridge was designed by the architect.

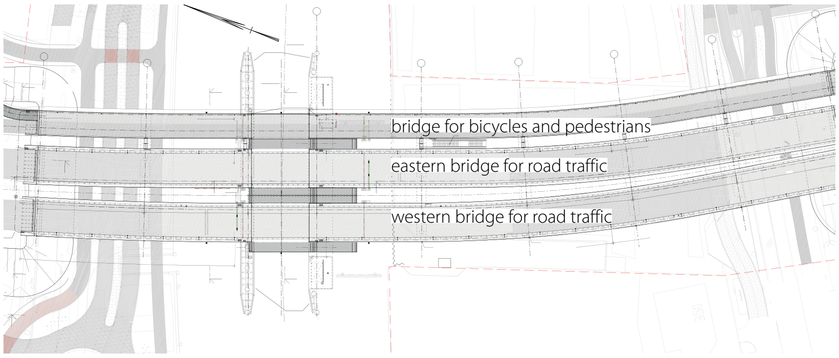
The space under the bridge at the south side will be turned into an area with foot paths and a water garden, with stairs as a connection for pedestrians to reach the bridge deck (photo 3).

Structural design

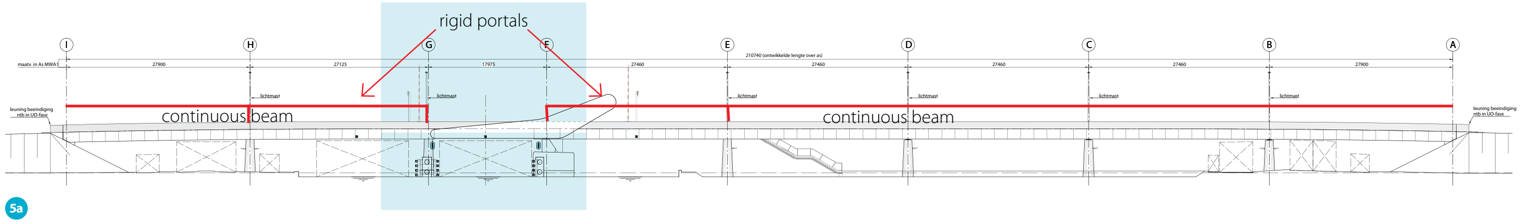
As can be seen in figures 4, 5 en 6, each of the three decks of the land span bridge in longitudinal direction is a continuous beam without visible joints at the locations of the intermediate supports. On both sides of the river the adjacent deck spans form rigid portals, together with the underlying columns. In this way, the movable bridge has rigid bearings in the horizontal direction and there is also a rigid support to withstand the load of a ship colliding with the river piers.
At the locations of the other piers, including the abutments, the bridge is supported by rubber bearings.

The five columns for each intermediate pier and the columns for the river piers are placed on a continuous foundation beam. At the intermediate supports, the foundation beams are supported by prefab concrete piles 450 mm square. At the river piers, steel tubes with a diameter of 610 mm are used for the foundation, in order to have enough strength to withstand a ship collision. Both prefab and steel piles are raking at 1:10. The abutments are grounded on reinforced soil constructions.

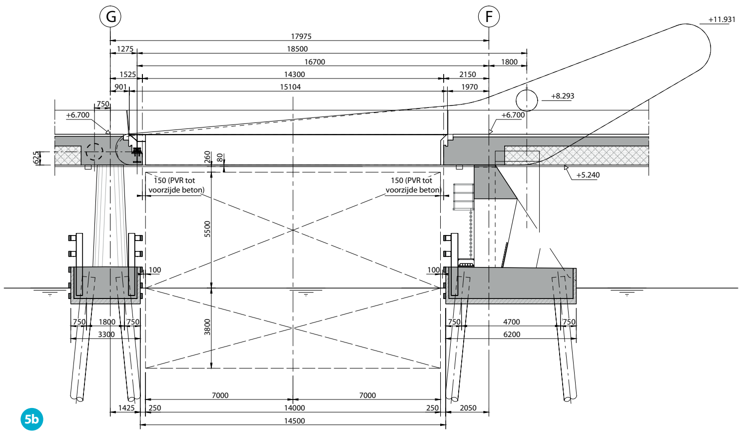
4



5 (a) Longitudinal cross section;
(b) river crossing with tail bridges
6 Cross section of the deck
7 The three dimensional finite element model designed with Scia Engineer
8 A view of the placing of the second movable deck
credits: Hollandia Infra and Sarens NL

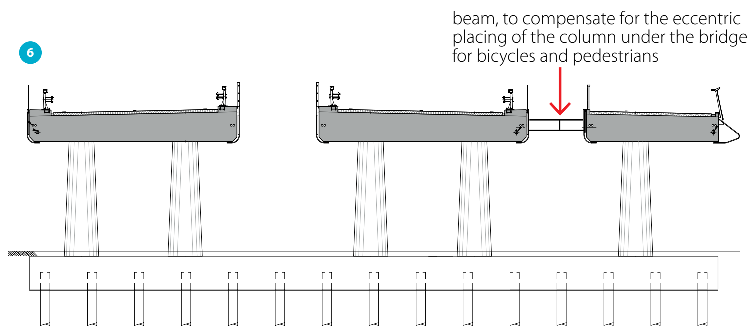


5a

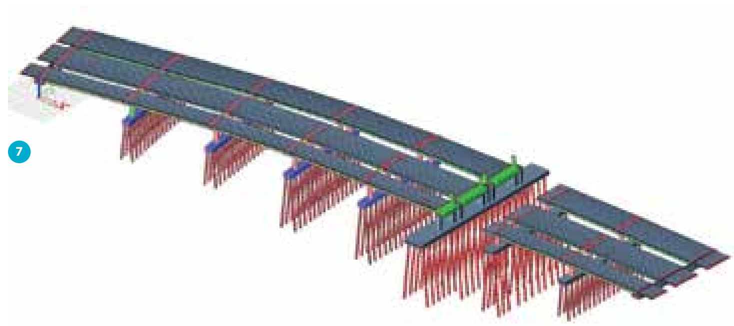


5b

6



7



The whole structure with all the loads was modelled in a three-dimensional finite element model with the Scia Engineer software (fig. 7).

Deck construction

The deck spans are made of prefab, pre-stressed beams with a cast in-situ concrete layer on top to form the traffic deck. At the location of the supports, prefabricated beams are connected by wet joints. In this way the prefab beams form a continuous girder and no cross beams are visible (photos 3 and 9).

Special load case

Additional to the traffic loads as listed in the Eurocode, the contract prescribed a special load, namely the transport of heavy concrete beams made by Consolis Spanbeton in Koudekerk aan den Rijn. Spanbeton, who also made the prefab concrete beams for the Queen Máxima bridge, is very happy with the realisation of the bridge, because now they can trans-

8

9

port without restrictions, beams with a length of about 100 m. The impact of this additional requirement on the structural design was minimal.

Execution

Because of the slender design, in a lot of places it was difficult to apply all the reinforcement that was needed. Especially at the locations of the wet joints (photo 10) a large amount of reinforcement was required.
As can be seen on photo 11, the circular coupling beam between the road deck and the pedestrian deck also needed much reinforcement. This beam has to compensate the eccentric placing of the deck for cyclists and pedestrians which results in bending moments and shear forces.

Extra care was taken when the steel movable decks (weighing 220 and 270 ton) were placed. The decks were placed using a crane with a fixed position on a pontoon. This combination as a whole was moved in the right position to situate the decks (photo 8).

Movement of the tail bridge

Each of the two separate movable decks of the river span has two tails supported by a column of the river pillar. The tails are filled with ballast, used as the counterweight. The movement of each bridge is enabled by two hydraulic cylinders, which are located in a recess in the columns of the bridge pillars. According to the contract the bridge is engineered to allow a non-availability for shipping of 3 days a year.
To withstand a collision between the bridge deck and a ship, the front of the deck is supported in the horizontal direction perpendicular to the bridge axis.

Environment

The Queen Máxima bridge is energy neutral and a sustainable structure; the slender design of the bridge required considerably less material than other types of bridges would have. The impact on the environment is therefore low. The bridge was opened for traffic on December 21, 2016. ☒

10

11

9 A view under the foreland bridges
credtis: Syb van Breda & Co Architects

10 A wet cast in situ connection combines prefabricated beams to a continuous deck construction above the piers
credits: Mobilis TBI

11 The reinforcement of the concrete coupling beams between the road deck and the pedestrian deck

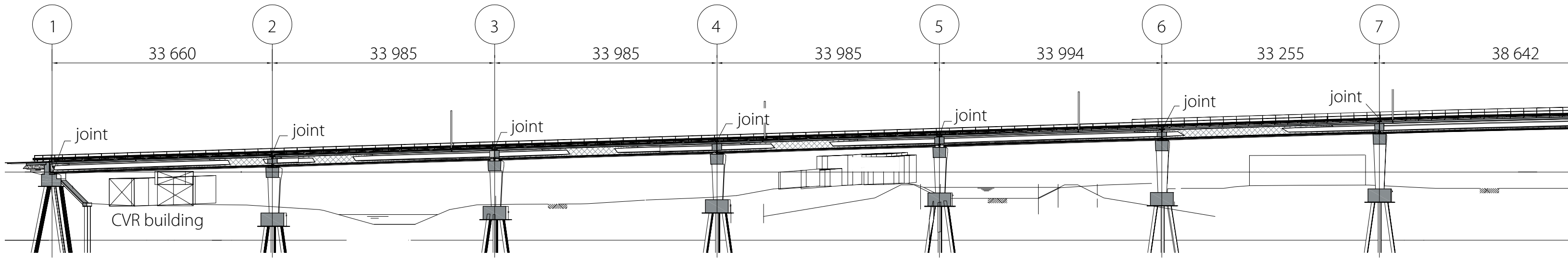
● **PROJECT DETAILS**

client Alphen aan de Rijn
contractor Mobilis TBI
parallel contractors Hollandia Infra (steel bridges)
Aannemingsmaatschappij Van Gelder (roads and earth works)
prefab concrete beams Consolis Spanbeton
architect Syb van Breda & Co Architects

New Gouwe bridge beside aqueduct will ease traffic on A12

Amalia Bridge Waddinxveen

Where the A12 and A20 motorways merge, before passing under the Gouwe aqueduct, both the flow and safety of road traffic become critical. In order to expand the road network around Gouda, under the name 'A12 Parallel Structure', the province has constructed two new roads: the Extra Gouwe Crossing and the Moordrecht Bow. Within the Extra Gouwe Crossing, the 'Amalia Bridge', designated also as 'ancillary structure KG', showed to be highly challenging; both structurally and in terms of fitting into the existing situation.



ir. Bas van den Berk
Heijmans Infra

1 Amalia Bridge over the Gouwe, Waddinxveen
2 Top view on the Amalia Bridge
3 Longitudinal cross-section

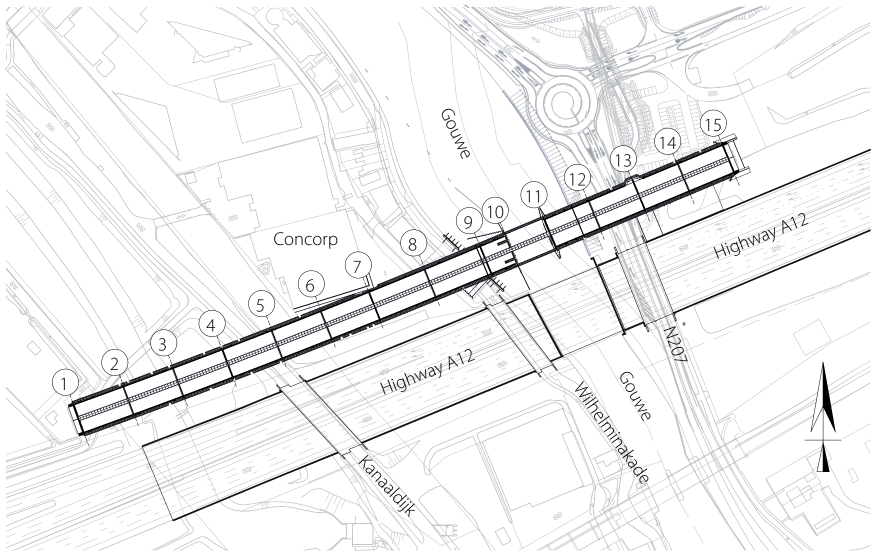
The road crosses the river Gouwe just north of the Gouwe aqueduct by means of a new drawbridge (photo 1). Apart from the Gouwe, this structure also crosses local roads (from west to east): Kanaaldijk (N484), Wilhelminakade and N207. The bridge consists of a movable section (the leaf, a steel structure), and concrete approach ramps to the east and west of the bridge (fig. 2 and 3).

Situation

Because structure KG lies immediately beside the existing Gouwe aqueduct, this object formed the de facto working boundary on the south side. With this in mind, and in order to minimize the impact of construction activity on the existing aqueduct, it was preferable to place the bridge as far as possible from the A12 (to the north). It was essential to take into account not only the visible parts of the Gouwe aqueduct, but also the subsurface grouted anchors. However, another barrier was formed by several commercial properties on the northern side. One of these properties is a confectionery manufacturer (Concorp), whose production depends on sensitive weighing equipment. Together with the aqueduct, these factors constrained the position of the bridge in the north-south direction. Likewise, the position of the western abutment was restricted by the presence of another existing object: a road-traffic control centre. This building is equipped with ICT equipment for control of traffic systems, and therefore has a critical function in traffic management. To avoid jeopardizing this building and its function, the western bridge abutment has been positioned at a sufficient distance. Fortunately, the location of the eastern abutment was not subjected to any positional constraints. What did determine the position was the maximum extent for the approach embankment in order to maintain the necessary landscape quality in the vicinity of the structure.

Design of deck structure

The total length of structure KG from eastern to western abutment is approximately 450 m. The bridge is divided into an

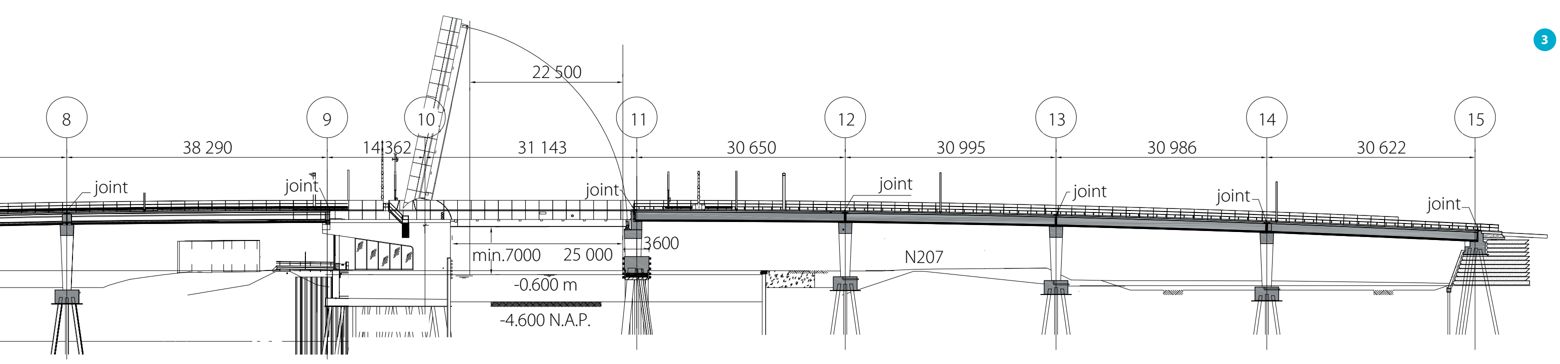


2

eastern approach ramp (124 m), the bascule pit and steel leaf (together 45.5 m) and the western approach ramp (280 m, all lengths approximate). The required 2 × 2 lanes, in combination with a median of about 3 m width (ensuing from the landscape plan), and bevelled fibre reinforced plastic edge elements (ensuing from the visual quality plan) result in a total deck structure width of approximately 21.6 m (fig. 4).

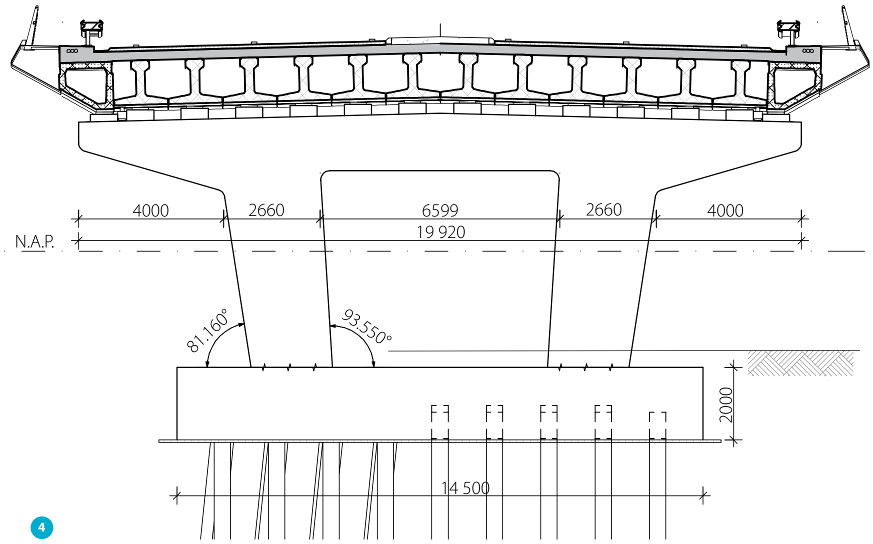
Construction method

To construct the approach ramps quickly and with minimal disruption to the surroundings, the deck structure was built using precast concrete beams. The first beams were placed in position from the side of the abutments, while the remaining sections were hoisted into position by cranes from each finished section of the deck. This working method meant that there were almost no interruptions to traffic on the underlying road network. Moreover, this also avoided the need for temporary structures to create a stable foundation for the cranes on the soft Gouda ground. Nevertheless, this did make it necessary to dimension both the deck and its substructure for the crane load.



3

4 Cross section
5 The land piers are designed as double T-pillars on a footing which is founded on precast concrete piles
6 T-heads that hold the rods to the ends of the deck supporting beams



4


5

Structure

The span dimensions are based on several preconditions. First of all, the beams could not be too heavy, due to the chosen construction method. Furthermore, the positional constraints arising from the current situation (i.e. traffic control building, Kanaaldijk, the necessary distance from the Concorp site, Wilhelminakade, intersection with the Gouwe and the N207) also played a significant role. Finally, it was desirable to choose a beam length that could be repeated as often as possible. For the eastern approach ramp this resulted in four spans of 31 m, and for the western approach ramp, six spans of 34 m and two spans of 38 m (approximate lengths). The transition from span to span is formed by non-rigid expansion joints and rubber expansion joints in a steel claw. The spans are constructed from precast concrete I-beams with tapered box girders at the sides.

Land piers

At rows 2 to 8 and 12 to 14 (fig. 2 and 3), the deck structure rests on land piers. These piers are designed as double T-pillars on a footing which is founded on precast concrete piles (photo 5).

6


This enabled the construction of a foundation structure that fits into the existing situation (grouted anchors of the Gouwe aqueduct) but is still wide enough to support the deck structure. The land piers are made of concrete with strength class C55/67.
To achieve sufficient load-bearing capacity in the soft Gouda subsoils, it was necessary to drive the precast concrete piles approximately 10 m into the firm sand layer. This resulted in a pile toe depth of 20 to 25 m below sea level. Piling and vibration analyses carried out beforehand indicated that pile-driving was feasible, and would not lead to unacceptable risks for existing objects, particularly the traffic control building and Concorp. The subsequent pile-driving work proceeded smoothly, and all piles were placed at the correct depth in the correct manner.
A challenge for the pier design was the fact that the outer box girders, each up to 38 m in length, had to be placed on a 4 m long cantilever on top of the pillars. As described earlier, it was also necessary to consider crane loads together with the hoisting weight of the precast beams. As a consequence, the upper reinforcement in the deck support beams incorporates several layers of Ø40 mm rods. These rods are mechanically anchored to the ends of the deck supporting beams by means of 'T-heads' (photo 6) in order to avoid complicated reinforcement detailing in that small space at the end of the construction.

River pier

Row 11 is the position of the support pier for the moving leaf of the drawbridge. This pier stands in the river Gouwe. The geometric and structural design of this pier is similar to that of the land piers. There is one major difference: this river pier has to be able to withstand a collision from water-borne traffic. As a result of the magnitude of this load, prefab concrete piles could not be used, so steel tubular piles were used for this foundation. To optimize the tubular pile dimensions, a more detailed analysis of the navigation channel and nautical traffic was carried out resulting in a reduction of the collision loads, which meant that

8

8 Bascule pit: the beam at row 9 is modelled schematically as a rib that is part of the 2D element, which itself is the roof of the bascule pit

smaller tubular pile dimensions would be sufficiently robust (photo 7).

Final phase of bascule pit modelling

The bascule pit (photo 8) was structurally analysed using a 3D schematic model in SCIA Engineer. In this model, the beam at row 9 is modelled schematically as a rib that is part of the 2D element, which itself is the roof of the bascule pit. This beam spans approximately 15 m, and bears the weight of the precast deck above. Because the forces acting in this beam are dependent on the stiffness of the corner columns that support it, the analysis was performed both with the cracked and uncracked columns.

Construction phase

The preceding paragraph concerns the final phase. However, the roof was not yet in position during the construction phase.

7

7 An analysis of the navigation channel and nautical traffic led to a reduction of the collision loads and smaller tubular pile dimensions

Obviously this had to wait until the steel leaf and ballast box were in place. Because the beam was still not finished at that point in time, and, therefore it lacked sufficient strength to support the prefabricated deck beams, a temporary support structure was built below the beam, which was later removed once the roof was ready. The same SCIA Engineer model was used for this phase as for the final phase, except without the roof. In addition to the models for the purpose of the overall structural analysis, a separate model was also made to determine the forces in the consoles. These consoles are cantilevered from the concrete wall, and are subject to dynamic forces from the moving parts. A push-pull rod transfers forces from the leaf to these consoles via the panama wheels: large pinion-driven gears that open and close the steel leaf of the drawbridge. A complicating factor in the structural analysis is the varying angles at which the forces act on the concrete consoles, depending on the position of the steel leaf. Another is the fact that the forces also switch from tensile (when the bridge is raised) to compressive (when the bridge is lowered).

On December 23 2016, the bridge was opened , providing the alternative route for traffic. As a result, one can choose such a route so that the traffic jam is avoided and with this, the problem of major bottleneck is solved. Although it was a challenge to fit the Amalia Bridge in the existing situation, Heijmans has managed to engineer and successfully build the bridge on time. ☒

PROJECT DETAILS

project Bridge over the Gouwe river (part of A12 Parallel Structure)
client Province of South-Holland
contractor Heijmans Infra
architect Zwarts and Jansma
structural design Heijmans Infra
first pile bridge KG 12 June 2015
bridge KG opening end of 2016


1

Approach bridge of Second Wuhu Bridge

The Second Wuhu Bridge connects the highways south and north of the Yangtze River in Anhui Province in China. The total length of bridge is about 50 km and is composed by two main parts: the main bridge over the Yangtze River, a cabled stayed bridge with a total length of 1622 m and a main span of 806 m, and approach parts connecting the main bridge with the highway from the both sides. The approach bridges, which are about 26 km in total length, are designed to be fully externally pre-stressed continuous segmental box girder bridges. It is the first application of this type of bridge in China, and maybe also one of the first in the world with such large amount of segments cast in the same project. The project is scheduled to be built in four years and should be finished by the end of 2017.

Design

Overall

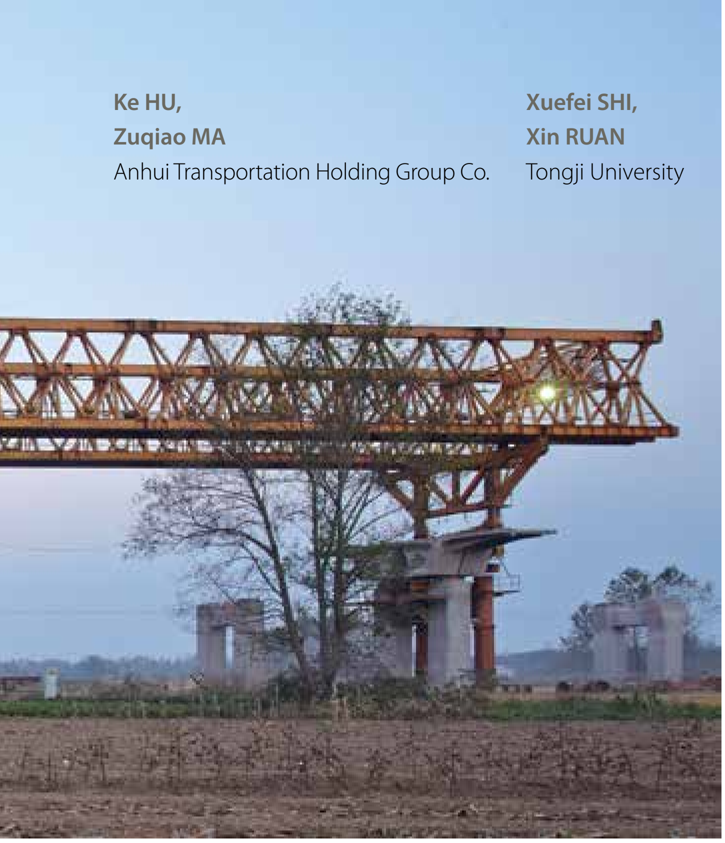
The approach bridges (from movement joint to movement joint) are designed to have three kinds of span arrangement. The basic one, consisting of six spans each 30 m long (6 × 30 m), is applied for bridges relatively close to the ground. The other two span arrangements are 5 × 40 m and 5 × 55 m. They are applied for approaches near the main bridge in consideration of budget balance between superstructure and substructure. There is a 200 mm wide movement joint between each span arrangement. Based on the road plan, the majority of bridges serve for six traffic lanes while a small part after a ramp serves four traffic lanes. So all the approach bridges can be distinguished as:

- 5 × 55 m with 6 lanes;
- 5 × 40 m with 6 lanes;
- 6 × 30 m with 6 lanes;
- 6 × 30 m with 4 lanes;

Span arrangements and representative ending spans of approach bridges are given in figure 2.

Ke HU,
Zuqiao MA
Anhui Transportation Holding Group Co.

Xuefei SHI,
Xin RUAN
Tongji University



1 The approach bridges of the Second Wuhu Bridge under construction
2 Deviator and diaphragm section details [m]

Cross section

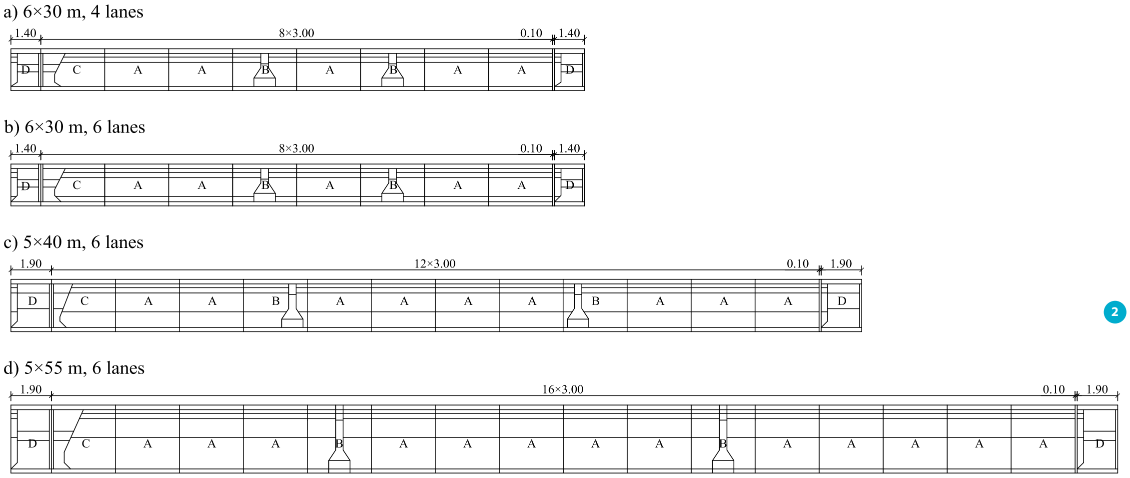
The cross section is designed to be a single-cell arrangement with side wings (fig. 3). The elements are made with C50 concrete. Constant depth is designed for each kind of bridge. The height for 55 m span is 3.25 m and 2.50 m for 40 m span. The height for 30 m span of both six lanes and four lanes are 2.00 m. The deck width of box girders of six lanes bridge is 16.25 m and 12.50 m for the four lanes bridge. Due to the application of fully externally pre-stressed tendons, no pre-stressed tendons are arranged in the web and flanges. The dimensions are optimized to reduce the segment weight. The minimum thickness of top slab and bottom slab is 220 mm and 200 mm respectively. The web is design to have various thickness and the thinnest part is 330 mm. Optimizations of the section dimensions helps reducing necessary lifting equipment capacity. The same parts of sections for four kinds of bridges are designed to be as much as possible similar to reduce the complexity of precast forms.

For four lanes bridge section, there are no transverse rib both inside and outside the box girder. For six lanes bridges, there are no transverse ribs inside the box due to its complex inner form, while transverse ribs are designed under both sides of the cantilevering deck to achieve the very long side wings. The rib section is inverted trapezoid for mold releasing.

Segment Division

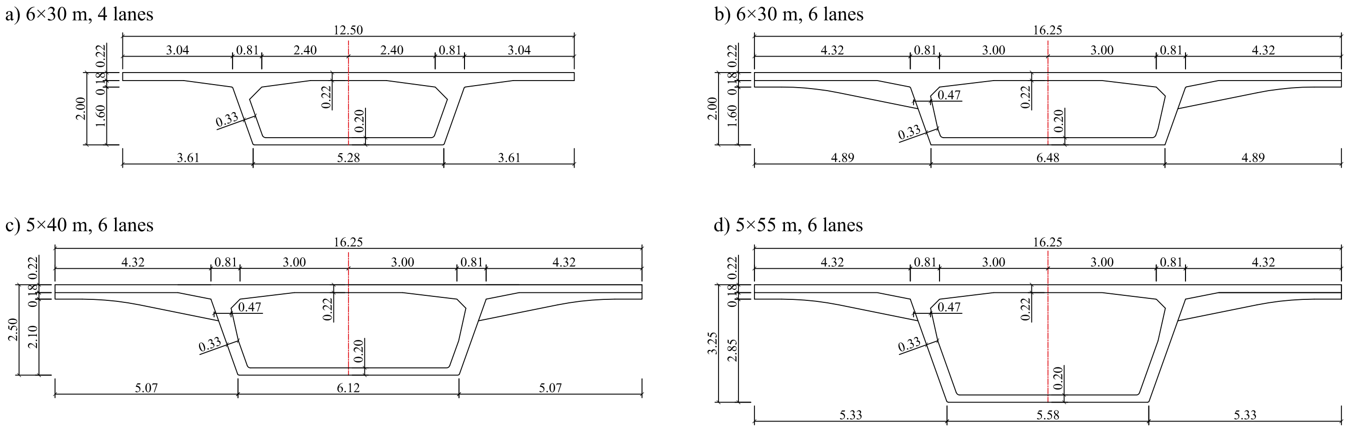
All approach bridges are designed to have only four types of box segment: standard segment (A), deviator segment (B), strengthening segment (C) and diaphragm segment (D) (fig. 2 and fig. 4). The length for the first three types of segments is 3.00 m. Considering the lifting capacity, the length of diaphragm segment is set to be 1.40 m. In this way, the heaviest weight of diaphragm segment is controlled to be within 100 t, due to the consideration of both lifting and transport machines' capacity, as well as the cost of enhancing the temporary erection road. Deviator segment is the place where tendons steer. The only difference for deviator segment from standard one is two more deviator blocks at inner box. Strengthening segment and diaphragm segment are designed for the anchorage of tendons. The huge and concentrated force is a big challenge for structural safety at these two types segments. Both theoretical calculation and FEM simulation have been applied to optimize the shape and ensure safety and durability.

For all approach bridges, the span at the end contains several standard segments, two deviator segments, two diaphragm segment and one strengthening segment (fig. 2). The span in the middle contains several standard segments, two deviator segments and two diaphragm segment. In total, there are 20032 segments for the whole approach bridge project.



2

3 Ending spans of approach bridges with their segment division
4 Standard section details [m]
5 External tendon profile
6 Storage of the segments
7 Construction site at the shore of the Yangtze River



3

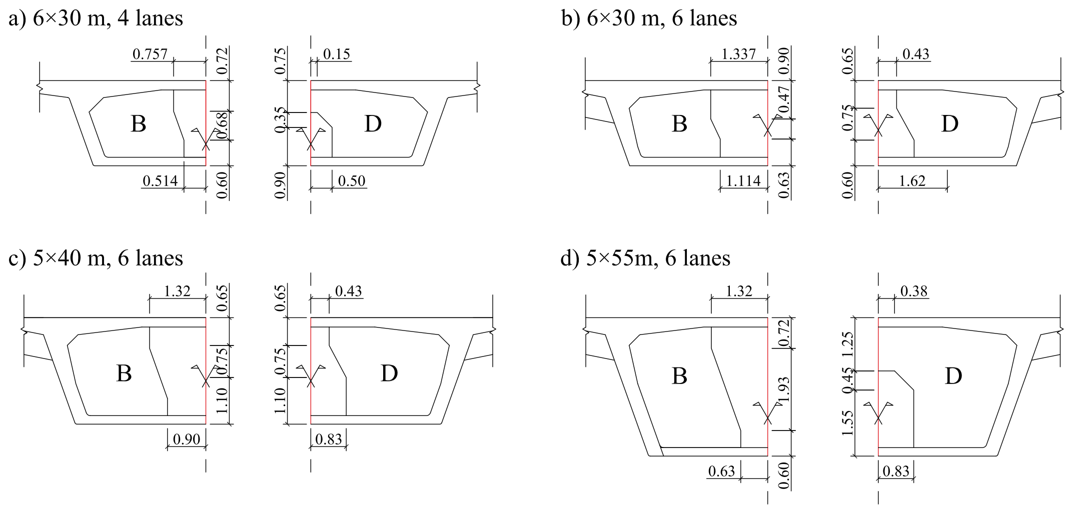
Prestress

The approach bridges are fully externally prestressed in longitudinal direction; the maximum prestressing stress in the girder is about 12 MPa. For each kind of bridge, there are eight tendons in one span. They steer at the same longitudinal position and are anchored at a different height of anchorage block (fig. 5). For approach bridges with 6 × 30 m span arrangement, each tendon is composed with 25 strands of high strength steel strand for end span and 26 strands for middle span. The nominal area of each strand is 139 mm^2 and the design yielding stress of strand steel is 1860 MPa.
For the girder top slab, there are transverse post-tensioned tendons to provide no more than 10 MPa compression stress in the concrete deck to make sure the concrete slab will not crack under traffic load in the operation period.

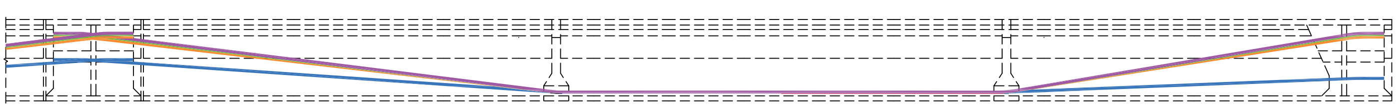
Joint

Epoxy is used for the joint between each segment. The material itself has a minimum compression strength of 60-70 MPa and serves a good function for segment connection. Although it takes time to daub when erection on site, it helps to enhance the durability of the joint. Each span has a 150 mm concrete wet joint as well. The concrete grade is the same as the precast elements. It is cast on site in case of segment cumulative misalignment.

4



5



6

Construction

Precast

The 20032 segments are cast in four casting yards with 90 cast cells. Steel rebar is assembled first at steel jigs in the cast yards. The details of steel cages are simulated accurately through CAD and the assembling sequence is optimized to ensure quality and speed of work. Then the segments are cast in the precast cells through short-line method–a new segment is match-cast against the preceding segment.

A cycle time of one standard segment per day per casting cell can be achieved by steam curing.

Transportation

A cast segment is lifted and transported to the storage yard through gantry crane. Due to the time schedule and site constraints, double layer stacking is applied and the storage sequence is carefully planned to avoid additional lifting work (photo 6). The storage area is close to the bridge and the segments are transported within the right-of-way of the approach project.

Erection

Span-by–span erection is applied as the approach bridge construction method (photo 1). Both overhead gantry and underslung gantry are used in the project. A cycle time of five days per span can be achieved by overhead gantry, and 4 to 4.5 days per span by underslung gantry.

Internet platform

To ensure the efficiency and accuracy of the construction, an internet platform has developed. The platform stores and delivers all the information and is involved in both segment precast and erection. With the help of the internet platform, only one engineer needs to be prepared for data handling and the response time of checking and providing precast or erection coordinate is within 10 min. The work efficiency increased three times as compared to previous similar bridge construction projects in China considering both the time and staff savings.

7

Conclusion

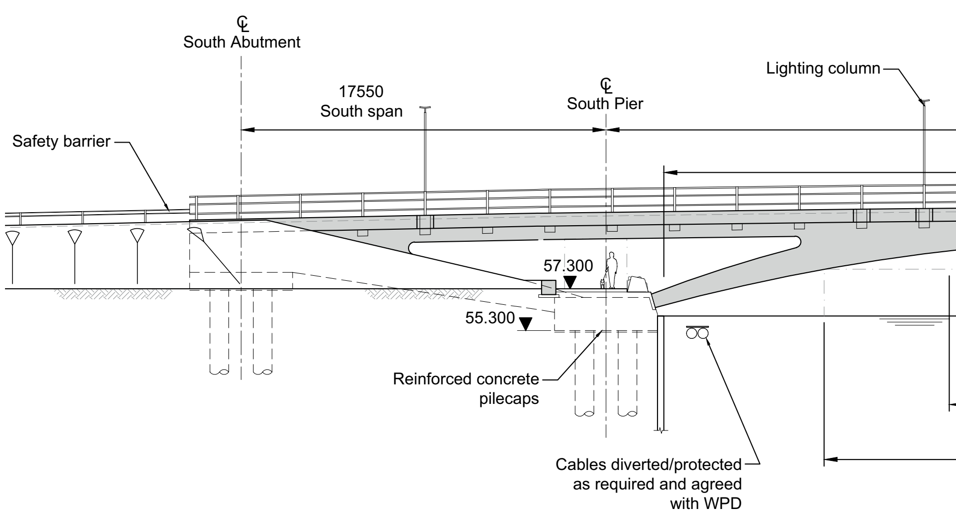
The approach bridge of the Second Bridge of Wuhu Yangtze River Highway Bridge is a good practice of industrialized construction. The whole project is scheduled to be finished in 2017. Application of precast segmental externally prestressed concrete bridge makes the design standard. Repetitive construction procedures reduce costs and construction time about 10 to 20 percent compared to traditional cast-in-place concrete girder scheme. Factory production enhances quality and minor site disruption. The project is a model for large-scale infrastructure construction (photo 7). ☒

Steel-concrete composite flat arch bridge

As part of a new University Campus development in the city of Northampton (UK), a new road access bridge was required (photo 1 and 2). The aspirations and the planning requirements were set to keep the character of the existing landscape while creating an appropriate landmark structure for the new campus. The client's specimen design included a concrete flat arch bridge spanning 49 m with a shallow rise of 3.7 m above the navigable river. An alternative design was developed using a steel-concrete composite structure solution for the deck. The awarded tender solution includes 220 tons of welded steel plates to form a shallow and flat arch structure.

The bridge design had to address a number of site specific challenges as listed below:

- accommodate a road alignment that would tie-in with Bedford Road Junction and the new campus;
- span without any support in the River Channel (48.5 m min) and provide 18 m wide navigation channel with 3 m clearance above normal water level;



Riccardo Stroscio
Tony Gee and Partners LLP

1 Steel-concrete composite flat arch bridge
2 Aerial view - November 2016
credtis: Commission Air
3 Reference design - elevation

2

- provide a clearance envelope beyond the navigation requirements to meet the flood risk design criteria with a 1 in 200 years return period;
- minimize disruption to the extensive number of buried services (11kV cables across the river, 33kV and 132kV in the North bank);
- maintain river navigation during construction period;
- minimal disruption to the river to maintain the ecology and biodiversity;
- create a safe and pleasant pedestrian/cycle environment along the river banks;
- address cost, statutory authority and build ability issues.

Reference design scheme

The client's engineer proposed an 89.7 m long structure comprising a single 49.0 m skew span concrete shallow arch structure supported at each river bank with 17.55 m approach span on each side connecting to the bridge abutments (fig. 3). Comparison of the shallow arch bridge in Northampton with other, similar span arch bridge structures is given in table 1.

Table 1 Arch bridges – comparison of similar spans (road bridges)

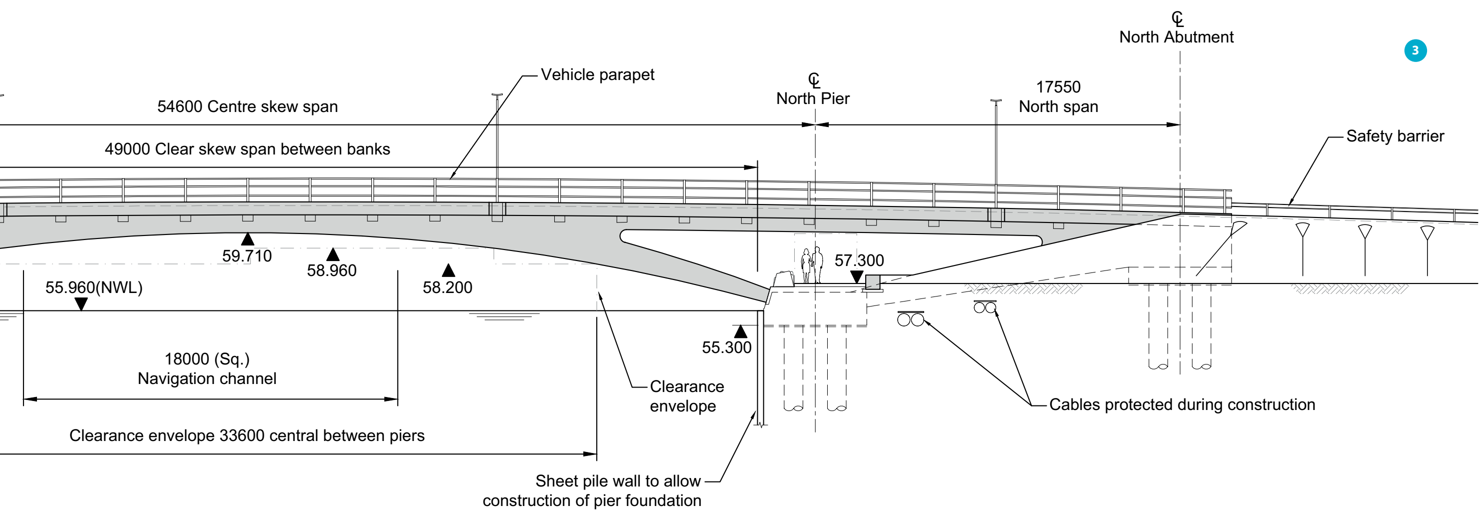
bridge	Tavanasa	Vessy	New Runnymede	Northampton
Year constructed	1906	1936	1979	2016
Span (arch) [m]	51.0	56.0	54.6	50.3
Rise [m]	5.57	4.77	6.96	3.61
Span/Rise	9.2	11.7	7.8	13.9
Structural depth[1] [m]	0.83	0.83	1.80	1.19

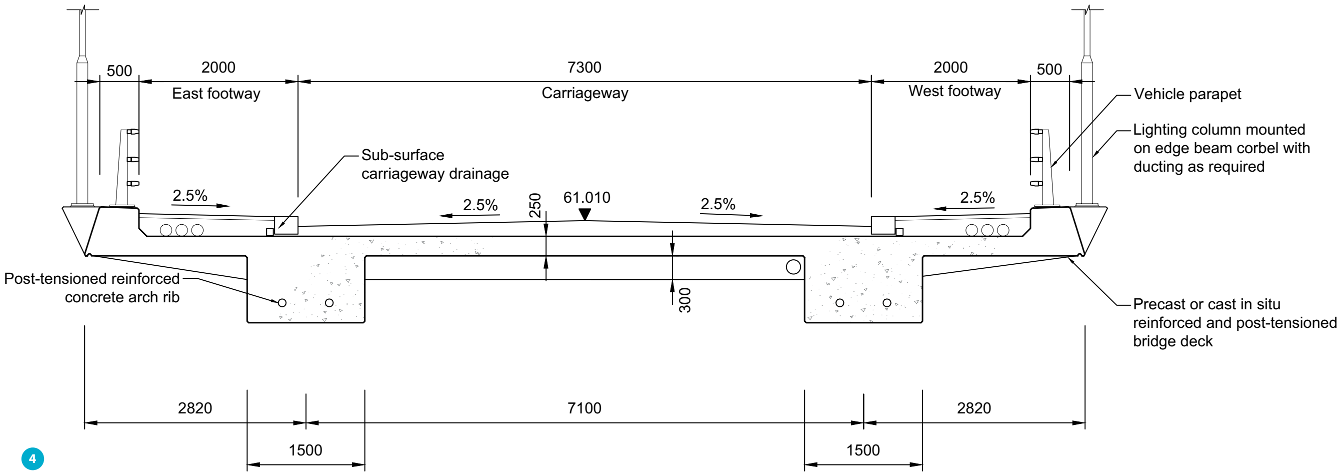
[1])*Structural depth at mid-span*

The proposed structure included a concrete ladder deck with deck beams supported on precast concrete arch ribs and forming balanced cantilever frames spanning from piled foundations on each bank (fig. 4).

Tender design

The constraints imposed by the planning documentation do not allow any deviation from the very flat arch requirements with a span to rise ratio of approximately 13. However, the geometry of the





4

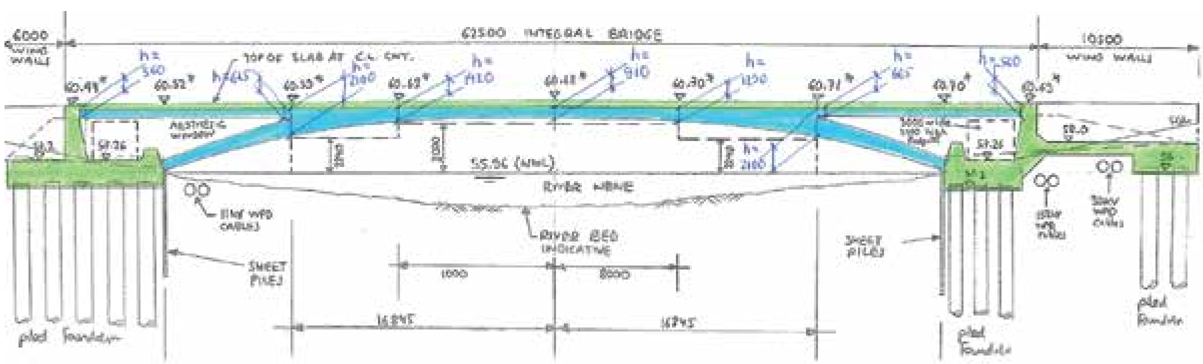
4 Reference design – deck cross section
5 Alternative tender - structural arrangement
6 Alternative design – abutments load path

aesthetic window on the riverside walkways enable to shorten the extent of the side spans and the overall deck length can be reduced to 64 m. The reference design 'piers' are thus being replaced with larger abutments and wing walls on each bank (fig. 5).

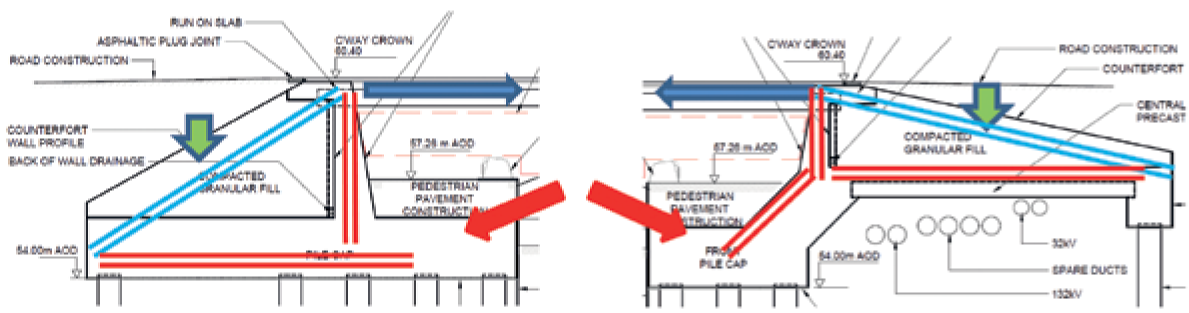
The setting-out of the foundation is derived from the layout of existing buried utility cables to prevent the need for any diversion. The North abutment is to be constructed in stages with the use of precast concrete elements in order to span the existing cables. For maintenance benefits, a fully integral bridge was chosen. The overall length of 64 m and the 20° skew remain in line with the geometrical range recommended in the UK for integral bridges. The depth of the arch is shaped with a curved intrados to provide the required stiffness and meet the clearance enveloped above the river. Deeper girders are provided at the arch-deck intersection zones tapering down to a shallower section at mid-span and also towards the springing levels. The deck structural depth of the side span is constraint by the road alignment and the clearance to the river side walkway. The bending moment, shear and axial force distribution is shared between the deck and the arch in proportion to their relative stiffness. The overall overturning moment to the abutments is reduced by the stabilising effects from the granular backfill material preventing any net tension in the piles.
The horizontal pull-out forces from the deck and the integral connections are anchored into reinforced concrete buttress walls within the abutments and the loads are transferred into the pilecap via strut-and-tie models in the combined reinforced concrete counterfort-wing wall systems (fig. 6).

The resulting net effects at the base of rigid pilecaps are resisted by the piles transferring the load effects into the ground. A review of various deck cross sections alternatives concluded that a ladder type deck option using steel main and secondary

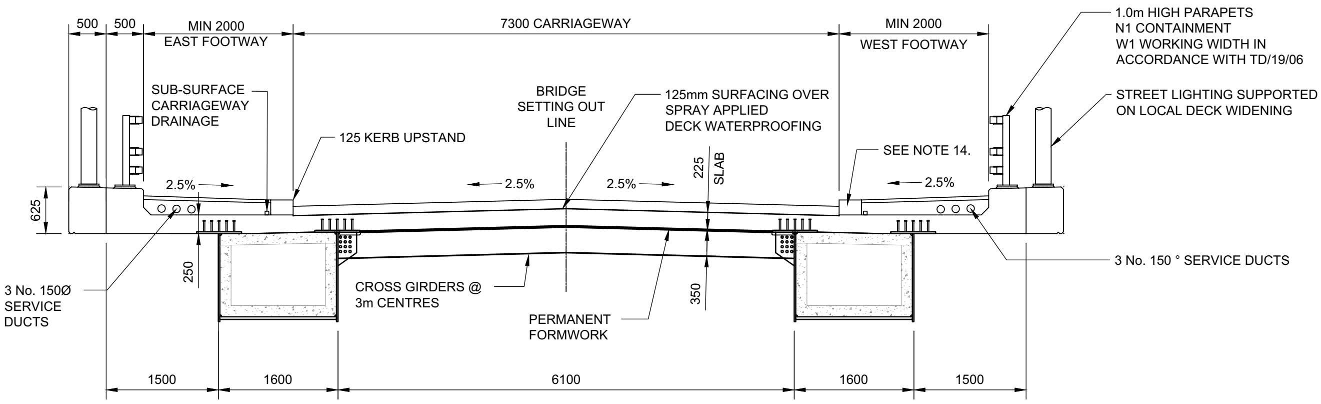
5



6



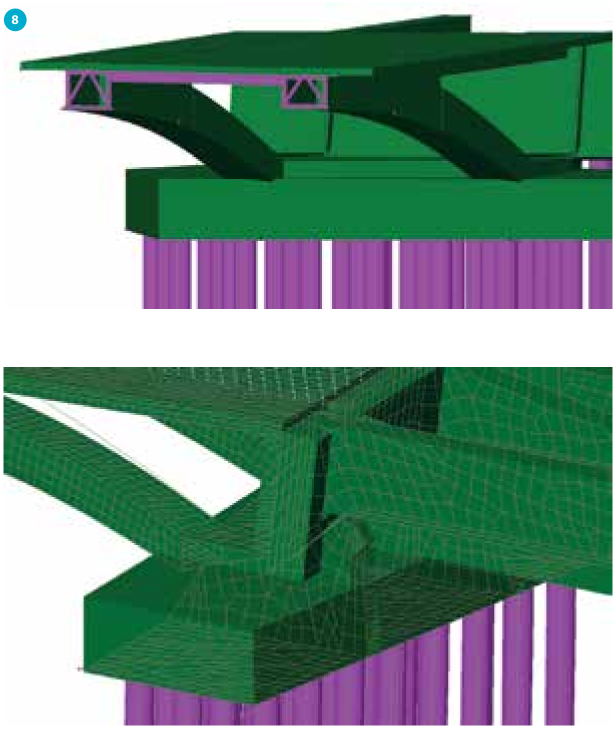
7 Alternative design – deck cross section
8 Extracts from computer idealised FE model
9 First the steel arch installation



7

beams offered the optimum solution (fig. 7).
The cross girders are set at 3 m centres orthogonal to the main beams to simplify the connections, the seating details for the concrete permanent formwork panels and the fixing of the deck reinforcement. The 1500 mm deck cantilever allows the adoption of a 250 mm reinforced concrete slab and the girders can be set back to remain relatively in the shadow of the deck slab. Furthermore, it allows the use of modular proprietary cantilever falsework system that facilitates the construction of edge projection of deck slabs. The cross girders are shaped to provide a 225 mm constant concrete slab thickness with transverse cross fall in line with the carriageway requirements. The main design constraints for the foundations are the proximity of a significant number of buried cables and the requirement to keep a 48.5 m minimum clear span between the banks. Sheet pile walls form the river side of the pilecaps and provide a scour protection to the foundations as well as offering a temporary cofferdam during construction. They are set within the minimum clearance from the centre line of any buried cable. Each abutment is supported by 750 mm diameter continuous flight auger (CFA) piles to provide the most efficient and economical solution for the given ground conditions. One of the main advantages of the steel bridge solution is the lighter weight, which means it can be erected from cranes on both banks without the need for extensive temporary works and associated costs.

8



9

Detail design

During late spring 2015, the contract was awarded to Volker-Fitzpatrick based on this alternative proposal. The detail design phase started in early summer 2015 by setting an idealised 3D finite element model to include the steel-concrete composite superstructure (arch ribs, deck girders and slab) and the reinforced concrete abutments. A series of computer models with shell and beams elements were produced to extract the stress build up within the composite sections model considering long term, short term and staged construction effects from arch ribs erection, deck girder installation, backfilling behind abutments and the concrete deck castings sequence (fig. 8).

The concrete deck slab connecting to the integral abutments was also designed as a concrete tension member with consideration of tension stiffening effects in line with EN1992-2 in order to control crack widths at serviceability limit state.

10 Steel arches and tie beams completed

10

The soil-structure interaction analysis and pile load derivation was computed by close calibrations between structural/global models and separate geotechnical analyses. Upper and lower bound pile stiffness were used for the global analyses of the bridge. Combined with a pile test on site, the detail analysis confirmed the tender design solution and offered a ten percent reduction in pile length for the 63 CFA piles.
The main steel girders are shaped with a variable depth and a constant width of 1600 mm. A stiffened steel box section was generally adopted except for the tie beams where a pair of 560 mm deep plate girders was designed to respect the pedestrian clearance on each banks. The combined arch-deck section is an open top steel box section keeping steel continuity of the web and the bottom flange. At its deepest, the girder is 2150 mm. The lower part of the arch is 600 mm deep at springing levels and it tapers up to 1400 mm before the arch meet the deck. It was decided to infill them with self-compacting concrete (SCC) grade C32/40 with slump-flow class SF2. Similarly, the 15 m long central portion of the span where the depth of the steel is shallower than 1200 mm, a 150mm thick concrete lining was specified to the inside faces of the steel box with lightweight expanded polystyrene void former in order to protect the steel against corrosion. The benefit of concrete lining in midspan and SCC infill within the lower part of the arch is that it provides additional robustness against accidental vessel impact actions. Stability of the arch girders and resistance to buckling was analysed using a 3D finite element analysis model and reviewed with hand calculation methods. As individual arches where assembled on site, a temporary bracing arrangement was required until the tie beams could be connected to the abutments and cross girders installed.

Construction

Piling started on the North side in December 2015 and the reinforced concrete pilecaps and abutments were completed by July 2016. With the backfill completed, the construction site could start preparing for the main steel lift with one 500 t capacity mobile crane set behind each abutment. In August, the main girders were delivered to site in 25 m long, 37 t sections. Each half arch could then be lifted individually and lowered into the predefined concrete pocket within each pilecap allowing the central bolted splice connection to be completed using tension control bolts and accessed using a mobile elevating work platform on a floating pontoon (photo 9).

The same operation was repeated for the adjacent arch girder and then followed with erection of the lighter tie beams (13.5 t each) and cross girders.

The concreting phase could follow with completion of the abutments and the parapet edge beams allowing installation of the road restraint systems. With the main structure finalized, the road pavement, footway verges and expansion joints could be installed and the bridge construction was completed by the end of 2016. ⊠

PROJECT DETAILS

client University of Northampton
design and build contractor VolkerFitzpatrick
civil and structural designer Tony Gee and Partners
highway designer Peter Brett Associates
independent checker Ramboll
client's Engineer CH2M
project manager MACE
bridge architect MCW
landscape architect LUC
steel Fabricator Briton Fabricators
piling contractor Van Elle
temporary works designer Tony Gee and Partners

REFERENCES

1 Lewerer, J.-P., Peçon, Y. (1993). La Souplesse d'un ressort tendu: le pont de Robert Maillart à Vessy. *Faces-Genève*, No 30 (1993/94), pp. 50-55.
2 Smyth, W.J.R., Benaim, R., Philipott, D.H., (1980). The new Runnymede Bridge. *The Structural Engineer*, Volume 58A/No. 1, pp. 5-11.
3 Solca, J. (1914). Die Rheinbrücken bei Tavanasa und Waltensburg. *Schweizerische Bauzeitung*, No 63/64, pp. 343-346.

Construction of an Arch Bridge by Lowering Method

Kano River Crossing Bridge

The Kano River Crossing Bridge is a reinforced concrete deck, arch bridge under construction in Shizuoka Prefecture of Japan: a bridge with a length of 171 m and an arch span of 110 m. Since the Kano River is one of the best-known pristine rivers in Japan, environmental considerations required that piers should not be erected in the river. This type of structure was selected based on conditions of construction and seismic resistance considerations.

Developed in Italy in the 1950s, the lowering construction method has been used to build the arch bridges, with either concrete (photo 6) or steel members (photo 7) used for the arch members. In Japan, having many steep gorges, this method evolved as an effective method to construct arch bridges with spans of about 100 m. The method is not used for completely steel arch bridges. Also, for larger span, other construction methods such as the suspension support method may be adopted because of economic efficiency.

Yuki Kaminaga, Takeshi Nakagawa, Hiromi Hosono
Sumitomo Mitsui Construction Co.
Hidetoshi Ichikawa, Masanao Kajiura
Ministry of Land, Infrastructure, Transport and Tourism

1 Kano River Crossing Bridge after completing closure
2 General view of the whole bridge
3 Construction procedure
4 Comparison of lowering construction equipment
5 Lowering construction procedure

Because of the ground conditions at the site and in order to reduce the weight of equipment and subgrade reaction from the construction method, a lowering method with steel arch ribs was used for the erection of the Kano River Crossing Bridge (fig. 2). These temporary steel members for constructing an arch structure are called Melan's rigid reinforcement. By using lighter steel members (in comparison to concrete ones), the tension in the lowering cables is reduced. The steel member, with a total weight of about 360 tons, is manufactured in the factory and consists of elements with a length of about 6.0 m each. They were assembled by bolt joining at the construction site. After constructing partial arch rib members quasi-vertically at each abutment, the arch is formed by using cables to lower the members, rotating them to the specified position using the base footing as the center and closing the arch.

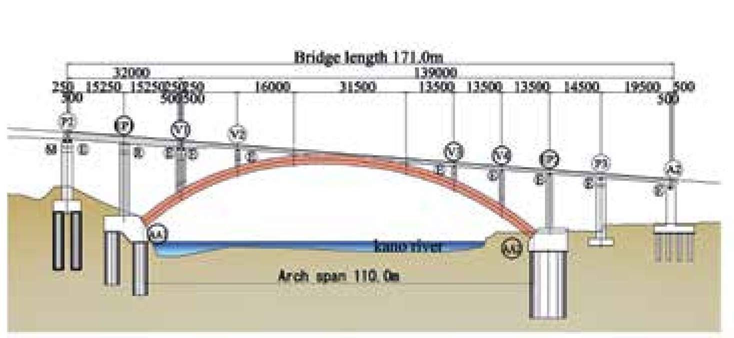
Construction Procedure

Figure 3 illustrates the entire construction procedure. The Melan's rigid reinforcement is erected using the lowering construction method to build the arch. The springing points, which are the base footings on both ends, are encased in concrete using falsework after closure of the arch. The form traveler is mounted above the springing point and the rigid reinforcement is encased in concrete, one step at a time on both sides, to complete the arch rib. Thereafter, vertical members and stiffening girders are constructed using scaffolding and falsework mounted on the arch rib to complete the bridge body.

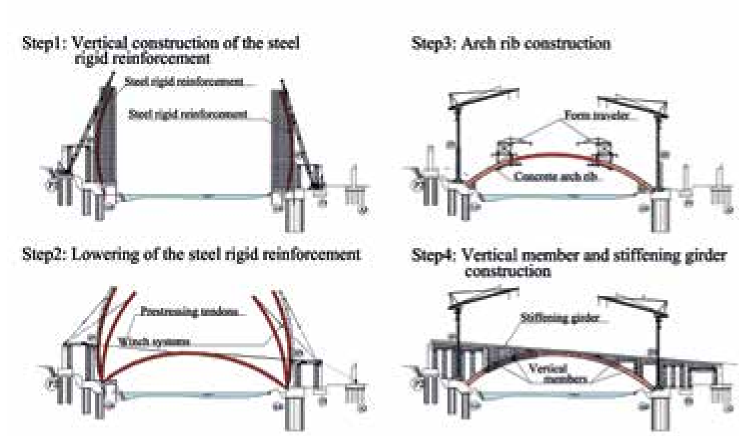
Lowering Construction method

There are three possible methods for lowering construction (fig. 4). When only a lowering jack is used on the prestressing tendons (fig. 4, option 1, and photo 11), safety issues arise concerning the wedge anchor of the strands because the tendon tensions are small at the initial stage of lowering. A method to pull in the rigid reinforcement with prestressing tendons from the opposite abutment is available, to ensure the minimum tension necessary to securely anchor the wedge. In this case, the equipment tends to be excessively large and construction time tends to be longer because of the difficulty of controlling tension during lowering. When only a winch system is used (fig. 4, option 2, and photo 8), several large winches are required, which makes it uneconomical because of the large-sized equipment, although construction time is shorter. Therefore, by conducting the lowering method with prestressing tendons using a winch system at the initial stage and a lowering jack at a later stage (fig. 4, option 3, and photo 10), both safety and economy can be attained.

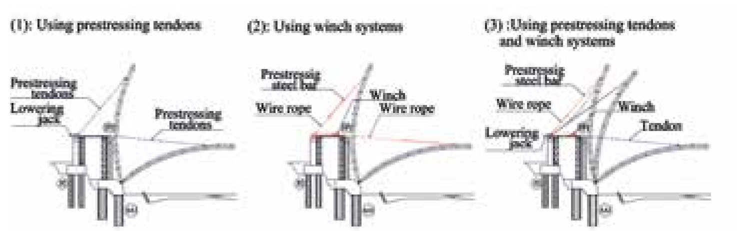
2



3



4



5

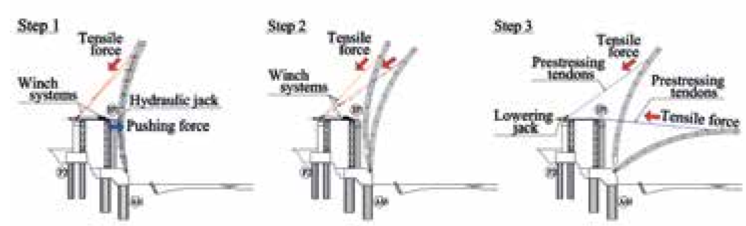


Figure 5 shows the lowering construction procedure adopted for this bridge, which uses different equipment according to the stage of lowering. In step 1, the rigid reinforcement is rotated forward by pushing with the jack since the center of gravity is at the end post side. During this time, the rigid reinforcement is being pulled by the winch system so that it does not fall suddenly while rotating. The procedure switches to step 2 when the center of gravity of the rigid reinforcement is in front of the

6

6 Lowering construction using concrete members
7 Lowering construction using steel members
8 Lowering construction procedure
9 Lowering with the winch
10 3 ton winch
11 Lowering jack system

center of rotation of the base. By loosening the winch cable in step 2, the rigid reinforcement is lowered by rotation under its own weight (photo 8 and 9).

When the angle of the rigid reinforcement is 18° and the tension is about 600 kN, the winch system is replaced with the jack system. Photo 10 shows step 3 of the lowering construc-

7

8

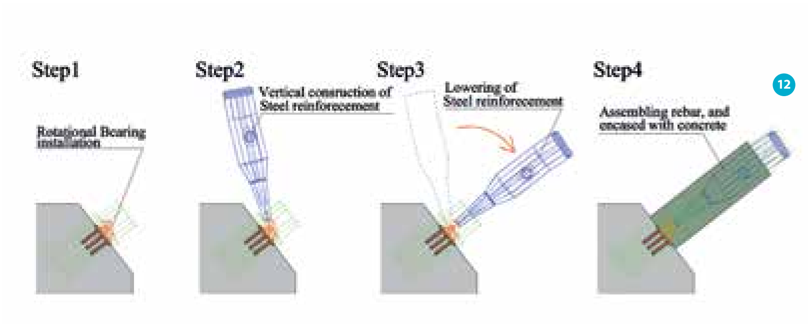
9

tion; Photo 11 shows the jack system. The jack system is composed of lowering jacks, prestressing tendons, hydraulic system and control panel. Two lowering jacks were installed at the rear of the concrete block set on top of the pier and were centrally controlled together from a control panel using two electric pumps. Prestressing tendons tension was at its maximum at 3040 kN immediately before closure. Two prestressing steel strands with 19 Ø15.2 mm strands were used for the prestressing tendons to obtain a factor of safety of more than 2.5 against rupture. Approximately 17 m of prestressing tendon was launched by the jack system during the lowering operation. A total of 110 strokes were used for launching, with 150 mm per stroke.

10

11

12 Construction procedure of the springing point
13 Rotational bearing installation
14 Rotational test



12

13

14

Rotational bearings with through pin

Figure 12 shows the construction procedure of the springing point. At first, rotational bearings are installed. Next, steel rigid reinforcement is erected vertically and lowered by rotation. The rotational bearing and the steel rigid reinforcement are then encased in concrete.

Lowering the rigid reinforcement to its position accurately is important, since the form of the rigid reinforcement after lowering construction will affect the form of vertical members and stiffening girders and the form of this arch itself after the arch rib is completed. The accuracy of the rotational bearing installation is critical, since it serves as the center of rotation of the rigid reinforcement. The two bearings at each side were connected by a pin to reconcile their axis of rotation. Moreover, the bearings were joined at the plant, transported and erected together at the site to improve installation accuracy (photo 13).

After completing installation of the rotational bearings, temporary steel members were installed on the lowering bearings to perform a test for checking the installation accuracy of the rotational bearings. Photo 14 shows the confirmation test. By actually rotating the front while suspended with a crane, it was confirmed that there were no problems with the installation positions of the rotational bearings. This measure reduced the error in the level direction after completing the lowering to about 20 mm.

Central closure

Central closure was carried out after the rigid reinforcement members on both sides were rotated and lowered to the specified height. The central closure spacing was 50 mm. To handle the gap between bolt hole positions on both sides of the rigid reinforcement, splice plates were plant fabricated after measuring for the actual hole positions. Immediately after lowering was completed, the rigid reinforcement on both sides was connected by temporary splice plates and bolt hole positions were measured during the night, when temperatures are stable. Photo 15 shows the central closure; Photo 1 shows the panoramic view after lowering was completed.

Arch rib encasement work

Encasement work for the rigid reinforcement involved encasing the first block at both ends with concrete from the falsework, and then assembling the form traveler over the arch rib. Figure 16 shows the structural drawing of the form traveler. The form traveler weighs 1050 kN. It moves by tensioning a prestressing steel bar with a 500 kN jack installed in front and propelling itself forward on the rail installed over the arch rib.

To make the arch rib structure and construction more efficient, a new cross sectional structure was adopted where the rigid reinforcement is not filled with concrete and is placed outside the web.

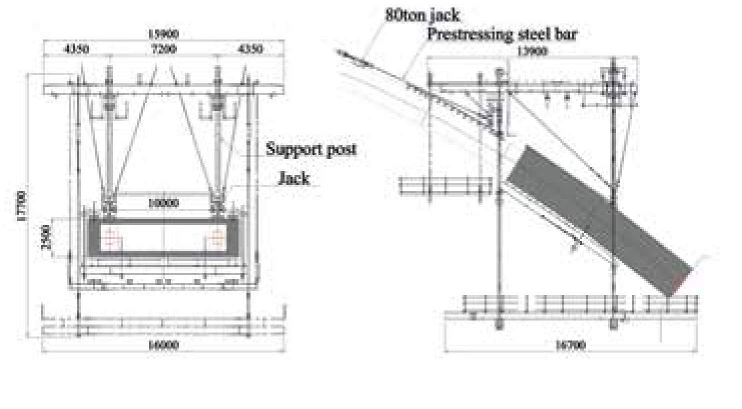
15 Central closure
16 Construction procedure with the form traveler
17 Structural details of arch ribs
18 Comparison of cross section of arches
19 Conceptual rendering of the bridge

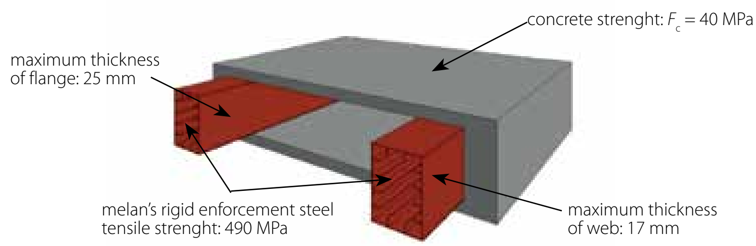
15

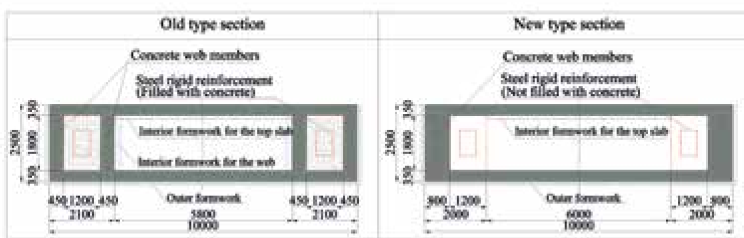
Melan's rigid reinforcement is a temporary steel member, and the interaction between the steel member and the concrete member is not considered. The maximum thickness of the member is 25 mm for flanges and 17 mm for webs, and steel tensile strength is 490 MPa (fig. 17). Figure 18 shows a comparison of the cross section resulting from the old approach with that resulting from the new one. By baring the rigid reinforcement inside the box girder, web thickness could be freely set to its structurally required thickness and became unnecessary, thereby making construction work simpler and more efficient. This resulted in lower arch rib weight and improved seismic resistance.

Conclusion

Lowering construction was employed for the steel rigid reinforcement of the Kano River Crossing Bridge. Concrete encasement of the arch ribs is currently underway. Figure 19 shows the conceptual rendering of the completed bridge. Construction of this bridge is scheduled for completion in February 2018. ☒

16


17


18


19


● REFERENCES
1 Makoto, K., Koji, T., (1998). Prestressed Concrete Slant-Legged Rigid-Frame Bridge Constructed Using the Lowering Method, Prestressed Concrete in Japan. Prestressed Concrete Engineering Association, XIII FIP Congress 1998, Amsterdam, HOLLAND, pp 33-36.
2 Hiroyuki, U., Naoki, N., Mitsuyoshi, N., & Osamu, A. (2015). Design and Construction for Durability and Maintainability of Reinforced Concrete Arch Bridge with Curved Girder. IABSE Conference Nara, Japan.

1

After 2,5 years construction time, in September 2014 the 50 meter high double curvature Sarvsfossen dam was completed in Bykle in the Aust-Agder province as part of the 'Brokke Nord/Sør' project. It is the largest concrete dam built in Norway since the 1980s. The structure dams the river Otra that flows south through the valley of Setesdalen. The annual energy output of this hydropower-project is 69 GWh. Together with additional energy from existing stations downstream, a total of 175 GWh of renewable energy has been added to the regional grid. Along the 145 m long dam crest a concrete bridge is built, connecting the Bykle community center in the west to the rural district Stavnes in the east.

Design

Sarvsfossen (photo 1) is the largest concrete arch dam built in Norway since the Alta dam built in the 1980s. In this respect, it is a unique structure in Norwegian context. The dam thickness tapers from 6.5 m at the bottom to 2.3 m at an elevation of 43 m higher. The total concrete volume is approximately 19 000 m^3. The dam is not anchored to the bedrock by bolts, but relies on its selfweight and the double curvature shape to transfer loads into the abutments. In simple terms the water pushes the arch structure towards the abutments.

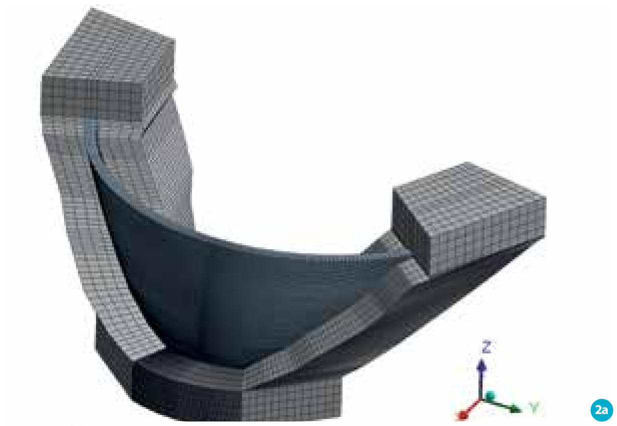
The general purpose finite element (FE) software Ansys, and a post-processor software performing non-linear concrete design of reinforced shell elements, MultiCon (Brekke et al. 1994, Multiconsult n.d.), was used for the design of Sarvsfossen dam. This has been the preferred tool for design of several large concrete structures in the offshore industry. This analysis package has been applied to model dam structures as they have many similar attributes to a typical gravity base concrete structure (GBS).

Erlend Eithun Aasheim,
Jan Lindemark,
Audun Hanssen Lundberg
Multiconsult ASA
Morten Engen
Multiconsult ASA / NTNU

1 The Sarvsfossen dam and bridge from downstream side
credits: Multiconsult
2 FE model with solid element mesh: (a) dam and bedrock (view upstream), (b) dam (view downstream)
3 Deformation of dam with full reservoir (deformation scale 400:1): (a) Vector sum of displacement [m], (b) vertical deformation [m]

The FE model of the dam, which consist of solid elements, is illustrated in figure 2. The model includes a bedrock volume which provides an elastic foundation for the dam. Between the concrete and bedrock elements, there are contact elements that can simulate sliding and/or uplift in the contact interface. The contact introduces non-linearities in the FE model (NLFEA). The contact region is illustrated in green in figure 2b.

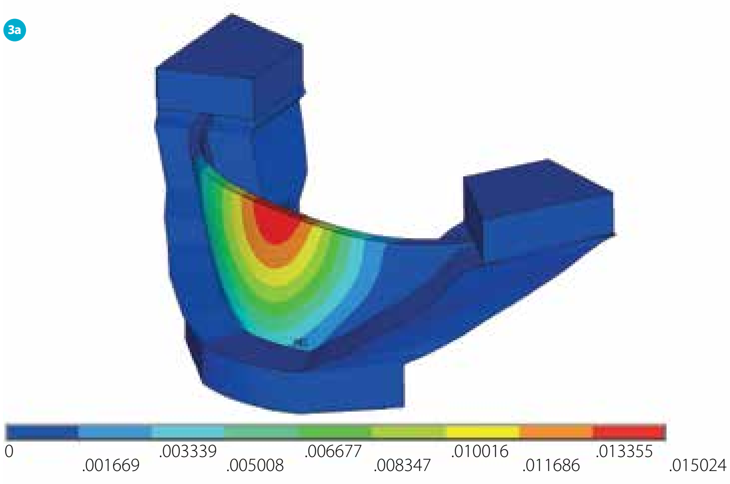
In addition to several water level configurations, ice, temperature and earthquake loads are simulated in the FE analysis (FEA). The rules for combination of loads, including applicable load factors, follow Norwegian Water Resources and Energy Directorate's guidelines for concrete dams (NVE 2005). This document states that the currently applicable Norwegian design code for concrete structures should be used, but with some special rules on e.g. load factors. The Norwegian Dam Regulations refer to general use of Eurocode 2: Design of concrete structures (Norwegian Standard 2008). This standard was initially used as a basis for design in agreement with the Norwegian Water Resources and Energy Directorate. However, due to its good track record for concrete structures with large shell thickness in a marine environment it was later decided to use the previous general concrete standard NS 3473 (Norwegian Standard 2003) as design framework. This standard has a good reputation from design of offshore concrete structures, including Concrete Gravity Base Structures (GBSes) in the North Sea. It is still used for this type of application. Through the process, it was also found that the shear tension capacity for concrete sections with large thickness is significantly higher in NS 3473 compared to Eurocode 2.

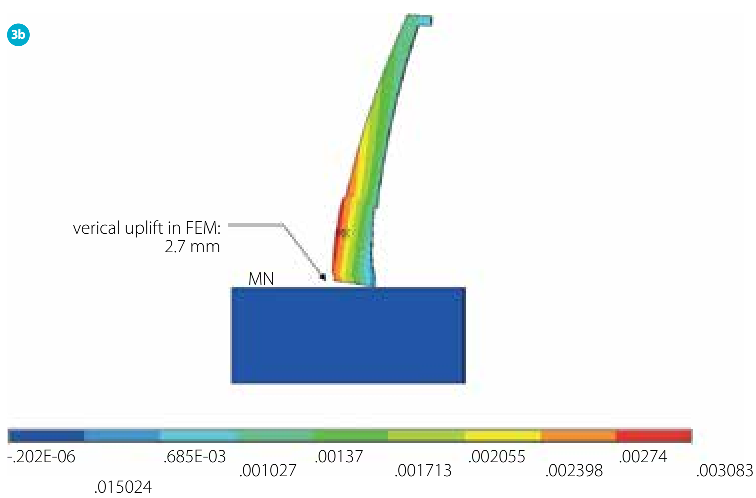

2a


2b

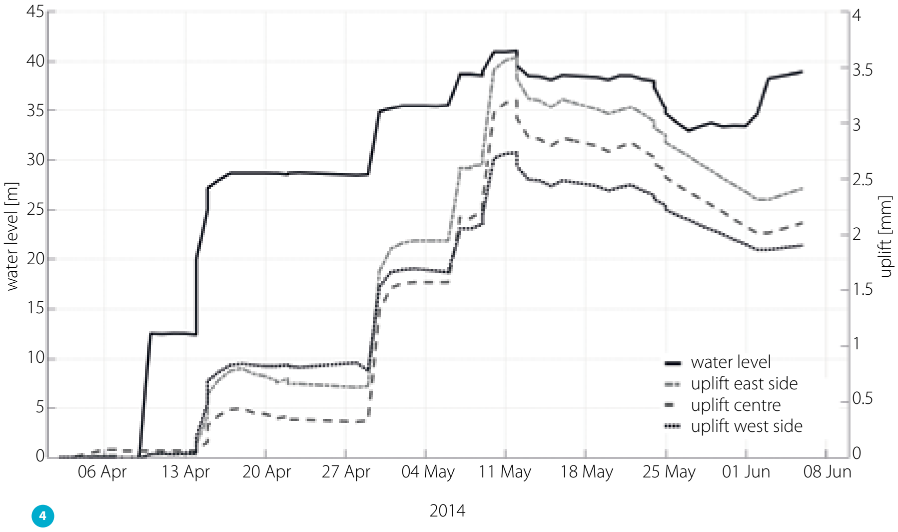
Simulation vs. on-site measurements

It was considered important to introduce contact interface elements in the FEA since no anchoring of the reinforced concrete to bedrock was part of the design of the dam. If the dam was modelled fixed to the bedrock, this would lead to very high reinforcement intensities in the vertical direction in the lower part of the dam on the water side due to clamping of the three-dimensional shell structure. This was found unfeasible since it would require a large amount of anchors/bolts and consequently it would increase the construction cost substantially. Modelling the dam

3a


3b


4 Relation between the water level and uplift of the upstream dam toe
5 Construction of the dam
credits: Multiconsult
6 Section of transition between foundation block and bottom of dam
7 Overview of dam site
credits: Otra Kraft



4

as fixed to bedrock would also underestimate lateral deformations. Lateral deformations of the dam were important in the design of the bridge structure.

As a consequence, it was decided to simulate the global response due to various load conditions in terms of non-linear contact analyses and rely on the dam's curvature vault shape to transfer forces to the abutments where uplift and sliding were permitted. The deformation of the structure for a typical load combination including water pressure is presented in figure 3. The plot to the left shows the vector sum, i.e. a combination of the three translational deformation components, in an isometric view, while the plot to the right shows the vertical deformation component in a section through the dam center. An uplift effect is observed at the bottom of the dam on the water side. Note that water pressure in the interface between bedrock and concrete was included with a linear distribution from the upstream side to the downstream side to simulate the effect of water intrusion.

In order to monitor the uplift during execution and operation of the dam, it was decided to install extensometers on three locations on the water side of the base of the dam before water filling was initiated. These extensometers measured separation between bedrock and the concrete structure above. Data from the extensometers are presented in figure 4. The measured uplift values are given together with the water level.

The measurements were conducted continuously through the first filling sequence of the dam after construction completion in April 2014. Good agreement with the predicted uplift in the FEA was observed. Both the water level for which uplift was initiated and the final uplift value for a full dam reservoir, were predicted with good accuracy. For the full dam reservoir a maximum vertical uplift of 3.5 mm was measured (fig. 4), while the maximum uplift was estimated to 2.7 mm in the FEA (fig. 3).

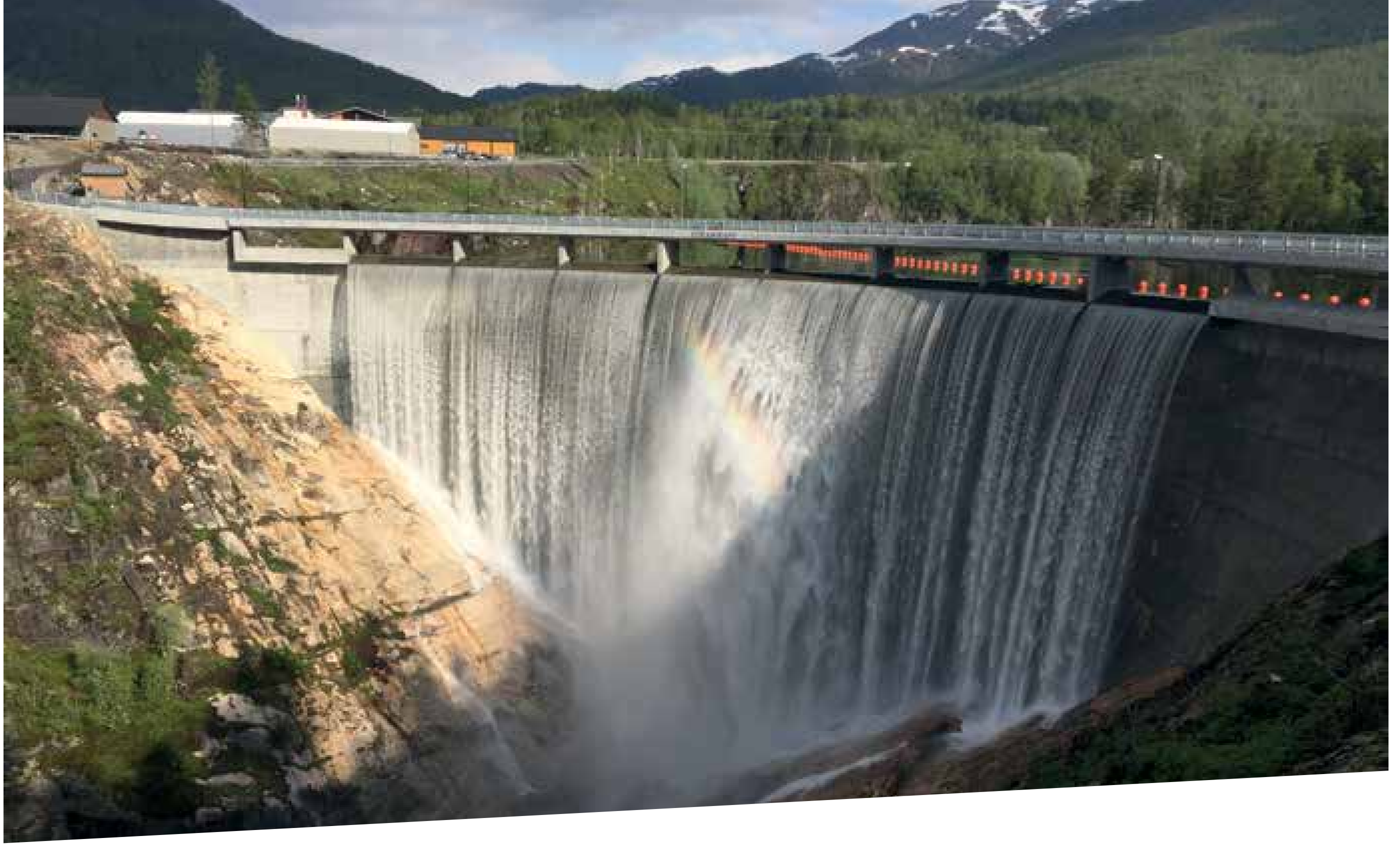
Construction

Initially the contractor investigated the use of sliding formwork for construction of the major part of the dam. The complex, double curvature shape combined with the asymmetrical layout proved this technique to be neither technically nor economically feasible. It was concluded to construct the dam in 5 m lifts divided horizontally in sections approximately 9 m wide resulting in 128 blocks to be cast. A Doka formwork system was used (photo 5). Towards the eastern abutment, the rock excavation on the upstream face was carried out as smooth excavation including stitch drilling. As a result, this face of the rock acted as the formwork for the twisting dam and the connection was improved by anchoring reinforcement in the rock.

Shear keys with upstream and downstream water stops were designed for both horizontal and vertical joints. A double set of injection hoses was installed, one for use before and one for use after impoundment. The reinforcement is continuous over both vertical and horizontal joints. There are wedges in the vertical edges of the separate casting phases to improve the interaction, especially on shear.

The predicted uplift of the upstream dam toe required careful considerations concerning water tightness of the foundation. The FE model showed the contact area to be relatively small in the bottom part of the dam while the contact area was larger in the sloping parts of the foundation. Due to poor quality of

5



7

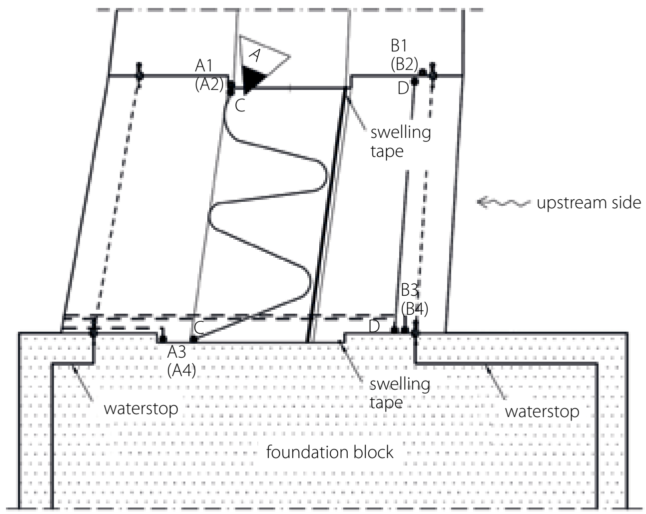
the bedrock, it was decided that a horizontal 30 m wide foundation block in the middle, underneath the dam required deeper excavation, i.e. 5-8 m. This block was cast with an expansion joint towards the dam (fig. 6). A shear key with an upstream and downstream water-stop and a swelling tape in the middle was installed. This measure should prevent leakage in the joint between the bedrock and the concrete caused by uplift of the dam toe. The completed dam structure is illustrated in photo 1 and 7.

Conclusion

In the Sarvsfossen project it has been favorable to establish a 3D finite element model with solid elements for concrete design purposes. This allows for estimations of the structural response without the need for (costly) conservative approximations. Using specialized design software for concrete shell structures, the necessary reinforcement amounts where efficiently calculated, satisfying relevant requirements in the ultimate and serviceability limit state. In particular, it was important to represent the complex geometric shape of the large concrete shell structure and its boundary conditions appropriately. After completion of the dam, this has been confirmed by on-site measurements that show satisfactory agreement with simulated data. It was concluded that one could solely rely on the geometric shape of the structure and its selfweight to transfer loads to the abutments. This facilitated a faster construction schedule and economic savings. ☒

Acknowledgements

Thanks to Otra Kraft for providing uplift data from extensometers and pictures of the dam.



6

REFERENCES

1 Lindemark, J., Aasheim, E. E., Mork, R. O., & Bjønnes, T. (2015). Sarvsfossen Dam – Design of a Norwegian Concrete Arch Dam, ICOLD 25th Congress / ICOLD 83rd Annual Meeting, Symposium Hydropower'15.

2 Brekke, D.-E., Åldstedt, E., & Grosch, H. (1994). Design of Offsore Concrete Structures Based on Postprocessing of Results from Finite Element Analysis (FEA): Methods, Limititations and Accuracy, Proc. Of 4th Intl. Offshore & Polar Engineering Conference.

3 Guidelines for concrete dams (2005). Oslo: Norwegian Water Resources and Energy Directorate (NVE). Multiconsult. (n.d.). MultiCon - Concrete shell design post-processing program (developed by Dan Evert Brekke).

4 Ansys. (n.d.). www.ansys.com.

5 Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings. NS-EN 1992-1-1:2004+NA:2008. Norwegian Standard (2008).

6 NS3473: Concrete structures - Design and detailing rules (6th edition). Norwegian Standard. (2003).


1

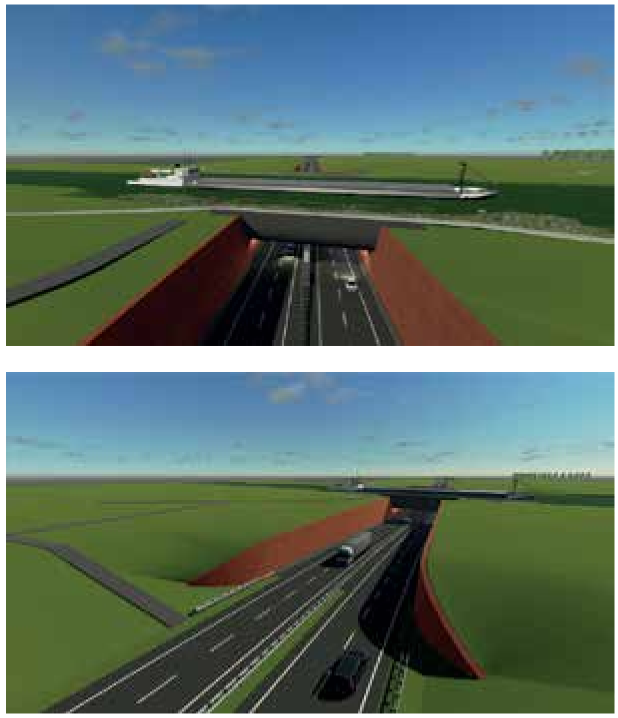
Hendrik Bulthuis Aqueduct Burgum

Across the Princess Margriet Canal in the province Friesland (The Netherlands) a new aqueduct is built. The Hendrik Bulthuis Aqueduct is part of the project De Centrale As, situated in the region of Dongeradeel, Dantumadiel, Tytsjerksteradiel and Smallingerland. De Centrale As will be the main access route to the northern part of the Netherlands and will improve the accessibility, safety, quality of life and the spatial and socio-economic structure of the region.

Sander van het Erve
Mobilis TBI

1 The Hendrik Bulthuis Aqueduct under construction
2 Artist's impressions of the finished aqueduct
3 Longitudinal section over closed part of aqueduct

The Hendrik Bulthuis Aqueduct crosses the Princess Margriet Canal at an angle of 76° (fig. 2). The aqueduct provides a highway with 2×2 lanes crossing underneath the Princess Margriet Canal (class Va vessels with maximum length 110 m). The aqueduct was realised by consortium Nije Daam (Mobilis TBI, Van Gelder and Friso Civiel) and opened for traffic on October 7 2016.
The structure of the Hendrik Bulthuis Aqueduct consists of open and closed parts (fig. 2). The closed part consists of sections 5 to 8 with marsh zones and an water cellar (fig. 3 and 4). The open, lower parts of the ramps consist of sections 1 to 4 and sections 9 to 13, and are built with steel sheet piles, concrete floors and pile foundations. The upper part of the open ramps are made using foil structures for a better integration in the area with green slopes.



2

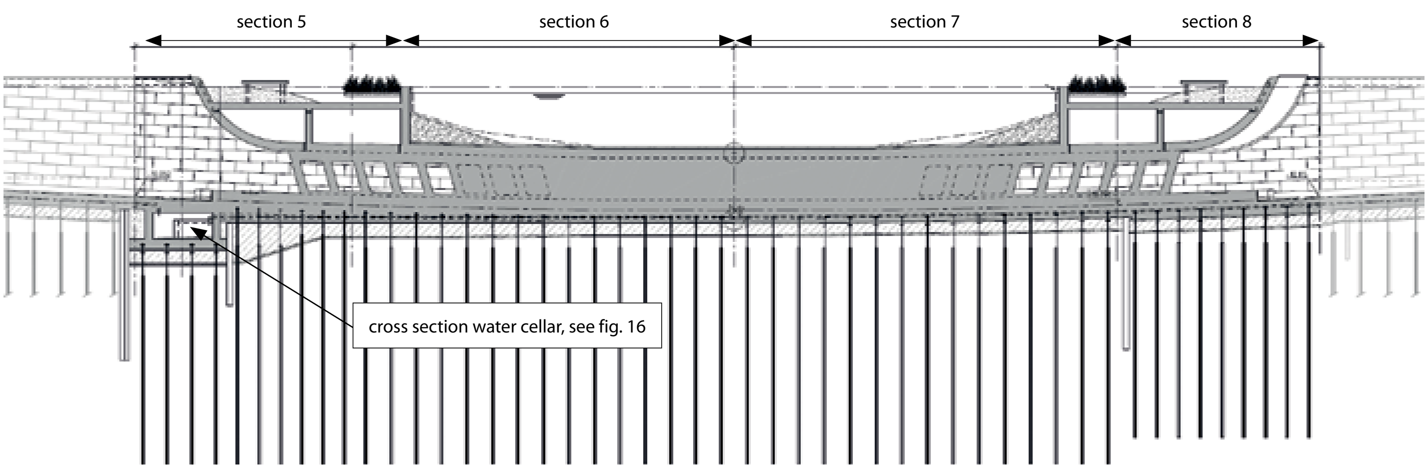
Modelling closed section for construction and operating phase

When the Princess Margriet Canal was opened over the new aqueduct, section 6 and 7 were finished. Sections 5 and 8 were built while vessels were already passing. Section 5 is monolithically connected to section 6 and section 8 is monolithically connected to section 7. An expansion joint (fig. 5) was installed between section 6 and 7, largely to prevent imposed deformations.

A distinction was made between different subsystems according to construction type:
a. Pile foundation
b. Underwater concrete floor C20/25
c. Final concrete construction C30/37

During the construction phase particular attention was given to the exceptional load acting on these sections due to possible collision by passing vessels. An underwater riprap slope at the front of the 800 mm vertical wall was installed before the waterway opened to reduce this load.

A structural 3D model was made for section 6 (fig. 6). Because of equal dimensions, the section 6 calculation in this phase is the same as the one for section 7. In this building phase, the foundations of section 6 and 7 were already laid on anchor piles; these piles, modelled as springs, can transfer both compressive and tensile loads to the substrate.

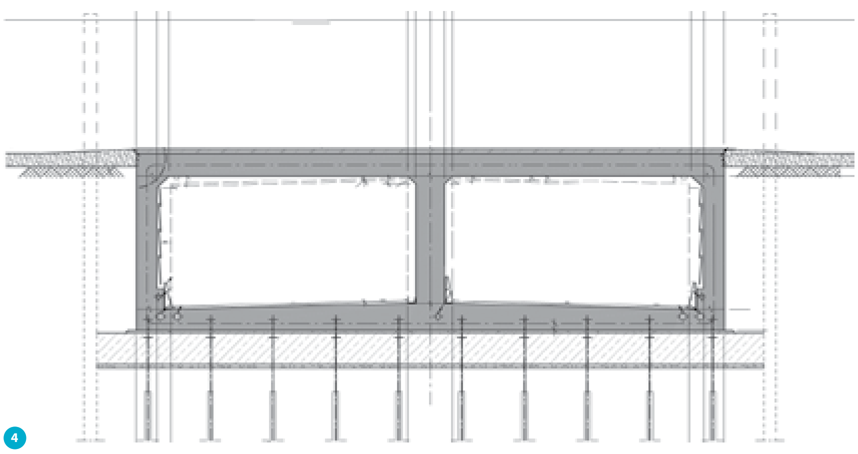


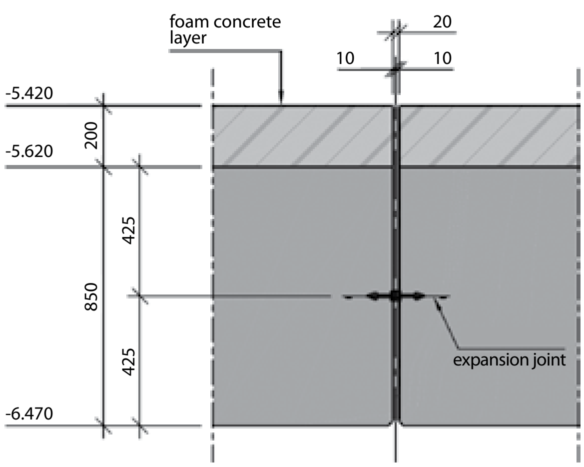
4 Cross section over the closed sections 6 and 7
5 Expansion joint
6 3D model section 6
7 3D model sections 5 and 6
8 3D model sections 7 and 8

In the operational phase calculations, section 5, 6, 7 and 8 are modelled in their entirety. The normative distribution of forces between construction and operational phase determines the dimensions and amount of reinforcement.

Attention was given to the curved ridge structures under the marsh zones at sections 5 and 8 (fig. 7, 8 and photo 9). Additionally, account had to be taken of the water cellar with access shaft at section 5 (fig. 3, 7 and 10). The dimensions of this were chosen in order to make sufficient water storage and pumping capacity available.

The marsh zones situated on top consist of a dry and wet zone and are necessary to maintain the ecology between the surroundings on either side of De Centrale As. A path at the marsh zone was created for a cycling and walking connection. Also attention was given to the specially shaped intermediate support between the driving lanes (fig. 10). For aesthetic reasons, visible ground-retaining walls were equipped with Corten Steel cladding (photo 11).

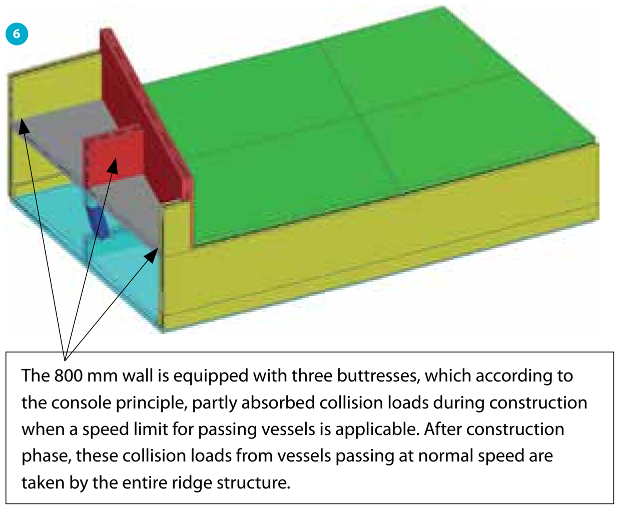

4

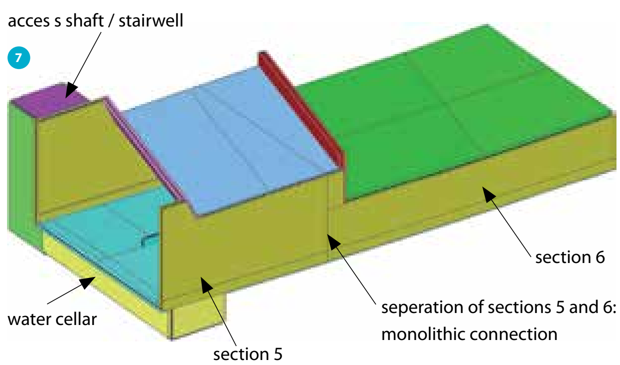


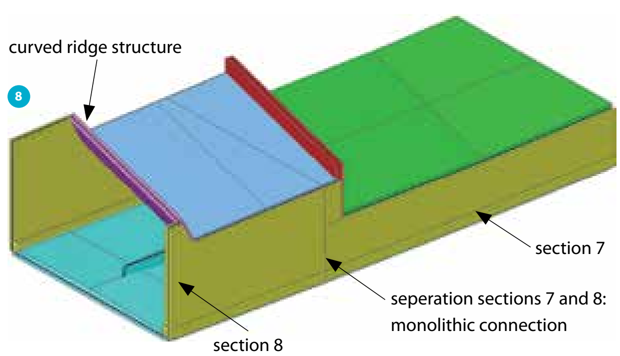
5

Loads

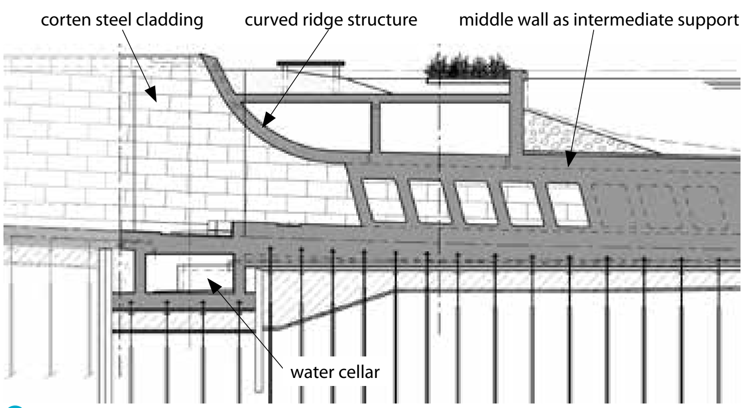
The loads used to design the structure comply with the standards and guidelines, and are introduced in the various models as described earlier. The SLS and ULS load combinations concern linear loads. A non-linear substrate bedding is not included. Only some typical loads are mentioned that were special for this project.

6


7


8


9



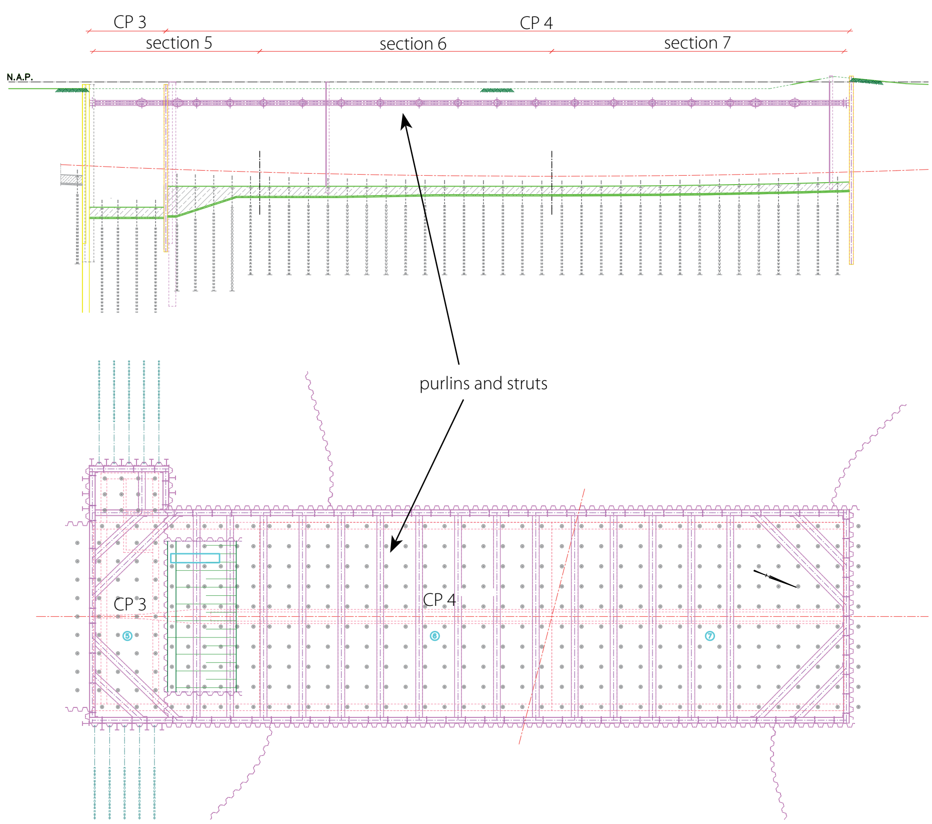
10

11

Ground pressure resulting in strut force during construction phase

During the engineering phase, the decision was taken to execute the compartments CP3 and CP4 simultaneously, being as a traditional cofferdam for building section 5 to 7 (fig. 12). The cofferdam is made with sheet piles, underwater concrete, purlins and struts. Purlins and struts are positioned at the top of the cofferdam and the underwater concrete floor is at the bottom.

To build concrete section 8, horizontal forces caused by difference in horizontal ground pressure in longitudinal direction had to be taken with struts. The horizontal forces of approximately 35 000 to 40 000 kN had to be taken up for the horizontal balance. This large horizontal force is transferred on the concrete deck of section 6 and section 7 (photo 13).



12

9 Building the curved ridge structures under the marsh
10 Cross-section marsh zone situated on top and middle wall, with columns, as intermediate support between the driving lanes
11 Entrance of the aqueduct with Corten Steel cladding
12 Is a longitudinal section of the entire CP3 and CP4 construction and layout

13

Loads during operational phase

Considered loads during operational phase were:

- *Calamity through fire*
- *Calamity through sinking vessel*
- *Calamity through falling anchor*

  For a falling anchor with a mass of 1600 kg a protective layer is required as absorption zone. This protection also prevents the deck from being affected by wear from vessels. For the shipping class within this project, class Va, a solution was chosen in which a 200 mm foam concrete layer is applied to the deck (fig. 5). A point load of 500 kN is calculated as load from a falling anchor.
- *Calamity through collisions*

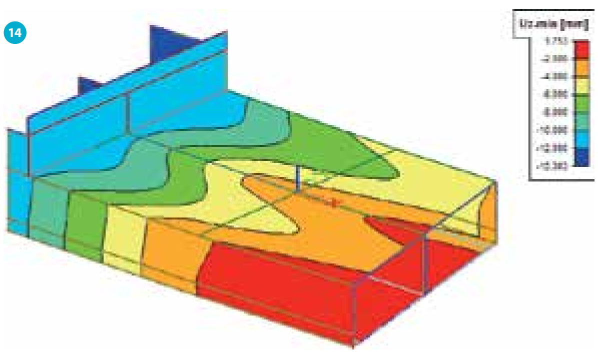
  Cruising speed of a vessel relative to the water (and vice versa) depends on:

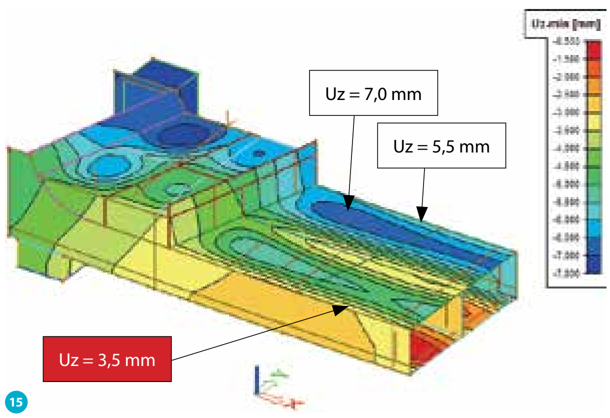
  - vessel type
  - waterway type (width and depth)
  - loading degree (loaded vessels are indicative)

  An underwater slope, installed to counteract colission from a the vessel, protects the concrete construction.
- *Calamity through ice load*
- *Calamity through vessel's dragged anchor*

  A vessel's dragged anchor can hook onto a construction. The anchor force maintained must correspond with the fracture strength of the chain/cable of the normative anchor, in this case 1540 kN. Protection against dragging anchors is provided by mounting a steel plate at the end of the water container floor.

14





15

13 Struts providing horizontal equilibrium during building section 8
14 Deformations section 6 during construction
15 Deformations sections 5 and 6 during operational phase
16 Distribution of forces in section 5
17 Sketch of the connection of wall to floor at section 8

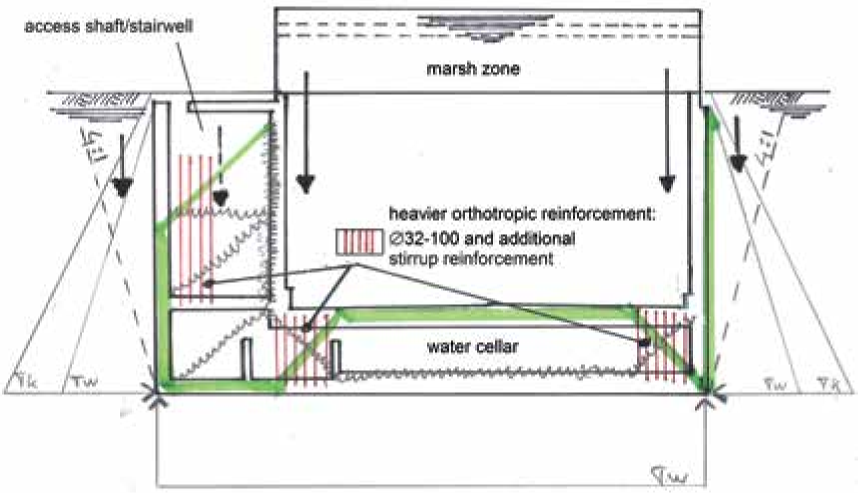
Deformation in construction and operational phases

Deformations during construction and operational phases are determined on the basis of SLS combinations (fig. 14 and 15).

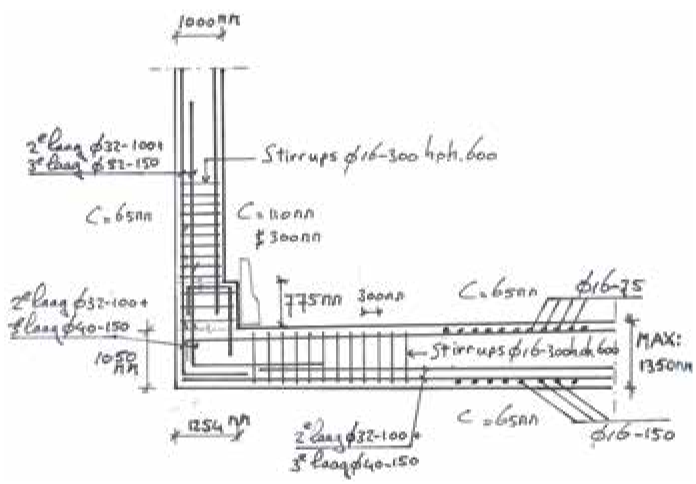
Maximum deformation in the middle of the deck is 7 mm. Maximum deformation of the wall is 5.5 mm. Relative distortion of the deck is therefore 1.5 mm at an un cracked section in the model. In reality the deck will become partly cracked so that the deformation of the deck hatch will increase by the ration of bending stiffness to - 4.68 × 1.5 mm = 7.0 mm. This is lower than the maximum allowed deformation of 10.85 mm, prescribed as 1/1000 of the span between the outer wall and intermediate wall.
It can be seen from the model that when load acts eccentrically the western outer wall deflects about 5.5 mm, and the eastern wall 3.5 mm. Only a very small misalignment occurs with an order of magnitude of 0.1 per mil in transverse direction.
In the longitudinal direction, the misalignment is even smaller.

Force distribution and reinforcement

Distribution of forces in section 5, including water cellar, access shaft/stairwell and marsh zone, is sketched (fig. 16). The location of this cross section is given in figure 3. Large downward vertical loads are acting on the outer walls. These loads are mainly caused by a ground wedge, dead weight of the walls and slabs and piled marsh zone, grain and water pressure on the piled marsh zone and grain and water pressure at the waterway location. The pile foundations under the flooring provide reactions for the vertical balance.
A strut and tie model is drawn in figure 16, the green lines and pencil lines respectively represent the tensile and the compressive forces. Total interaction of forces must be externally balanced. So vertical loads are absorbed by vertical pile reactions. Horizontal ground pressures will also be balanced together. There must also be internal balance. The finite element package shows how the normal and shearing stresses run in various parts of the construction. This serves as a tool for determining the strut and tie model.
If an orthotropic reinforcement mesh is used, tensile and compressive forces can be converted to vertical and horizontal directions. Consequently, llarger quantities of reinforcement are required than when reinforcement is applied in the direction of the tensile forces. This is logical because the reinforcement is no longer placed perpendicular to the crack direction and is therefore less effective. After consulting the executing body, it was concluded that orthotropic reinforcement should be used in certain areas. Therefore heavier reinforcement was installed in the areas with red vertical shading.



16

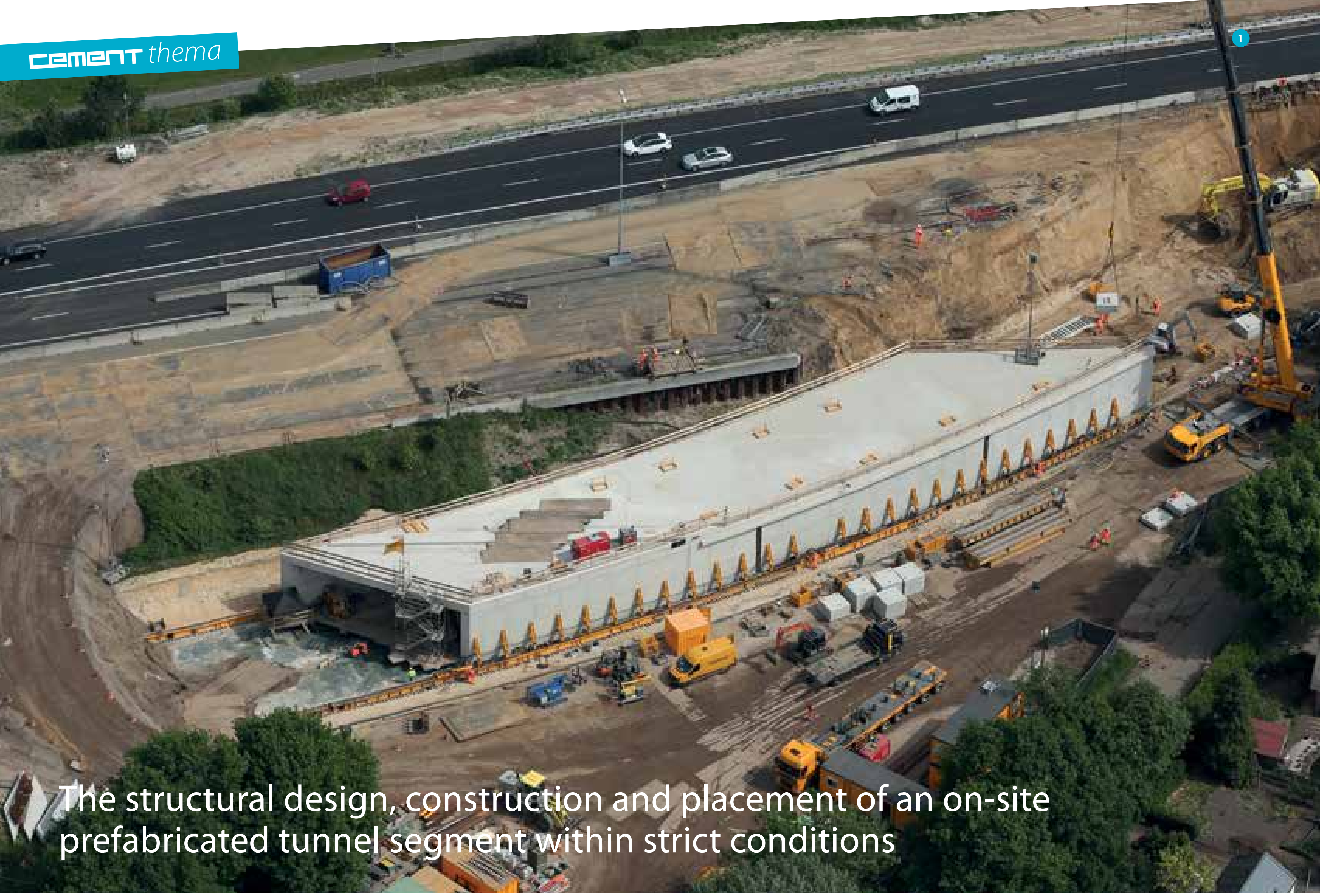


17

At section 8, one wall is both vertically a considerable overhang as well as horizontally. There is also no access shaft with water cellar (fig. 7 and 8). In relation to the maximum reinforcement percentage, floor and wall have acquired greater thickness. Floor thickness at section 8 increased from 800 to 1050 mm at the outside and from 1100 to 1350 mm in the middle. Wall thickness increased from 800 to 1000 mm (fig. 17).

Experience within the project

The very important challenge in the project was to achieve good balance between design, planning and construction team. The most challenging part was the curved ridge structure due to the phasing and the various loads that had to be taken into account. ☒



The structural design, construction and placement of an on-site prefabricated tunnel segment within strict conditions

Tunnel underneath highway A12 near Ede

In the Dutch highway A12, near Ede, a new tunnel structure crossing the existing highway was designed and built. This structure is commonly referred to as ODG A12 and provides the underpass junction for the Parklaan and hence an improved accessibility of the city of Ede. The structure consists of several tunnel segments. The largest segment, which provides the actual crossing, was prefabricated on-site and then was placed at its final destination in just one weekend.

ing. Stefan Schoenmakers MSEng,
ing. Erik de Rooij PMSE
Wagemaker

1 View on the monolithic, on-site prefabricated tunnel segment from the air
credits photos: Rijkswaterstaat
2 The tunnel segment during the sliding operation
3 Typical cross section

ODG A12 is part of a metamorphosis of the highway section Veenendaal - Ede - Grijsoord. The highway was upgraded with additional fast lanes as well as several new structures such as bridges and viaducts, allowing an appropriate connection with the adjacent junctions (Grijsoord and Waterberg). Primary objectives for this upgrade were an increased traffic flow and improved traffic safety. In order to realize this upgrade existing structures needed to be modified (e.g. widening of bridges) and sound barriers were placed. Also the use of special asphalt mixtures exhibiting low sound emission properties contributed to a sustainable design solution.

The design of the new structure ODG A12 had to meet a variety of conditions, amongst others architectural requirements and limitation of traffic hindrance during construction and nuisance for the inhabitants and surrounding environment. In this respect it was chosen to apply a sliding operation of the full monolithic structure in order to limit hinder and maintain an operational highway as long as possible. Due to its geometry (15 m wide, 80 m long and approximately 6 m in height) this was a challenge.

The structural calculation models incorporate all subsequent stages of the building process and associated loads, as well as different soil parameters. The section dimensions were optimized to limit the dead weight to approximately 3800 tons. Within the models, soil pressures underneath the structures during the building sequence were carefully monitored in order to limit settlements.

Structural design and design considerations

The main design aspects that had to be taken into account, are:
- the tunnel crosses the highway at an angle of 33°. This means that the tunnel segment length is large compared to the width of the highway (width of the highway is approx. 36 m, length of the tunnel segment is approx. 80 m);
- the road design for the tunnel is horizontally curved. In order to optimize driving comfort as well as material usage, it was chosen to adopt the same curvature for the complete length of the tunnel. This results in a constant cross section of the tunnel segments;
- soil parameters showed that a shallow foundation was possible;
- essential within this project was to limit traffic hindrance to an absolute minimum. This resulted in an additional requirement in terms of construction technology and intended building phases;
- the aesthetical appearance was mandatory and was derived from the aesthetical vision ‘Regenboogroute A12’, setting an extensive list of requirements. Masonry appearance of the structure was mandatory. At the inner side a finishing consisting of tiles and light armatures was required.

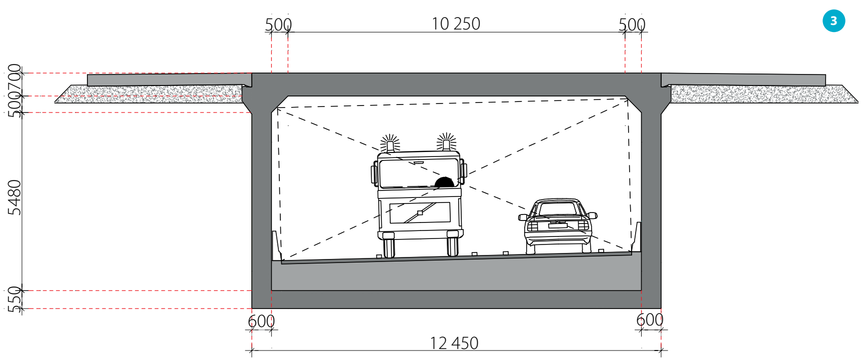
2

The entire structure is made of reinforced concrete (without prestressing). The strength class of the concrete used was C45/55. Figure 3 shows a typical cross-section.

The structure was modelled in Autodesk Revit. In close cooperation with the architects, solutions were designed to satisfy the aesthetical vision. Visuals were created, based on the Revit model, which have been used in several stages of the project, such as requesting the building permit and during meetings with inhabitants.

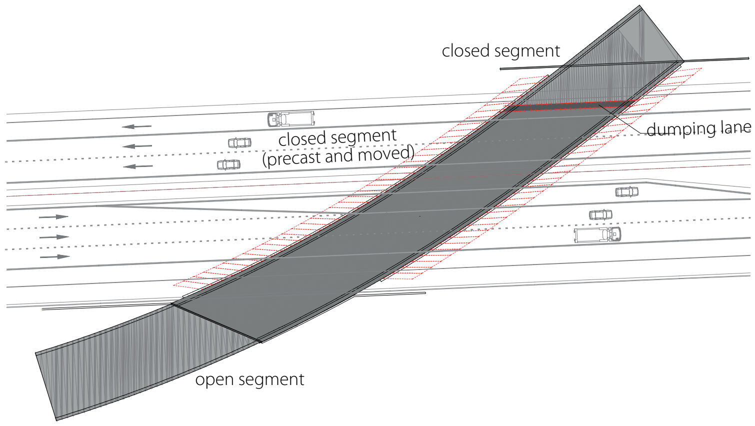
Figure 4 and photo 5 show an architects impression and a photo taken from approximately the same viewpoint.

3

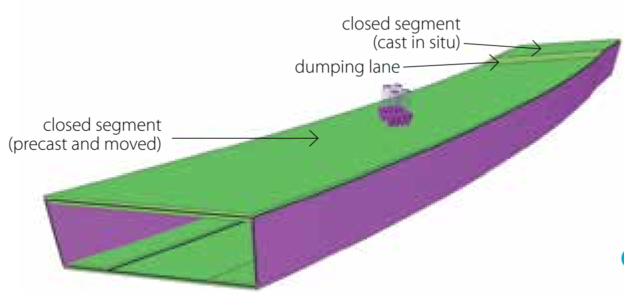


4

5



6



7

4 Architectural impression
5 Built situation (under construction)
6 Plan view of the complete structure
7 3D-model of combined tunnel loaded by a tandem axle load
8 Distribution of principle moment m[1] [kNm/m[1]]; (a) interior; (b) exterior

Construction phases and relation with structural design

In order to minimize traffic hindrance it was chosen to adopt the following building schedule:
- Building of one tunnel segment at the final position;
- Prefabrication of the major part of the tunnel on-site at a temporarily position;
- Sliding of the prefabricate segment towards its final position;
- Building open tunnel segment.

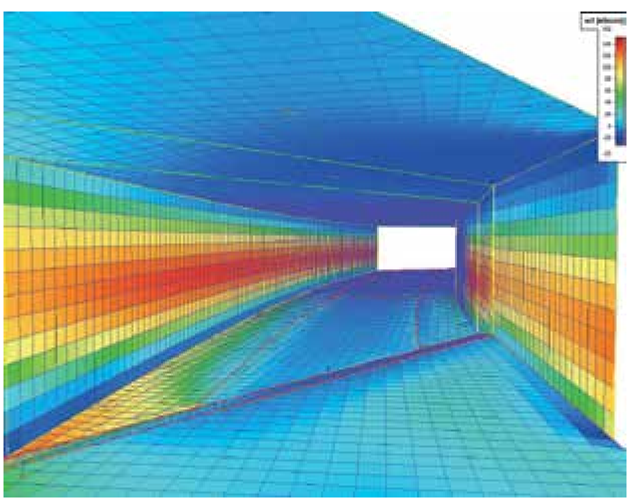
In figure 6, a plan view of the complete structure is given.

In adopting this building sequence, the highway A12 could remain operational during the entire construction time (photo 1) with the exception of just one weekend. During this weekend tons of earth have been moved to create an opening in the soil body providing the space needed to position the tunnel. Also, at the crossing location, the existing road has been demolished and a new road has been built within this time span.

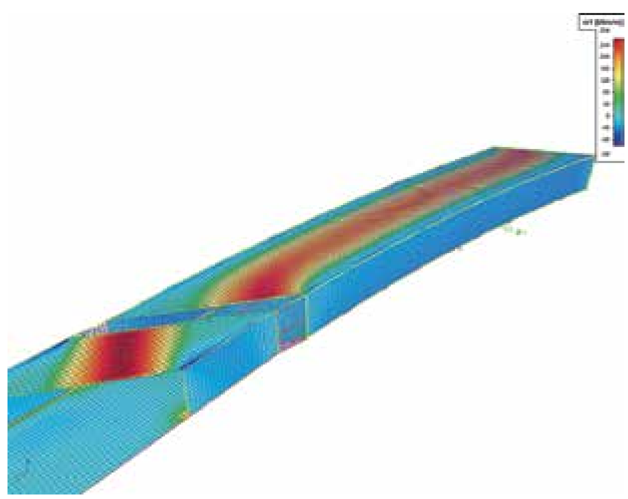
In the preliminary stages other design solutions, such as the use of (prestressed) concrete slabs combined with temporarily structures (e.g. sheet piles), have been considered. However, all alternatives appeared to be less preferable than the monolithic structure, as now built.

It was decided to apply a sliding operation to move the tunnel segment to its final position. Performing a sliding operation involves the use of hydraulic jacks to lift the structure. To allow for the local jack loads, dowels were cast in with associated local reinforcement. Due to the horizontal curvature, the sliding operation was non-conventional. Since sliding was the preferred displacement technique, the total amount of dead weight should be limited. In the design of the concrete dimensions (e.g. wall

8a



8b



9

10

thicknesses, slab depths) this was taken into consideration at all times. It was chosen to adopt a phased casting sequence for the middle part of the floor slab since during the sliding operation the structure was loaded only by the dead weight. After positioning the tunnel segment, the final thickness of the floor slab was created by the second cast. The connection was realized by reinforcement at the interface. Soil compaction allows the ground underneath the floor to be fully mobilized.

Photo 1, 2, 9 and 10 show impressions of the sliding operation.

Calculation method

The structure was modelled in 3D SCIA Engineer. In total three models have been used:

- a shell model of the first tunnel segment (built at its final position);
- a shell model of the monolithic segment to be slided to its final position;
- combined model of the both closed tunnel segments at the final position.

11

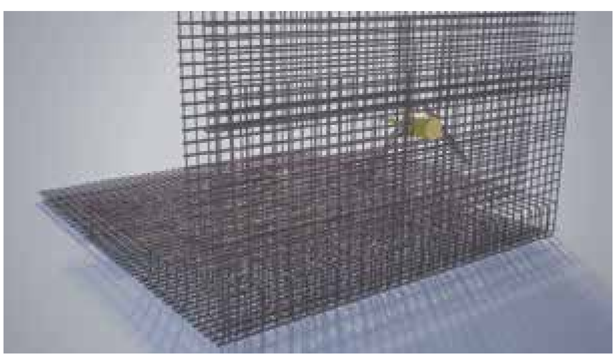


Figure 7 shows an impression of the shell model consisting of both tunnel segments. In between both segments a dumping lane of 2.5 m in width is used.

The building sequence is explicitly taken into account in the models. Based on the results in governing cross sections, amounts of necessary reinforcements are calculated taking into account both ULS and SLS. Within the models, occurring soil pressures underneath the structures during the building sequence were carefully monitored in order to limit settlements. Soil pressures of approximately 250 kN/m^2 were considered allowable.

Figure 8a and 8b show the distribution of the first principle moment in SLS-load combinations. The dumping lane allows for initial settlements of both segments without involving undesirable forces.

A special aspect within this project was the local load on the tunnel walls introduced due to the presence of the hydraulic jacks necessary for sliding. The sliding operation consist of several stages. First, the tunnel segment was lifted initially to obtain clearance between the bottom side of the floor and soil. These jacks are mounted on the steel sliding frame. All dead weight is transferred at the location of the dowels. Figure 11 indicates an impression of the 3D-Allplan model of the reinforcement at this dowel location. Additional reinforcement to prevent breakout is indicated as well. Since lots of jacks were present, the load per jack was relatively low and hence, only a few bars were necessary.

Project specifications / resumé

Heijmans acted as contractor to build this project but is also responsible for the maintenance over a period of 16 years. Wagemaker, in commission of Heijmans, was the structural engineer. The project was started in 2014 with the preliminary stages and has been completed in October 2016. ☒

9 View on the sliding tunnel segment from the side
10 Detail of jacks and sliding girders
11 3D reinforcement model (only one dowel is drawn)

Complex railway bypass

Combined bypass railway and tangent road near the station of Mechelen (Belgium)

In extending the existing railway infrastructure from Brussels to Antwerp, a new double track railway is foreseen as a bypass along the railway station in the city of Mechelen. In combination with this, the station will be extended with an underground parking building. This car park will be reached by a new tangent road connection between the southern and northern part of the city, which should improve and the accessibility of the railway station and reduce the traffic in the center of Mechelen.

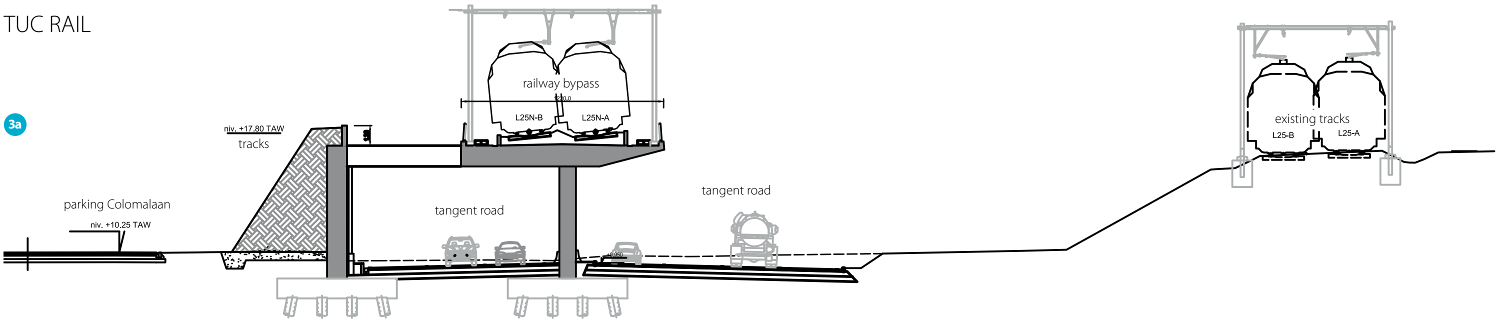
The road connection is situated parallel and underneath the new railway bypass for half of the project (fig. 2). This results in a combination of heavy concrete road tunnels and railway viaduct infrastructures in an urban environment. In December 2017 the project will be finished.

The tangent road tunnel

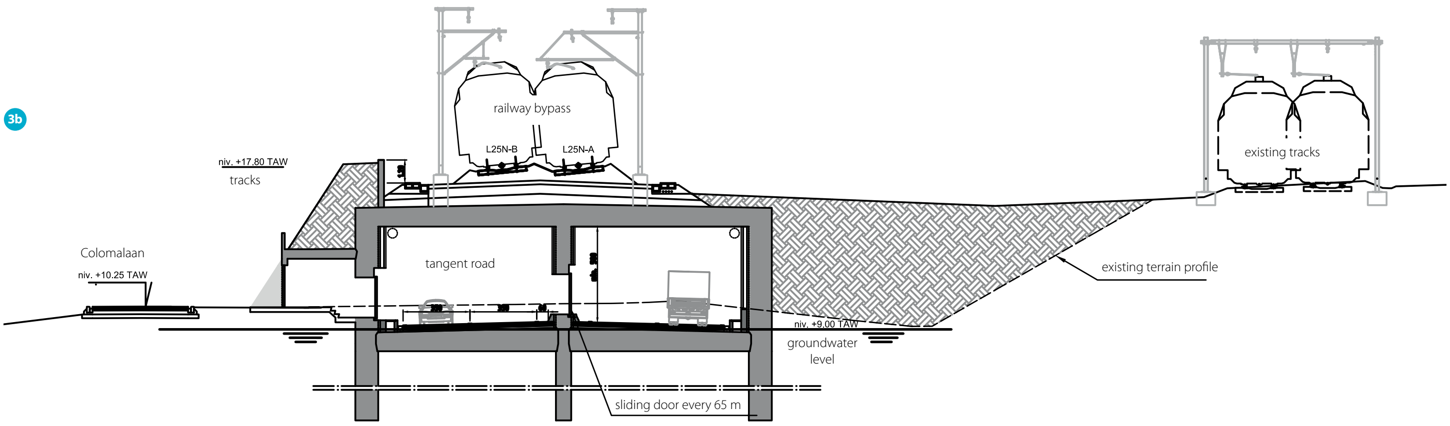
The southern tunnel entrance in the combined road and railway project consists of different cross sections (fig. 3). The end of the concrete viaduct of the bypass railway infrastructure in elevation is the start of the road tunnel for a double 2-lane road infrastructure. An optimization is achieved by bundling of

Bart De Pauw
TUC RAIL

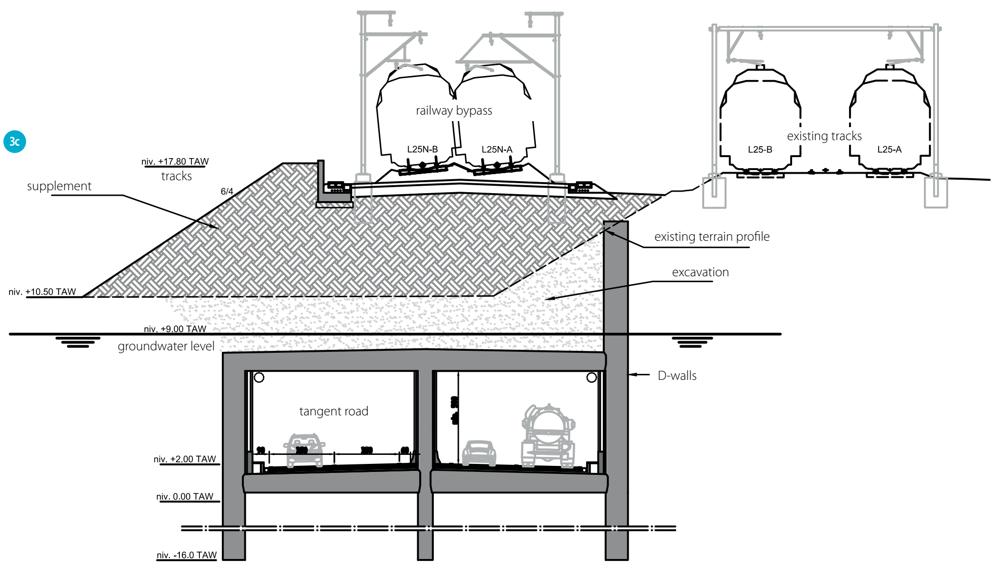
3a



3b



3c

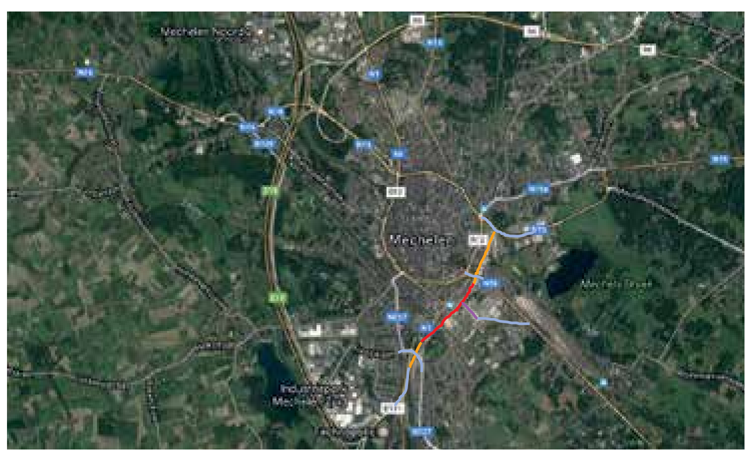


1 View on the finished rail viaduct over the Leuvense Steenweg
2 Plan view of the road connection project (yellow: full road connection, red: tunnel part, purple: additional entrance to tunnel and parking building)
3 Cross sections of the tangent road tunnel entrance in the southern part of the project

both infrastructures and limiting the required space. The tunnel structure consists of diaphragm walls of which the outer wall is a temporary retaining wall to the existing railway infrastructure (fig. 3c). After construction of the diaphragm walls, the tunnel deck plate will be constructed after which the tunnel can be excavated and finally the tunnel floor can be executed. In the deepest part, the tunnel structure is fully situated underneath the ground water level.

One particular structure near the tunnel entrance is the railway bridge over the Jubellaan. The railway bridge is a continuous bridge over three spans (28 m – 44 m – 28 m) with the superstructure weighing 2521 tons. The cross section consist of a

2

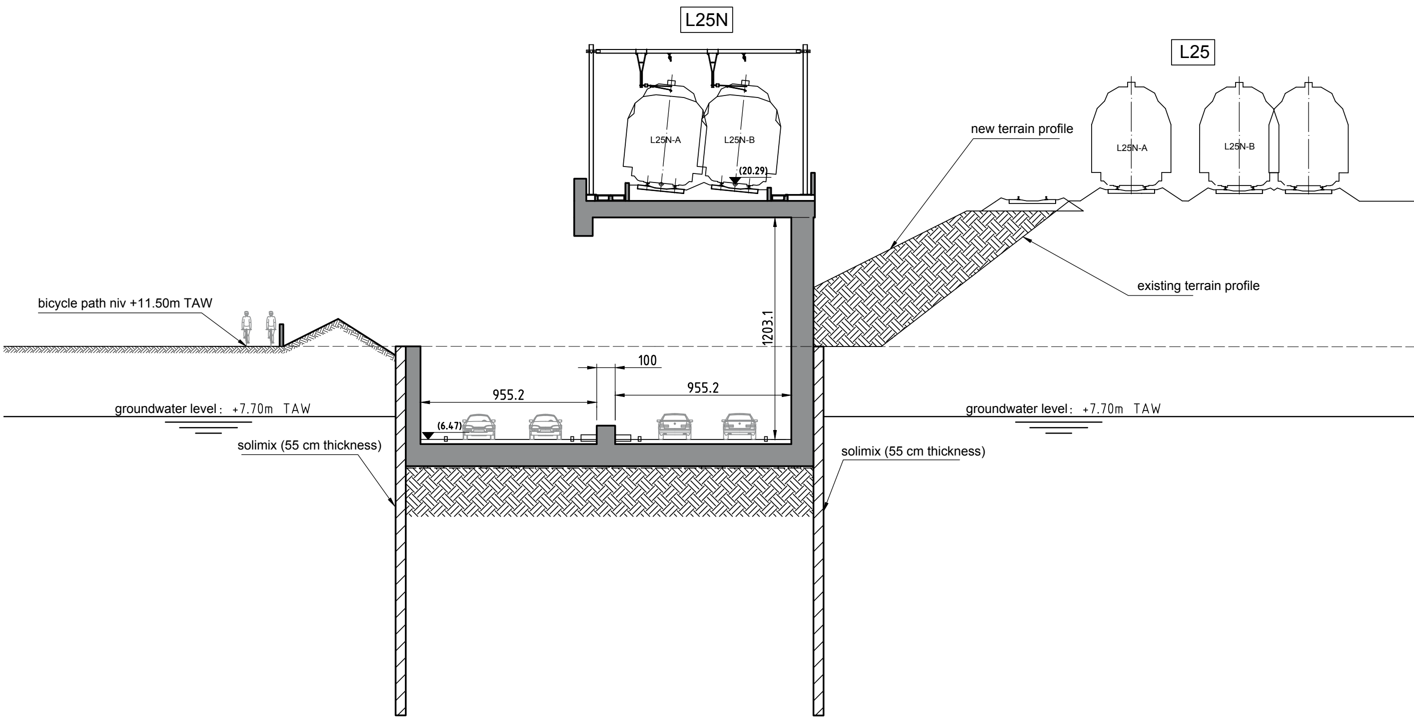



sarens
sarens

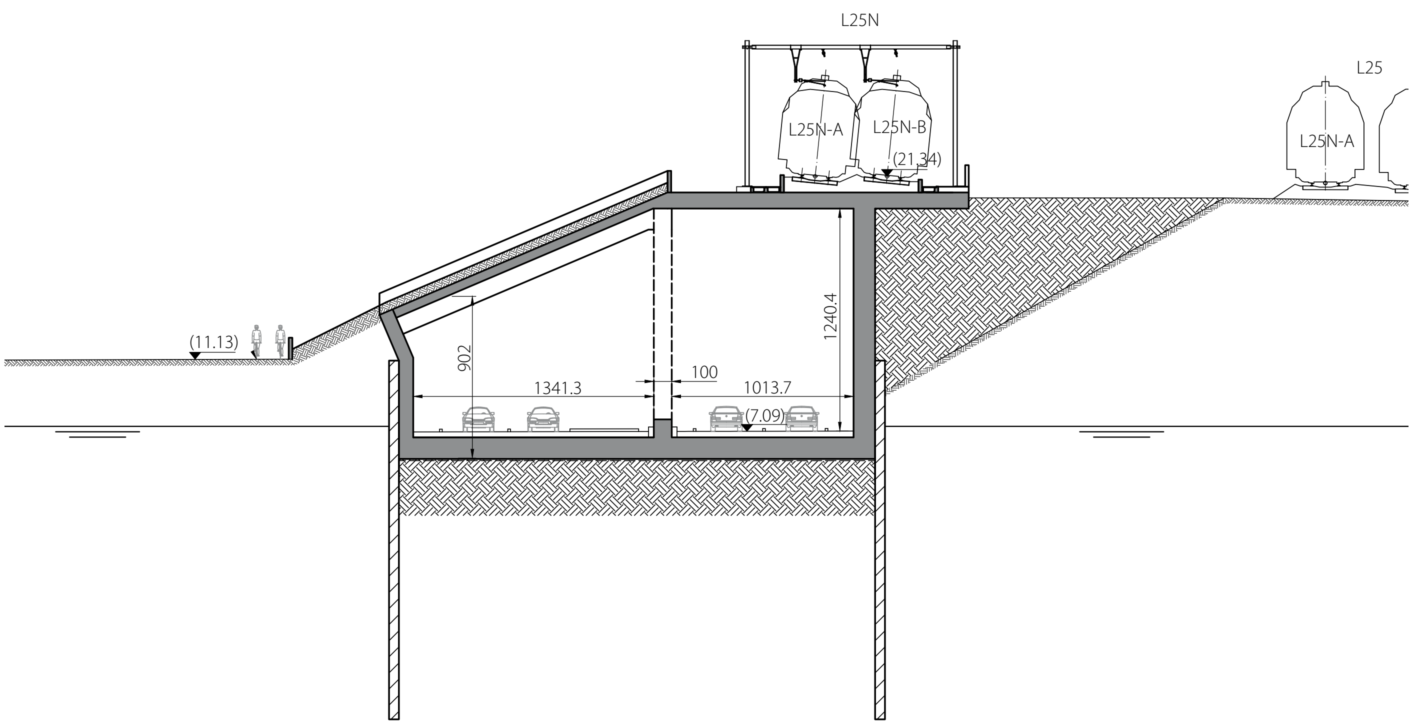
4

5

6a


L25N
L25
L25N-A
L25N-B
(20.29)
L25N-A
L25N-B
new terrain profile
existing terrain profile
bicycle path niv +11.50m TAW
1203.1
100
955.2
955.2
groundwater level: +7.70m TAW
groundwater level: +7.70m TAW
(6.47)
solimix (55 cm thickness)
solimix (55 cm thickness)

6b


L25N
L25N-A
L25N-B
(21.34)
L25
L25N-A
(11.13)
902
1341.3
100
1013.7
(7.09)
1240.4

7

8

hollow concrete cross section with post-tensioning cables in the inner ribs and external post-tensioning cables in the inner hollow parts. The cross section has variable height. Special friction tests have been performed on a test beam on site since no ATAG certification was available for the type of bonded post tensioning cables that were used. The bridge was erected some 250 m from its final position. During a weekend closure of the Jubellaan, the bridge was transported to its final position by means of a SPMT (Self Propelled Modular Transporter). Afterwards, jacking was performed in lifting and redistributing the total weight of the bridge over the pier structure and abutments (photo 4).

In the northern tunnel part of the tangent road project, road and railway alignment is split again smoothly (photo 5 and fig. 6). For architectural reasons, the roof of the tunnel opens partly and carries a green roof. When additional lanes of the tangent road structure appear towards the crossing with the existing road, the Leuvense Steenweg, the roof structure is made skew (fig. 6b). Soil mix retaining walls are made in limiting the excavation since the ground is contaminated with a historic remnant. The base of the tunnel entrance structure is situated underneath groundwater level. The rest of the tunnel structure consists of reinforced concrete. The structure is partly a retaining structure for the existing railway infrastructure as well as a viaduct structure for the bypass railway infrastructure.

Another structure near the tunnel entrance is the railway bridge over the Leuvense Steenweg. The bridge section is an extension of only the railway section of the entrance (fig. 3a). The railway bridge is a continuous bridge over three spans (31.5 m – 33.5 m – 31.5 m) with the superstructure weighing 2750 tons. The cross section consists of a U-shaped section with outer main beams and a hollow concrete deck. Both main girders and the deck are reinforced with unbounded post-tensioning cables (photo 8). In contrast to the other railway viaduct over the Jubellaan, this bridge is cast on site, immediately at its final position (photo 1).

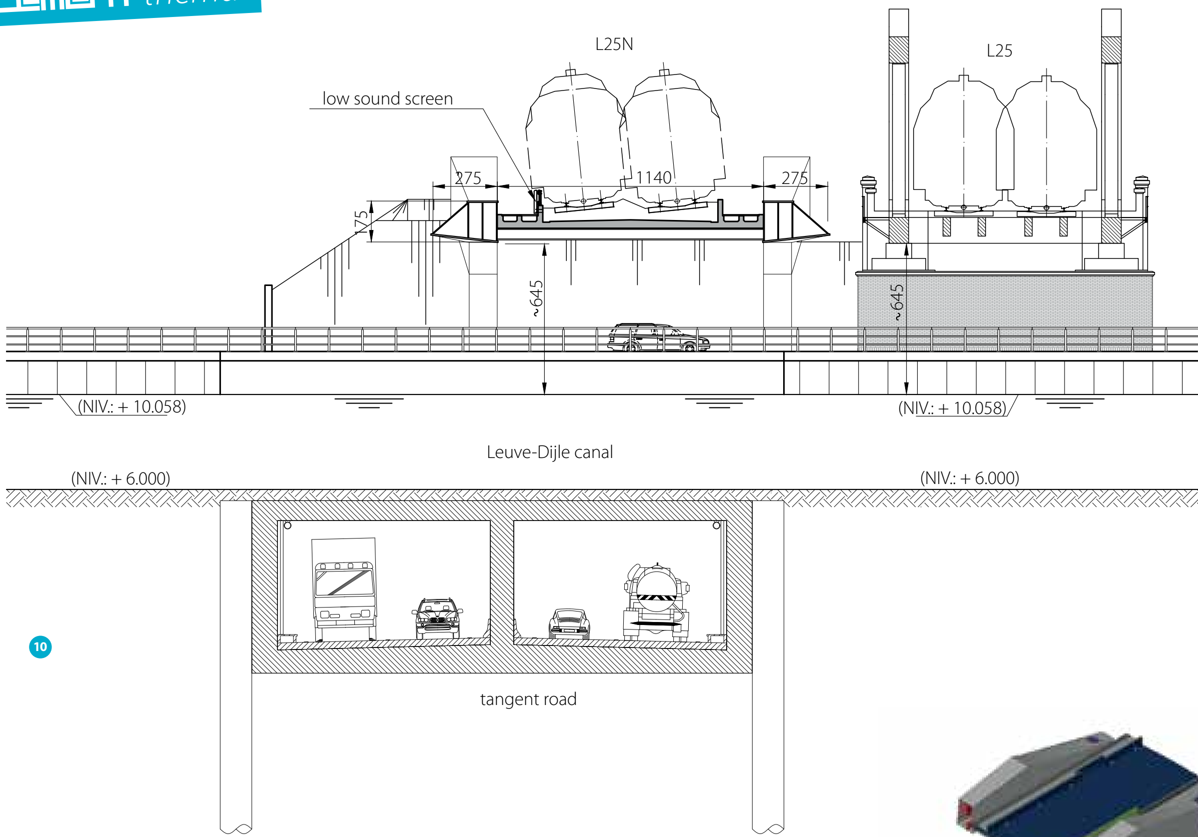
Skew post-tensioned concrete railway bridges

At both sides of the project to arrive to the tunnel entrances, the tangent road needs to cross the existing railway consisting of four tracks approaching the station under a skew angle of 45°. This is achieved by four similar skew post-tensioned concrete continuous slab railway bridges (photo 9a and 9b). The cross sections of the solid slab bridges with a width of 14 m are slightly curved and have side girders. The continuous slabs

4 View on the railway viaduct crossing the Jubellaan near the tunnel entrance
5 View on the northern tunnel entrance during construction
6 Cross sections of the tangent road tunnel entrance in the northern part of the project
7 View on the tangent road tunnel entrance in the northern part of the project
8 View on the rail viaduct over the Leuvense Steenweg under construction at the northern tunnel entrance
9 View and cross section of the skew post-tensioned concrete railway slab bridges

9a

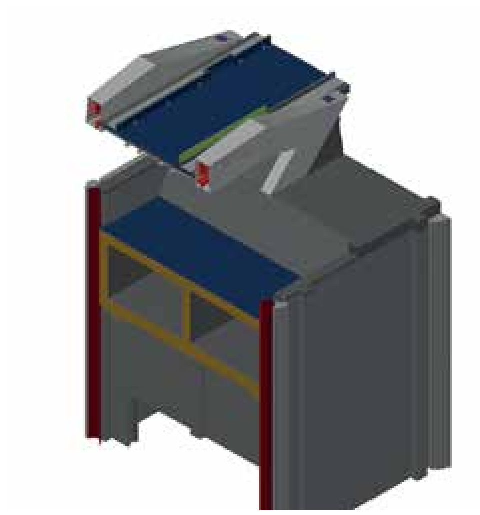
9b



10

10 Cross section at mid span containing existing and new railway infrastructure and a new road tunnel
11 Artist's impression of the final design with steel portal structure
12 Front view and cross section of the total project with tunnel and bridge
13, 14 Fixations of the steel structure with post-tensioning bars

have a double span of about 25 m and a total length of 50 m. The influence of the skew angle is analyzed and the post-tensioning in the edge beams has been optimized. The total railway line breaks were limited to four weekends per bridge. In the first three weekends the preparatory works are carried out in making the foundation piles and the sheet piling of the abutments and piers and in installing temporary steel bridge decks under which the abutments can be built. In the mean-time, the superstructure is constructed and prestressed on a building scaffold 200 m from its final position. During the last line break, the bridge has to be transported to its final position.



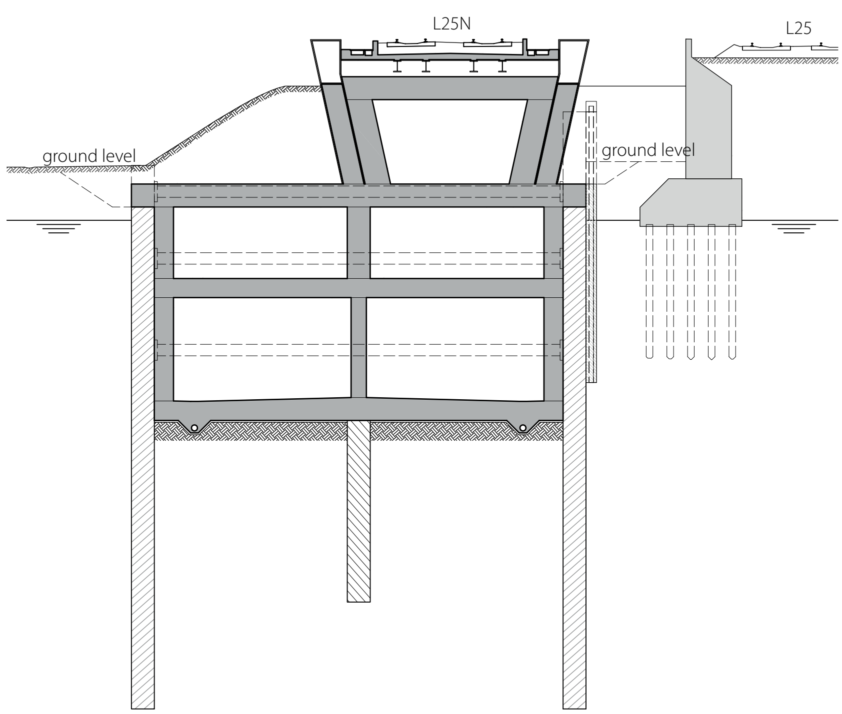
12a

The steel bridge crossing the 'Leuvense Vaart'

A particular spot in the tunnel alignment is the crossing with the 'Leuvense Vaart'. This crossing with the canal of both road and railway infrastructure is located near the existing historical steel Vierendeel bridges. The crossing of the bypass railway happens above the canal, while the tangent road infrastructure crosses the canal underneath being the deepest part of the tunnel (fig. 10). An important design criterion is keeping the full view of the historic Vierendeel bridges. A final design was developed in using an integral steel portal bridge.

The bridge consists of two lateral main girders having variable rectangular sections and is designed as an integral structure without bearings. The span is about 65 m, which is 5 m more than the arch Vierendeel bridges and has a reversed curvature near to the arch springs of the Vierendeel bridges (photo 11). The inverse curvature of the new bridge only makes sense if the abutments are fully integrated in the supporting tunnel structure underneath.

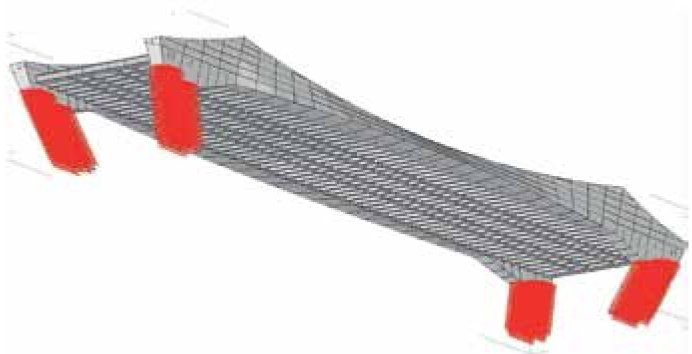
12b



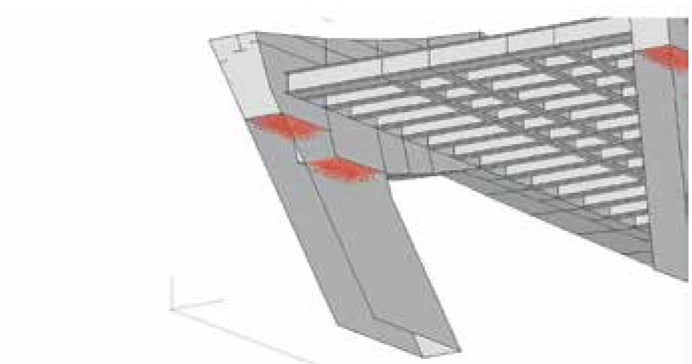
11

The foundations of the new integral railway bridge consist of reinforced concrete slabs spreading the reaction forces to several diaphragm walls which are part of the tunnel structure. This is done by a complex concrete structure with internal concrete shells transferring the forces from the main box girders to the vertical walls of the tunnel (the two outer and the inner wall).

The both outer diaphragm walls are not only temporary retaining walls for construction of the tunnel, but also the final vertical deep foundations of the total structure. A particular concern in the design of the bridge is the abutment stiffness providing the clamping of the steel superstructure in the concrete foundation. Therefore, a parametric study by finite element modeling has been performed (fig. 12 [2]). The steel construction is fixed by post-tensioning bars diameter 47 mm and 75 mm as well as by steel dowels on the outer steel webs which are fading into the concrete abutments. The fixations with post-tensioning bars are simulated as fixed points at the level of horizontal steel plates (fig. 13). Photo 14 shows the preparation of the bars during execution.



13a



13b

14

Conclusion

In completing the high speed railway network in Belgium, a new railway bypass between Brussels and Antwerp will be constructed in the city of Mechelen. This includes the extension of the existing railway station of Mechelen. The optimal bundling of both railway and tangent road makes it a complex situation. As a railway infrastructure, these structures are exceptional in the use of post tensioning concrete as well as in its design and shape. ☒

REFERENCES

1 Pauw, B. de, Van Bogaert, Ph. (2011). The design of an integral steel railway bridge in Mechelen (Belgium). Proc. Eurosteel 2011, Budapest, Hungary.

2 Pauw, B. de, (2014). The new combined road and railway infrastructure crossing the canal 'Leuvense Vaart' in bypass of the railway station of Mechelen (Belgium). Proc. Congress AFTES Lyon 2014, Lyon, France.

Revolutionary iso-static structural design of hinged double-decked tunnels in Antwerp Belgium

Oosterweel Link

To improve the traffic flow in the Antwerp region, it was decided to complete the R1 Ring road by building the Oosterweel Link. The initial solutions were considered either non-sustainable or were out of budget. The final solution, explained further in this project presentation, resulted in a pioneering iso-static 2×2 lane Double Decked Cut & Cover (DD C&C) tunnel. This solution satisfies political, financial and geotechnical boundary conditions, creating a sustainable solution within the targeted budget. The final design stage is nearly completed and the anticipated construction start is by early 2019.

In 1969 the Ring road (R1) of Antwerp was opened to the public in its current state. However, since then, and for more than 50 years now, the 10 km north-western quadrant of the R1 is still missing (fig. 2, box 2,3 and 4). With increasing traffic passing the Antwerp region every day and the disability to divert traffic in case of an accident, the R1 is extremely vulnerable to congestion. Nowadays, the Antwerp R1 is ranked fourth of Europe's most congested highways.

To improve the traffic flow in the Antwerp region in the year 2000 the decision was made to complete the R1 with the realisation of the Oosterweel Link (Masterplan 2020). The Oosterweel Link closes the R1, improves the access to the port of Antwerp (Europe's second largest harbour) and creates a second main road connection between the banks of the tidal river Schelde (fig. 2, box 2). Originally, the €3.2 billion Masterplan for the Oosterweel Link consisted of a 2x2 lane, 1500 m long, 25 m high, double decked cable stayed bridge over the harbour (fig. 2, box 4) and a 2×3 lane 900 m long immersed tunnel (fig. 2, box 2) underneath the river Schelde. However, by late 2009 the residents of Antwerp blocked the realisation of the bridge via a referendum. The environmental impact of the bridge on the (future) expanding residential areas in the old Docklands of Antwerp was considered to be non-sustainable.

Immersed tunnel versus Cut & Cover Tunnel

By replacing the bridge with two aligned, 2×2 lane immersed tunnels on almost the same location an alternative solution with less environmental impact was suggested in 2010 (fig. 3 and 4). This alternative design addressed most referendum

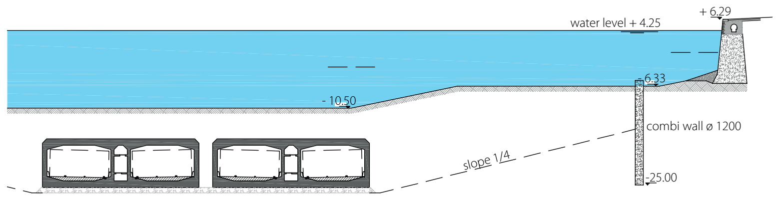
ir. Frank Kaalberg, Okke Los, Jan Ruigrok, Richard Roggeveld
THVRoTS (JV of Witteveen+Bos and SWECO)
Gert Osselaer, Benoit Janssens
Beheersmaatschappij Antwerpen Mobiel (BAM NV)

1 Oosterweel Link cross over junction from a birds-eye view
credits: Zwarts & Jansma Architects
2 Location of Scheldetunnel (box 2), Kanaaltunnel (box 4) and OKA tunnel (red intersection of box 5 & 6)
3 Immersed tunnel alignment
4 Cross section of the immersed tunnel
5 Double Decked Cut & Cover tunnel alignment

2

objections but it appeared to be €500 million costlier and thus the politicians aimed for substantial cost reductions.

Hence, in 2011 employer BAM NV appointed THV RoTS to review the immersed tunnel design with the aim to get the design within the original budget, but still meeting the same functional requirements. THV RoTS initiated a brain storm session which eventually resulted in an alternative 2x2 lane Double Decked Cut & Cover (DD C&C) tunnel design in the Canal Zone (fig. 2 box 4, fig. 5 and fig. 6). This DD C&C tunnel is supposed to be built by means of a top-down method tunnel with 45 m deep Diaphragm walls. This is definitely an out of the box solution as the deep walls (D-walls) of this DD C&C tunnel can only be realised after creating a long (but narrow) man-made island through the Antwerp Harbour by means of a 25 m wide cofferdam of sheet piles filled with sand (fig. 7). For comparison, an immer-sion trench for two aligned immersed tunnels is 120-150 m wide and requires the replacement of more than 1000 m of deep quay walls as well as the replace-ment of an intersecting bridge. The immersion procedure would also block the Harbour for 10 weekends, whereas the DD C&C tunnel saves the existing adjacent objects and keeps the harbour accessible at any time.



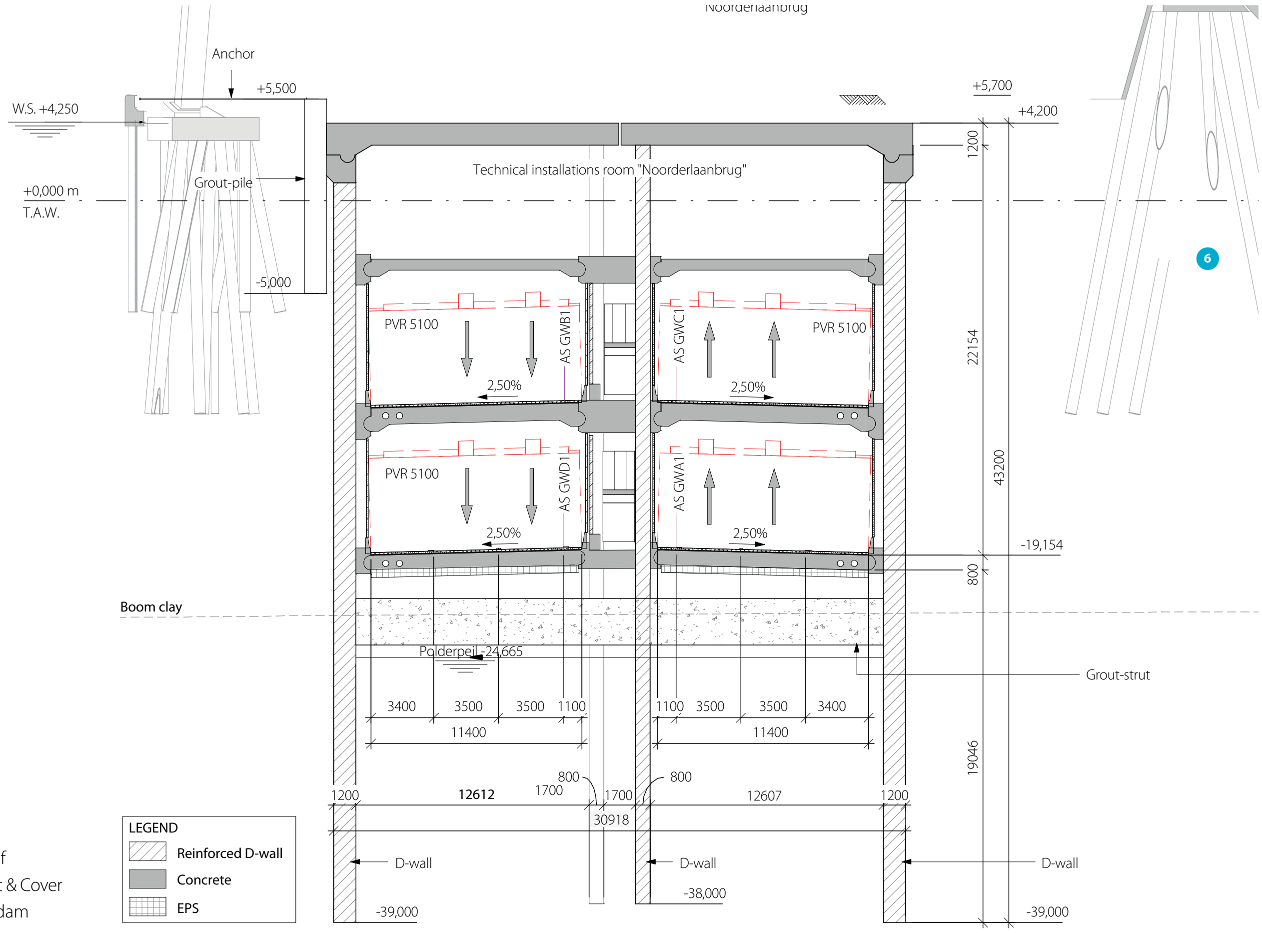
4

3

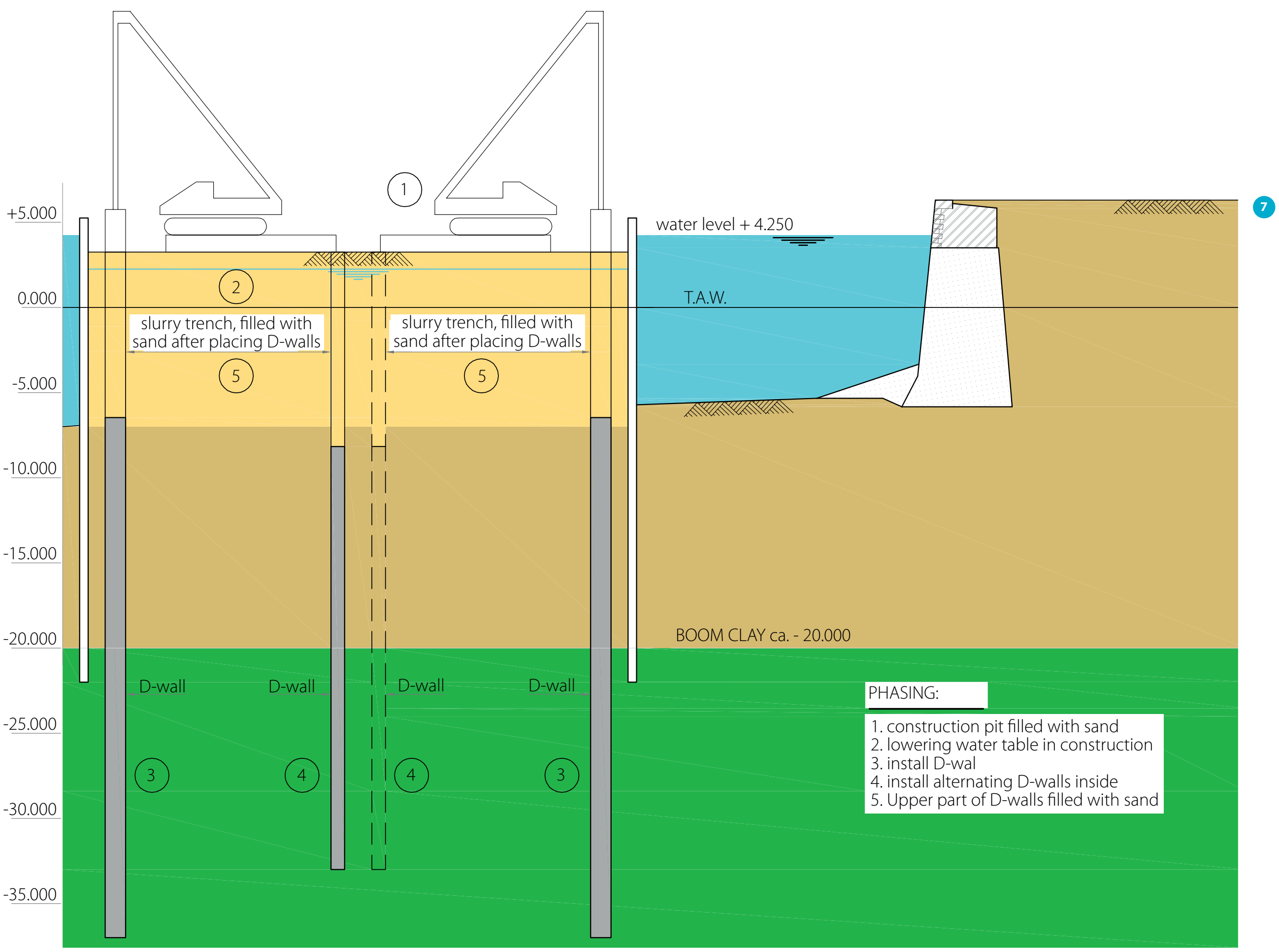
5

Double deck Cut & Cover tunnel

The approximately 25 m deep DD C&C tunnel proved to be a cost-effective design as the double deck configuration econo-mically uses the same D-walls for both road deck levels. This solution also generates cost reductions by eliminating six viaducts up to 25 m deep in the sub terrain Oosterweelknoop-punt (fig. 2, box 3 and fig. 1) as it allows an easier cross-over junction between the remaining 2×3 lane immersed Schelde-tunnel and the 2×2×2 lane DD C&C tunnel in the Canal Zone. The specific Antwerp geological conditions are particularly favourable for a DD C&C tunnel as at a depth between 20-30 m below the surface an up to 80 m thick impermeable clay layer is present, the Boom Clay. This Boom Clay layer ensures the long-term absence of high ground water pressures against the bottom slab of the tunnel by simple drainage, which provides optimal conditions for an efficient structural design. This 'polder concept' was also adopted for the very wide open C&C sections of the Oosterweel Link (fig 2, box 3, 5 and 6) thus in total delivering the unlikely cost reduction of more than €400 million compared to the original double immersed tunnels. As most of these costs reductions originate from reduced amounts of concrete and earth works, the DD C&C tunnel design reduces many tons of CO_2 and is thus providing a more sustai-nable solution. It was politically decided to invest a part of these savings again by replacing the remaining bridge intersecting the Albert canal with the OKA tunnel (fig. 8) as it provides an even more sustainable solution. This underground tunnel junc-tion is to be built with the same DD C&C construction method.



6 Cross section



7 Installing D-walls of Double Decked Cut & Cover tunnel from cofferdam

8

9

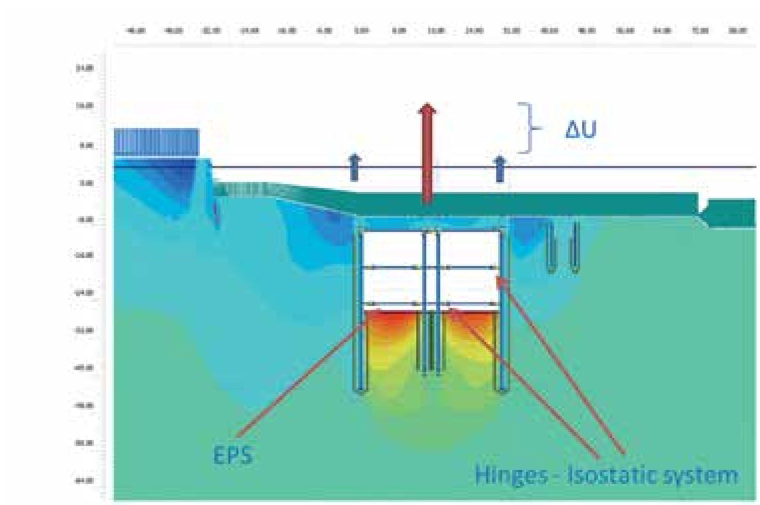
Soil-Structure interaction: Design by Testing

The Boom Clay not only provides advantageously dry excavation conditions, but also initiates several structural challenges. When excavating a deep construction pit in the overlaying Pleistocene sand layers, the Boom Clay has the tendency to swell. This not only happens at short notice but will continue over a long period of time. This will generate a substantial long term swelling pressure against the deep concrete bottom slab. As the swelling behaviour of the Boom Clay is supposed to be rather substantial but very hard to quantify, a 20 m deep full scale 20 × 20 m^2 trial pit (fig. 9) was initiated based on a 'Design by Testing' design philosophy [1]. In order to assess the driveability of sheet piles through the hard glauconite sands also a full-scale sheet pile driveability field test was initiated [2].

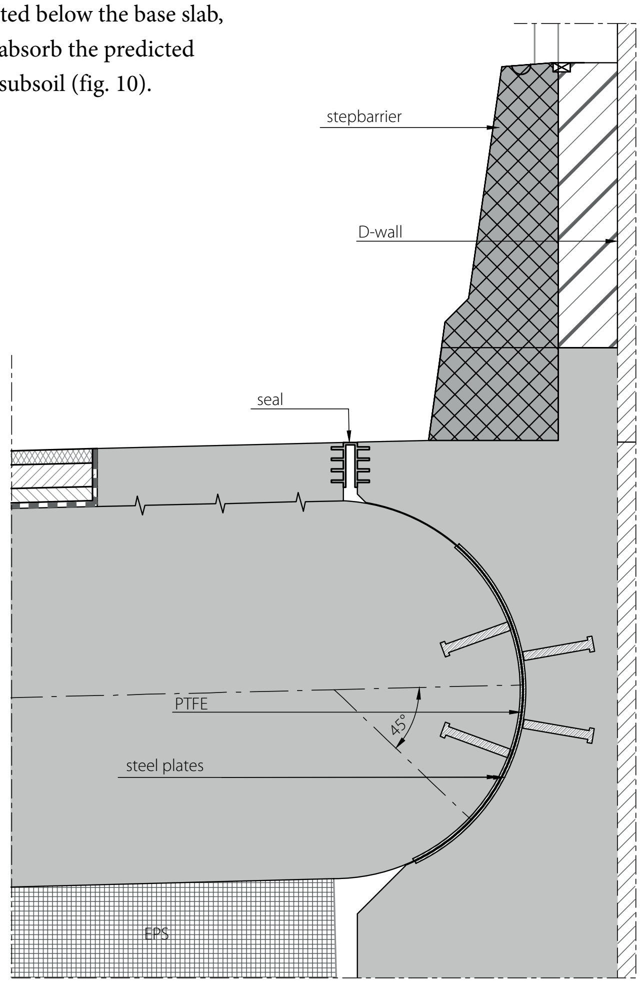
The results of the full-scale trial pit provided a lot of valuable information about the short term swelling behaviour and pore pressure development in the Boom Clay during excavation. This is very important input for the advanced FE modelling (PLAXIS) which is widely adopted for the pioneering soil-structure interaction design of these tunnels. Despite the optimisation of the horizontal stability of the (shorter) D-walls originating from the trial pit results, the absolute value for the long-term swelling remained hard to quantify as it can continue for decades, although its significance will reduce in time. Based on a literature study, the trial pit results and long term monitoring results at other locations, a long term (100 years) swelling of 190 mm is predicted. To counteract this issue a 400 mm Expanded Polystyrene (EPS) layer is implemented below the base slab, specifically designed to absorb the predicted ongoing swelling of the subsoil (fig. 10).

8 Projection of underground tunnel junction of Kanaaltunnel and OKA tunnel (in red box)
9 Full scale trial pit
10 Differential heave in FE model results
11 Detail of concrete hinge design at the base slab

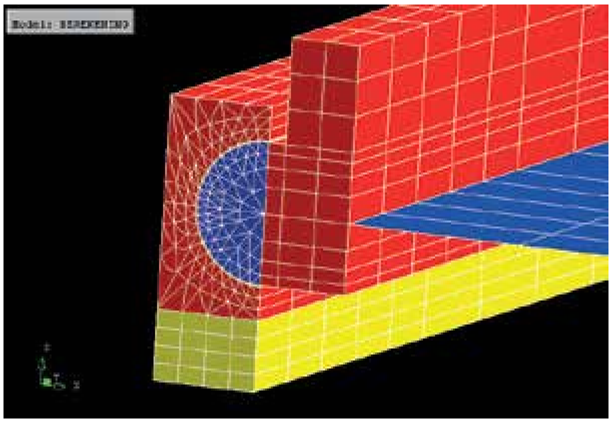
10



11



12 DIANA model of the hinge (floor modelled as a plate)
13 OKA tunnel in BIM view (looking in northern direction)
14 Cross section OKA tunnel (see fig. 13)

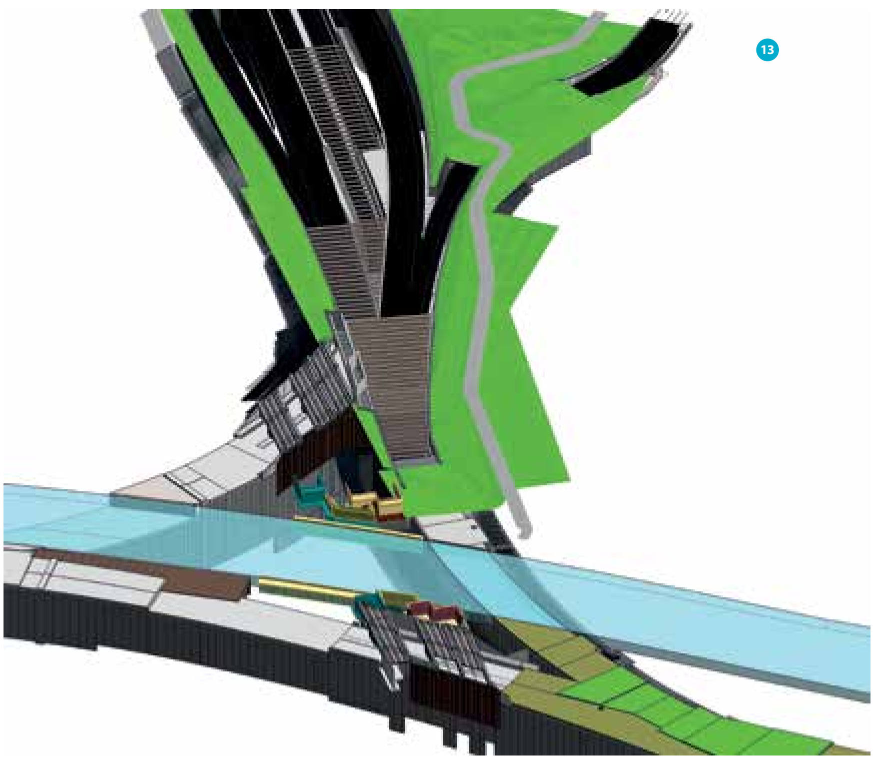


12

Iso-static double decked structural tunnel section

As the effects of long term swelling are strongly related to soil-structure interaction, significant differential heave rates between the external D-walls and the middle D-walls were acknowledged with extensive FE modelling (fig. 10). As this will eventually initiate unacceptable stresses in the usually adopted heavy monolithic concrete tunnel structures, a possibly unprecedented structural design is implemented by THV RoTS. Based on a risk driven design approach it was decided to use hinges between the concrete decks and the D-walls, in order to create an iso-static tunnel structure which allows for differential heave.

Because of the application of hinges in each joint the horizontal stability of the double deck tunnel section has to be guaranteed by horizontal soil- and water pressures as well as the deep fixation of the D-walls in the Boom clay. The long-term behaviour of the Boom clay (from undrained towards drained) affects this fixation. Hence a large number variations of FE calculations were made; variations in ground conditions on the short and long term behaviour (based on the trial pit), variation in stiffness of the D-walls, asymmetric soil loadings, load combinations etc.



13

Hinge design

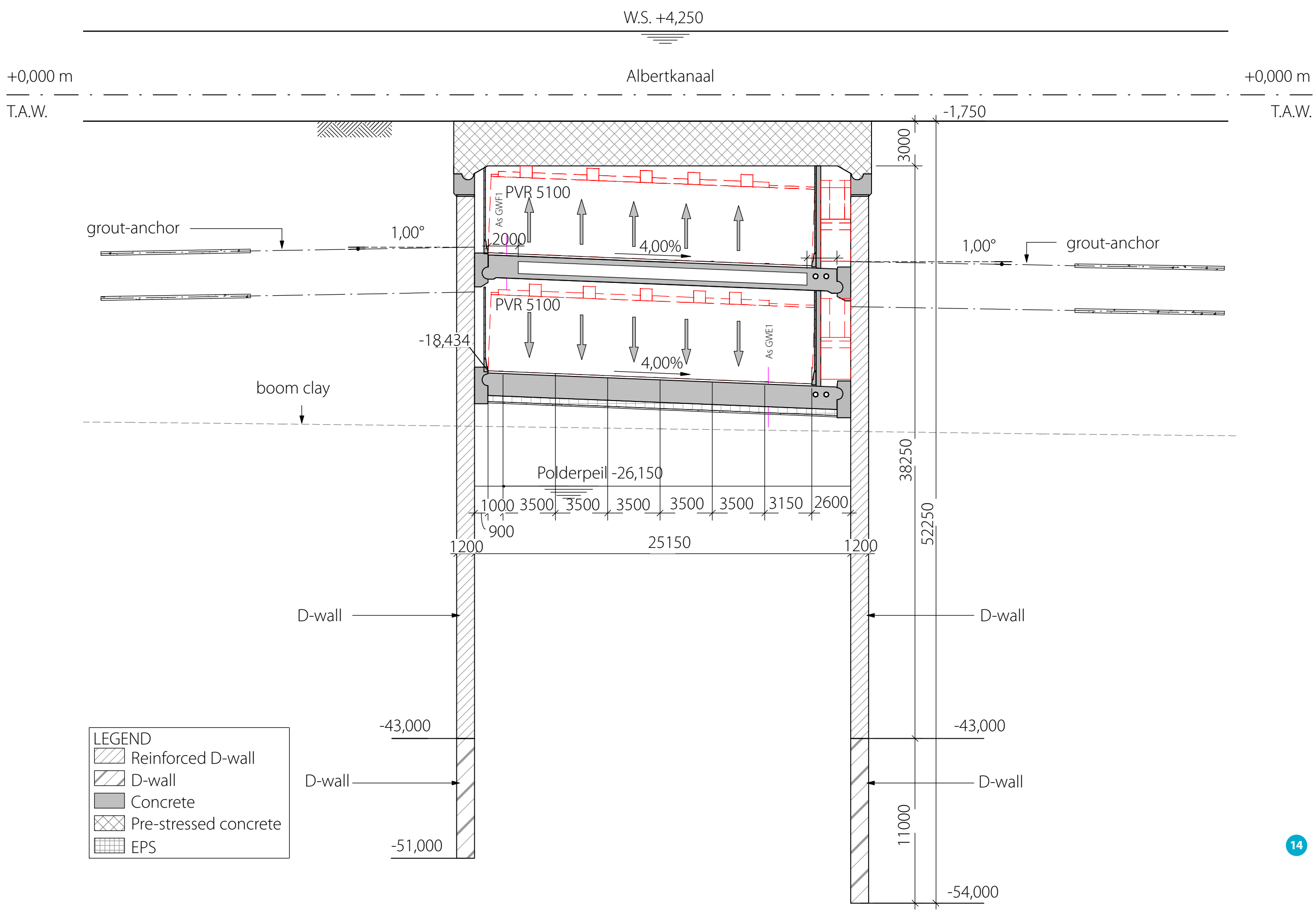
Based on the iso-static tunnel section, the boundary conditions for the concrete hinge design require to account for:

- Rotation due to: differential heave, deflections of the D-walls during excavation of the tunnel section, deflection of the decks due to creep and traffic loads resulting in a total, absolute rotation from 10 mrad up to 40 mrad;
- Strut forces: the normal forces in the decks vary from 1500 kN up to 7000 kN per running meter (ULS);
- Impact of geometrical imperfections during construction should be minimized;
- Functionality under extreme conditions as explosions and inundation should be guaranteed;
- Need for long term maintenance should be minimized;
- Water tightness of the hinges between the roof deck and D-walls should be assured.

Several known concrete hinge designs were considered but none of them could meet all the above requirements. Hence a customized circular cam-pocket design was developed (fig. 11). This concrete hinge design was extensively researched with 3D FE models (DIANA) (fig. 12). Although initially a full concrete hinge contact seemed feasible, in order to simulate microcracking and verify this design full-scale trial testing was needed. Finally, it was decided not to pursue this pioneering route but to introduce two steel plates with an intermediate PTFE layer in the circular cam-pocket in order to obtain a non-disputable behaviour of the hinges.

OKA tunnel

This newly developed DD C&C iso-static tunnel design is also adopted for the even more challenging OKA tunnel in the Oosterweel Link which is intersecting the Albert Canal (fig. 8 red box). This OKA tunnel will consist of an unprecedented 800 m long 5-lane double-decked tunnel, which is designed to



14

accommodate all transport of dangerous goods (EU-TEN Tunnel, category A). The safety level of this tunnel and the risks involving dangerous goods transport has been extensively analysed, by means of Quantitative Risk Analyses (QRA), Computational Fluid Dynamics (CFD) and by studying structural blast loads [3]. As it was decided that the OKA tunnel should remain in a state of 'not beyond repair' in case of an extreme (LPG) explosion it was decided to adopt the concrete hinges here as well, allowing the intermediate decks to fail without damaging the water tightness of the outer D-walls. In case of such a calamity the D-walls remain supported by means of additional implemented grout anchors.

Conclusions

Challenging political, financial and geotechnical boundary conditions have led to a pioneering DD C&C iso-static hinged tunnel design to complete the Ring road R1 of Antwerp, creating a sustainable solution with estimated construction costs within the targeted budget. The final design stage is nearly completed and the contractual inception stage is entered. Meanwhile, public debate is slowly converging to a widely-supported integral transport covenant in which also the remaining 2×7 lane open cut section (fig. 2, box 6) is supposed to be covered with a tunnel roof. In anticipation of the outcome of this process this open cut section is already structurally and geometrically prepared for tunneling. The anticipated construction start of the Oosterweel Link is by early 2019. ☒

REFERENCES

1 Nijs, R.E.P. de, Kaalberg, F.J,. Osselaer, G., Couck, J. van, & Royen, K. (2015). Geotechnical monitoring of a trial pit excavation towards the Boom clay in Antwerp (Belgium). Proceedings of 5th ISGSR Conference, Rotterdam, 13-16 October 2015.

2 Nijs, R.E.P. de, Kaalberg, F.J., Osselaer, G., Couck, J. van, & Royen, K. (2015). Full scale field test (sheet)pile drivability in Antwerp (Belgium). Proceedings of XVI ECSMGE Conference 2015, Edinburgh, 13-17 September 2015.

3 Heijden, M.G.M. van der, Snel, A.J.M., Geukens, B., Kaalberg, F.J., & Osselaer, G. (2016). Safety analyses of a five-lane double-decked "full-ADR" TERN tunnel in Antwerp Belgium. Proceedings of 7th ISTSS Conference, Montreal, 16-18 March 2016.

World's largest sea lock

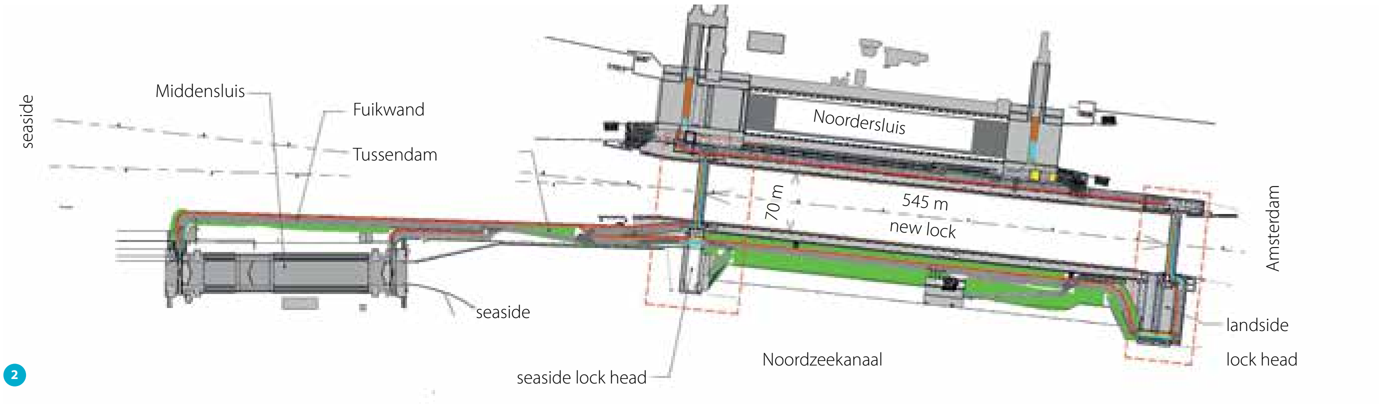
Noordersluis in IJmuiden replaced by a new gigantic concrete lock

In IJmuiden, located at the mouth of the 27 km long Noordzeekanaal that links Amsterdam and its port with the sea (fig. 1), the Ministry of infrastructure and the Environment is building the world's largest sea lock. The extremely tight construction site and the tight schedule add to the complexity of the project. The gates in the new lock will reach almost 8 m above the current water level, thus providing defence against rising sea levels. The enormous lock will be made out of 290 000 m³ of concrete.

IJmuiden has a long history as gateway to the Noordzeekanaal for sea going vessels. In 1876, the Noordzeekanaal was opened with the still operational Kleine sluis (Small lock) and Zuidersluis (Southern lock). The rapid development in the early years was crowned by the opening of the Middensluis at the end of the 19th century. At that time the Middensluis was the largest lock in the world. The construction of the Noordersluis in 1929 completed the lock complex in IJmuiden. With a length of 400 m, width of 50 m and depth of 15 m it became the largest lock in the world then.
Replacement of the Noordersluis is necessary after being in use for nearly a century. A new and larger lock should improve

ir. Paul Wernsen
BAM infraconsult
ir. Leon Lous
Volker Infradesign

1 Impression of th new lock and existing Noordersluis (to the right)
credits: ZUS
2 Overview of the new lock situated between the existing Noordersluis en Middensluis
3 Typical cross section lock chamber.



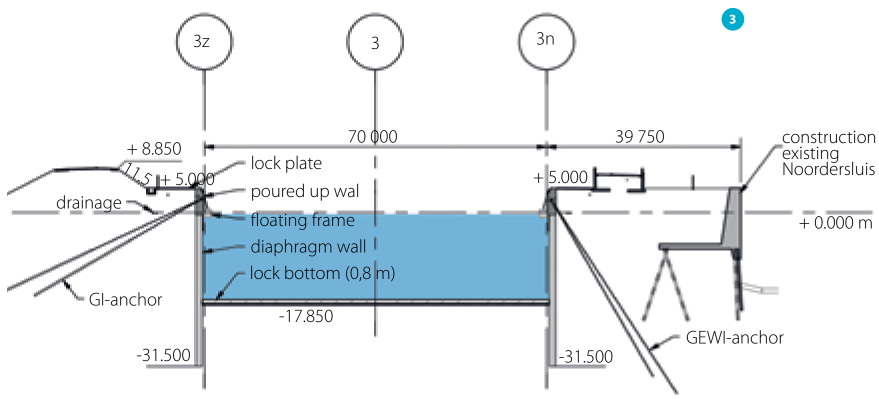
2

accessibility to the port of Amsterdam and strengthen the economy of the region by offering a tidal independent access for vessels constantly increasing in size. The new lock will be 70 m wide, 500 m long and 18 m deep (a typical cross section is shown in fig. 3). This lock will be the largest in the world and is situated just in between the existing Noordersluis and Middensluis, which are two locks currently in use (fig. 2). Continuous operation of these locks has to be guaranteed during construction of the new lock, thus requiring numerous considerations for the impact of construction on the existing locks.
The lock complex, besides having the obvious navigation facilities, features multiple other functions. A key feature is the primary flood defence. Water management (management of the North Sea Canal), passing road- and water traffic and environmental objectives (fish migration) complete the main functionalities of the new lock.

The walls of the new lock are primarily constructed as anchored diaphragm walls. The execution of this type of structure has a lot of advantages compared to the execution of steel combi-wall or sheet pile walls with respect to (sound) nuisance and vibrations which could influence the stability of the existing locks and of the primary flood defence.

The new locks will be provided with rolling steel gates that are parked in a gate chamber when the gate is in open position. The seaside upper lock head will have one gate chamber. The lower lock head will have two chambers, one for the operational gate and an additional chamber for a spare gate. The concrete structures holding the gate chambers have an area of 80 × 26 m² and 80 × 55 m² and a height of 30 m.

The contract for the design, construction, finance and maintenance during 26 years was awarded to the combination OpenIJ, consisting of BAM-PGGM, Volker Wessels and DIF. Design of the lock started in September 2015 and construction in 2016.



3

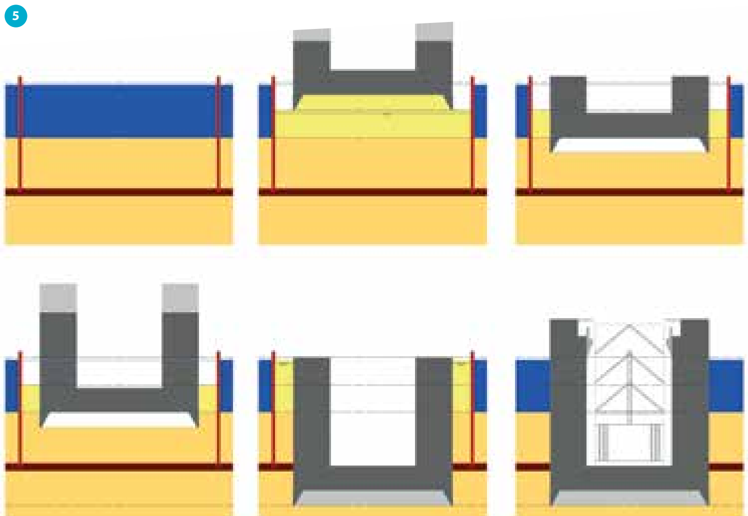
Functioning and stability of the Noordersluis and Middensluis

The realization of this large lock in the lock complex forms a risk for the stability of the Noordersluis and Middensluis. The lack of space in the complex means the construction site will be very close to these existing locks.

The lock complex has a complicated soil profile due to its history with liquefactions and major breaches. Rijkswaterstaat has undertaken extensive ground investigations as part of the preparation for the project. This resulted in limitations on the horizontal and vertical deformation at the top of the existing locks, limitation of vibration of the existing structures and limits on ground water levels and groundwater pressures near the existing locks. With the obligations to monitor deformations and ground water pressures in time, it is possible to construct the new lock at the prescribed location with a minimum risk of failure of the existing locks. For example: the horizontal displacement of the existing locks is limited to 10 - 30 mm. This is feasible with the use of diaphragm walls.

4

5



4 Digging of the diaphragm walls
credits: Ko van Leeuwen
5 Installation of door chambers by the pneumatic caisson method

OpenIJ has carried out further research in order to determine the influence of the selected construction methods and validate the design. Field trials were undertaken to validate the prediction models used for vibration and settlement. The validation models make it possible to take effective mitigation measures, which include decreasing embankment slopes, installing rock layers and drainage.

The design of the OpenIJ is characterised by a number of execution methods which were chosen to minimise the impact on the existing locks. As much as possible, the vibration-free diaphragm wall method will be applied for the construction of the chamber walls. The two chambers for the rolling gates are constructed on ground level and installed using the pneumatic caisson method. On locations where diaphragm walls are not possible, sheet piling or combined walls will be necessary. When the impact on the existing locks is too large, sheet piles will be applied in a cement-bentonite slot and tubular piles for a combined wall will be drilled.

Flood defence system

The lock complex in IJmuiden fulfills the function of primary flood defence. The existing flood defence system and flood defence formed by the new lock must be assured at all times.

In the preparation phase Rijkswaterstaat has undertaken research on the requirements for the primary sea defence system for the new lock, taking into account the possible sea level rise due to climate change. The most important requirement is the retaining height of 8.85 m + NAP for the flood defences with exception of the sea side lock gate which must meet a retaining height of at least 7.8 m + NAP. This reduced retaining height results in an additional volume of waterhazard. Considering the complete system of flood defence, this additional volume is limited, because the width of the lock gate is only a very limited part of the flood defence line. Considering the local situation, it is checked that the water storage capacity of the Noordzeekanaal is more than sufficient to deal with this volume.

Part of the contract of Rijkswaterstaat, amongst other things, is providing sufficient robustness of the flood defence system. OpenIJ created this additional robustness by using the flood defence height of 7.8 m NAP for both the seaside and landside gates and applying walls along the lock chamber with a retaining height of 8.85 m NAP. As one gate will always be closed there is always a closed flood defence, even in the unlikely event that the seaside gate cannot be closed during a storm surge.

Diaphragm wall

Because the new sea lock will be built between the Noordersluis and Middensluis, a large portion of the Middensluiseiland has to be removed. This is necessary for the lock itself but also for the approach channel on the seaward side.

Since the Middensluiseiland is part of the flood defence, a structure must be built west of the new lock to facilitate the approach to the new lock as well as to protect the inland against high water levels. This structure, the so called Fuikwand and Tussendam (fig. 2), connects the Middensluis with the new lock. Additional robustness has been provided by the use of diaphragm walls for this structure.

The Fuikwand is an anchored diaphragm wall and the Tussendam is a coffer dam consisting of a diaphragm wall on the seaward side and a combined wall at side of Noordzeekanaal. A diaphragm wall has been selected here in favour of a steel sheet pile or combined wall because of possible ship collision. This comprises all types of collisions, including the governing collision with a bulbous bow. In case of a collision the damage of a diaphragm wall will be less than that of steel sheet pile or combined wall. A collision could result in a hole in a sheet pile or combined wall through which sand from behind the wall could run away causing damage to the dam.
The construction of the diaphragm walls started in January 2017 (fig 4).

Gate chambers

At the end of 2016, sheet piling and tubes were installed to create the temporary building pit for the construction site of the gate chambers. To construct the chambers for the seaside and landside gates, a dry working level will be created inside the building pit at 5 metres below the current water level (fig. 5). At this elevation the cutting edge of the pneumatic caissons will be build, formed by concrete triangles 2.5 m high. On top of the cutting edge the 4 m thick floor will be constructed, followed by the lower section of the up to 7 m thick walls. This first part of the caisson will be lowered by 5 m by the pneumatic caisson method. After reaching this depth the second stage of the walls is constructed and the structure is than lowered 13 m to its final depth. Use of the pneumatic caisson has the advantage that a building pit with heavy combined walls, heavy supporting frames and an anchored underwater concrete floor could be avoided. By doing so vibrations and noise could be minimized.

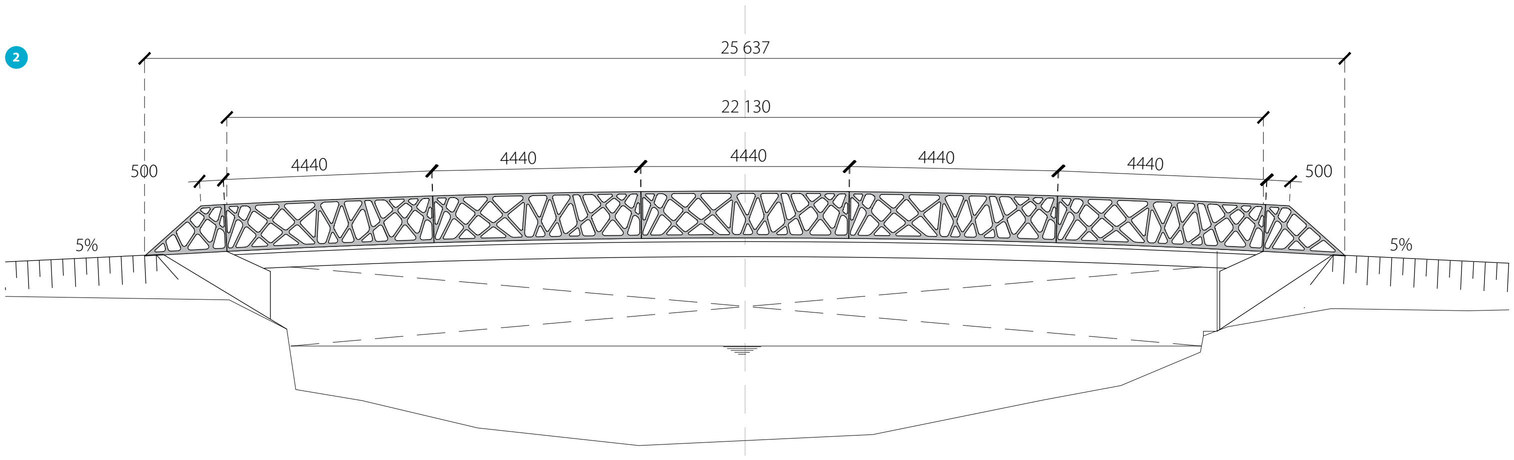
The IJmuiden sea lock is one of the so-called Connecting Europe Faciliting and is partly financed by the EU's CEF programme. At the moment construction of the lock is in full swing. The lock should, after a period of testing, be operational by the end of 2019. ☒

6 New sea lock under construction
credits: Topview Luchtfotografie

The strongest UHPC bridge in the Netherlands

Durable pedestrian bridges have become one of the most important applications of ultra-high performance concrete (UHPC) in the Netherlands. In 2015 FDN engineering designed and built an UHPC pedestrian bridge called 'Zwaaikom', over a water channel in the city of Eindhoven, The Netherlands. This project has shown that UHPC can be competitive with other materials such as timber or composite for this specific application. The proposed bridge won an open tender thanks to its exceptional durability and attractive architecture, which fits into the surrounding.

2

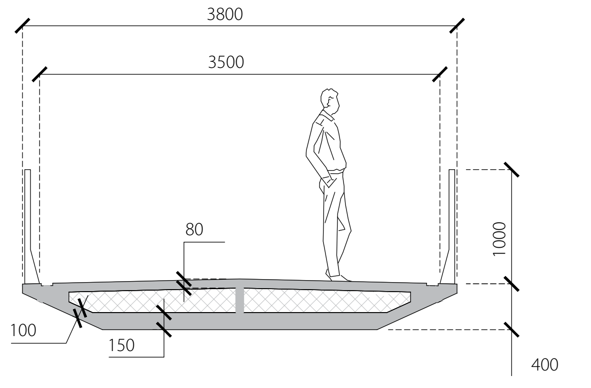


Jan Falbr
FDN Engineering + Construction

1 UHPC pedestrian bridge 'Zwaaikom' in Eindhoven
2 Side view of the bridge
3 Cross section of the bridge deck
4 Longitudinal cross section of the anchor head

Design

The bridge, fully made of ultra-high performance concrete (UHPC) C175/190 class, has a total length of 25.60 m, a clear span of 21.40 m and a total width of 3.80 m. The bridge deck is only 400 mm high. This corresponds with the slenderness of 1/55.
The cross-section of the bridge with a hollow girder box can be seen in figure 2. The hollow part is filled with polystyrene, which was used as a lost formwork during the production process. The average thickness of the cross-sectional walls is around 100 mm. The rib in the middle of the cross-section does not have a structural function. It is just for better control of concrete pouring in the bottom slab.



3

Apart from having compressive strength higher than 175 MPa, UHPC contained steel fibres which made it very ductile. In order to fully utilize advantages of its high compressive strength, beside traditional reinforcement, pre-stressing is applied.
The bridge deck was post-tensioned with five tendons (photo 12), each with 13 strands. The cross-section is hence subjected to the relative large compressive force (around 13 500 kN) resulting in large stresses (around 17 MPa). In order to accommodate the anchor system and withstand large splitting forces, a solid end beam was designed at both ends. Additional reinforcement was applied in the bottom slab to avoid pull out of the ducts from concrete since the cover of the duct was only 30 mm.

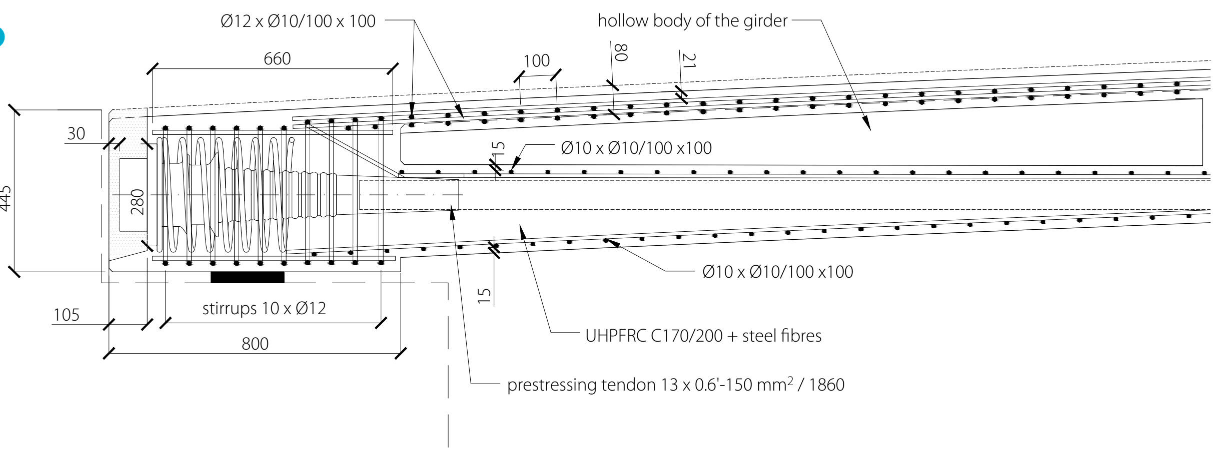
The railing elements were also made from UHPC, with a cubic compressive strength of 150 MPa. The railing has a bionic form with randomly distributed struts. (fig. 6). The railing does not contribute to the general load bearing capacity of the bridge, but it must still withstand several loads. Thanks to the high strength of the concrete, the railing struts are only 50 mm thick. The anchor bolt rails have a zinc protection against the corrosion.

Material

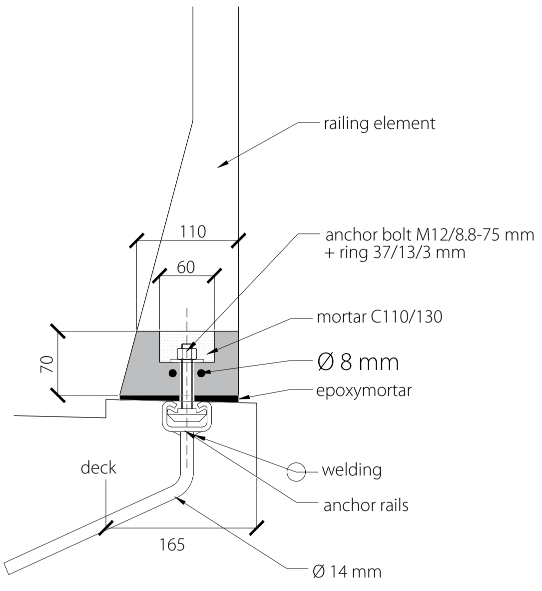
The concrete mixture contains large amount of binder (white Portland cement 52.5 with a rapid hardening process) and a high-quality calcinated aggregate (bauxite), with a grain size 0-6 mm. The water-cement ratio was 0.17. The proper hydration and thixotropic behaviour in the fresh state was assured by additives such as super-plasticizer and un-hydrated micro-silica.

The biggest challenge was to reach the required creamy-beige colour of the concrete, because the natural colour of the UHPC is dark grey. This was This was achieved by using a combination of white Portland cement, microsilica and a corresponding mixture of the pigments. Therefore, an additional experiment-

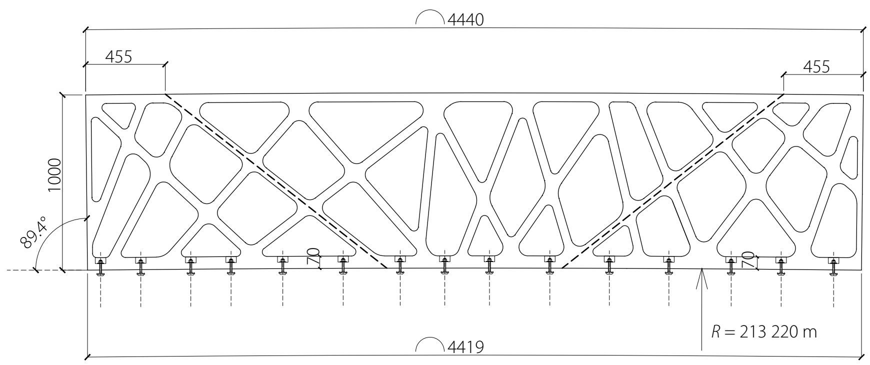
4



5 Detail of connection between railing and deck
6 Standard railing element – side view
7 An example of a dynamic response of the structure, based on calculation, to 37 joggers running over the bridge [4]
8 Calculation model for determining of the bending capacity in ULS. The distribution of stresses in the figure corresponds with an un-cracked and cracked cross-section (from left to right) [1]



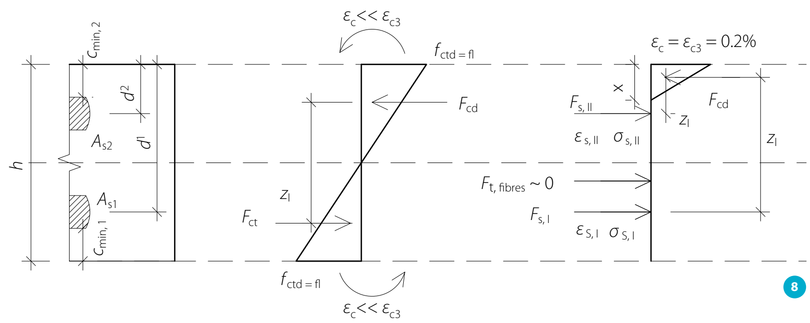
5



6

ing with these materials was necessary. The pigment tends to reduce the final strength of the concrete. Hence the designed amount of the pigment was lower than 5 % of the cement content.

In order to reach a satisfactory ductility and tensile strength of concrete, 200 kg/m³ of straight steel fibres with tensile strength of 2000 MPa have been used. The length of the fibres is 12 mm and the diameter is 0.4 mm.



8

Dynamics

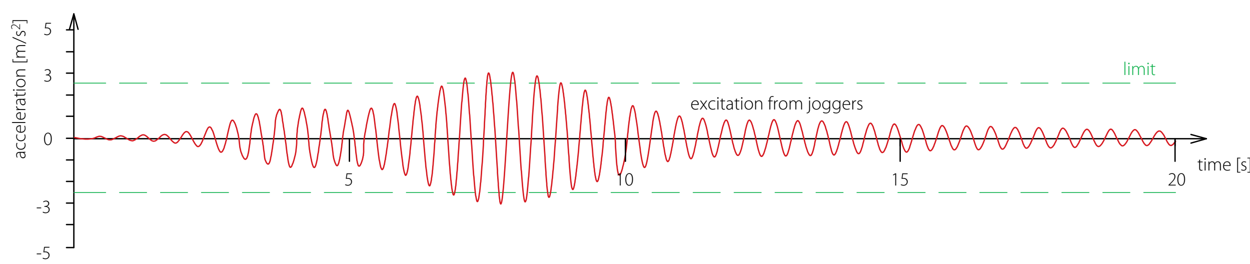
The natural frequency of the bridge is only 2.29 Hz. This value lies in the critical range for pedestrian and joggers, who can move on the bridge with the same frequency and cause unintended vertical vibration (fig. 7). Hence a detailed calculation had to be carried out. The most common methods such as SDOF and Response spectra method seemed to be still conservative since they are based on non-realistic loading conditions and simplified structural properties. The dynamic calculation was hence completed with additional differential equations, which described boundary conditions more precisely. The whole calculation was also supported by a probability study, which investigated a chance of a load occurrence and its consequent structural response.

Guidelines and calculation methods

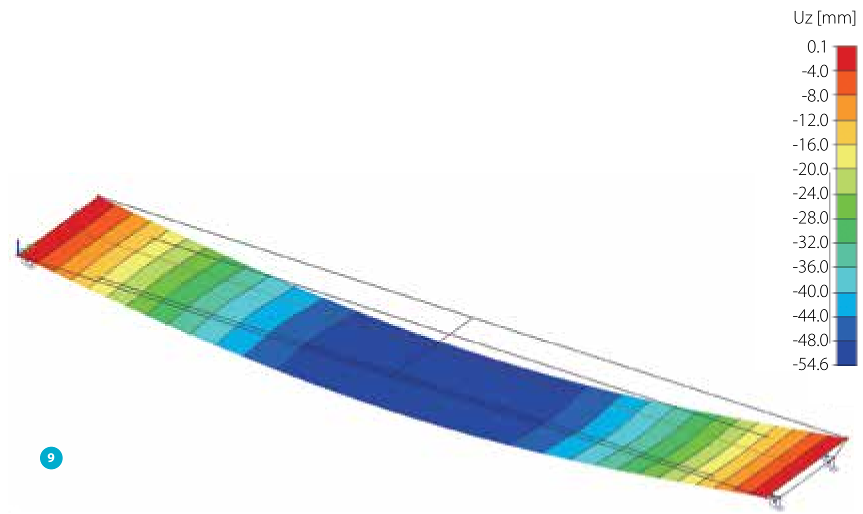
Design of this bridge has been partly affected by deficiency of understandable and reliable codes and recommendations for UHPC. The applied calculation has been primarily based on combination of French recommendations AFGC-SETRA [3] and experience, which FDN gained from the previous project. Furthermore several small and full-scale tests had to be carried out in order to give a guarantee to the client.

Due to lack of guidelines, the design of the bridge has been rather conservative in some aspects. For example it was assumed that the whole cross-section of the deck must be in compression anytime during any load case both for SLS and ULS. Tensional stresses in transversal direction in the top deck have been dealt with in a similar way as in standard concrete.

7

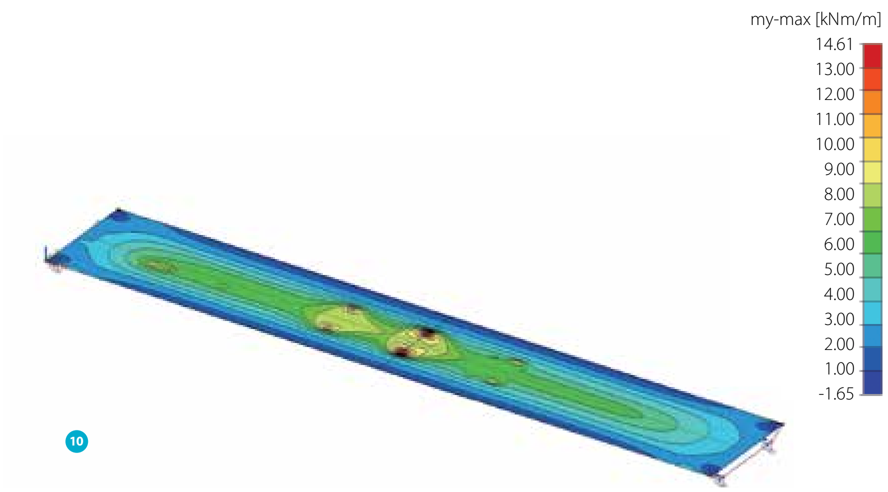


9 Output from a computer model: Vertical deformation in SLS of the bridge deck under uniformly distributed load according to EN-1991-2 [2]
10 Output from a computer model: Moment distribution in ULS in the top deck under the load combination envelope with a maintenance vehicle [2]
11 Setup for casting of one deck element. The rib in the middle ensures better control of casting of the bottom slab



9

The maximal allowed tensile stresses in reinforcement in the relation to a maximal crack width has been assessed according to EN 1992-1-1. As for the calculation of bending capacity of a cross-section in ULS, the contribution of steel fibres in tensional zone of concrete was disregarded and distribution of stresses in compressive zone has been assumed linearly, without any plastic redistribution. Due to relative brittle properties of UHPC, the maximal compressive strain, ε_c, has been limited to 0.2% (fig. 8). The random fibre orientation can negatively affect tensile strength of concrete. This has been considered by so-called factor *K*, which distinguishes global and local effects of stresses. For the case of the bridge in Eindhoven, global factor *K* = 1.75 has been adopted for the whole calculation.



10

Production

The bowed bridge deck consists of five prefabricated elements. Each deck elements was cast separately up-side down in order to control the concrete flow and to get a profiled surface of the bridge deck against the slippage (fig. 11). A proper workability of the fresh concrete was assured by a large vibration table, which was installed underneath the mould.

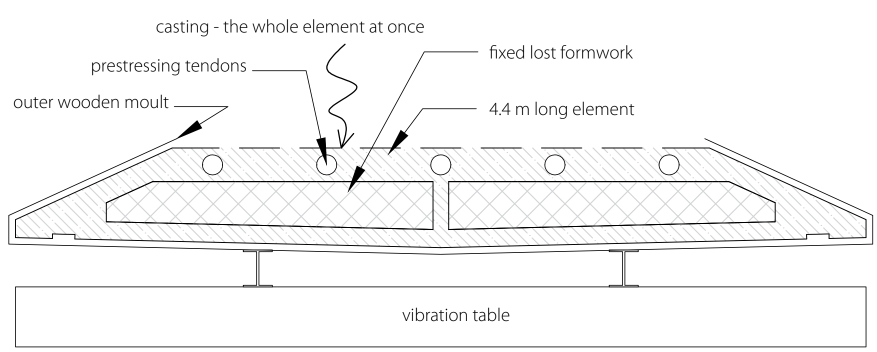
Due to a large autogenous shrinkage and fast hardening of the concrete, the casting process and curing was carefully considered. Maximum air temperature during production was limited to 25°C. The concrete was delivered in containers in several batches for each element. The pouring was performed from a height of minimal one meter in order to get a better compaction of concrete. The orientation of fibres is random and no special attention was paid to assure a homogenous distribution. After approximately two days, the sides of the mould were dismantled and the element was taken out. The elements did not undergo any thermal treatment. Only the top surface of concrete was sprayed by a convenient curing compound and covered by a plastic sheet for few days after casting.

After four weeks of hardening time, the deck elements were positioned against each other and fully post-tensioned (photo 12 and 13). No shear lock was used between the elements. The interfaces of the elements were only roughened with a special hammer and a layer of high-strength mortar was applied on both sides of every adjacent element, just before the post-tensioning. The mortar prevented stress concentration at the interfaces.

The UHPC railing elements were produced separately. A horizontal wooden mould was used. The problem to cast the complex shape of openings between the railing struts was solved by polystyrene blocks, which were made by a computer-added trimmer. Proper vibration was necessary after each casting. After post-tensioning of the deck, the elements were fixed by a special anchor system, which was cast into the deck elements (fig. 5).

The whole bridge was transported in one piece to its final location.

11



12

12 Post-tensioning of the bridge-deck elements
13 Detail of a connection between two adjacent elements

13

Testing

Due to lack of relevant guidelines for UHPC, additional testing was necessary. Both testing of material in a laboratory and full-scale tests of the whole bridge were demanded by the client. The full-scale test was performed at the production yard in order to check vertical deflection of the whole bridge. The uniformly distributed load of 4.3 kN/m^2 was simulated by water containers and concrete blocks, which were placed on the bridge deck. The measured deflection proved the theoretical calculation and showed that the bridge is safe. ☒

Conclusion

This project has shown that UHPC is a convenient material for small and medium-sized prefabricated pedestrian bridges and can compete in public tenders. Thanks to its high strength and durable properties, the bridge has an attractive design. Slender and light elements enable easy manipulation during the production process. Large durability of UHPC gives an indubitable advantage in terms of maintenance and life span. For these reasons UHPC can be considered as a serious competitor with other traditional materials such as timber or steel.

REFERENCES

1 Falbr, J. (2014). Berekeningsrapportage brugdek UO – Official design document of Zwaaikom bridge, Amsterdam, The Netherlands.

2 Falbr, J. (2014). 3D Computer modelling – Official design document, Amsterdam, The Netherlands.

3 Ultra-High Performance Fibre-Reinforced Concrete – recommendations (2013). AFGC/SETRA, France.

4 Beers, F., Buur, M. (2014). Dynamische berekening Zwaaikom – Dynamics of Zwaaikom bridge, Amsterdam, The Netherlands.

The foundation of an all glass arch bridge for the Green Village on the Delft University campus

Green glass arch bridge

The construction of an entrance bridge to the Green Village, a sustainable development for the Delft University of Technology (DUT), has started. This bridge, spanning 14 m, will be constructed with massive glass blocks (photo 2), in a shallow arch, with no adhesives: just interlocking blocks under pressure supported by two foundation blocks on piles.

The glass arch introduces considerable horizontal forces on the foundations. Especially the lateral stiffness of these foundations has an significant impact on the structural behaviour of the arch. Therefore the design process and the interaction arch-foundation required special attention.

General

The Green Village is a terrain on the campus of the Delft University of Technology (DUT) where all kinds of technical, sustainability related features will find a home. Between the Green village and the campus there is a 14 m wide canal over which a new, 2.20 m wide bridge, has to be constructed. A bridge to the Green Village has to be Green as well. Therefore, the Green Village worked out a strategy to build each five years a new, at that moment as sustainable as possible, bridge. The old bridge is to be recycled.

For the first bridge a limited design competition for employees of the DUT was announced. Since the department Structural Design of the Faculty of Architecture of the DUT had a good working experience with an experimental glass façade for the Chanel shop in Amsterdam, it was decided that the same building material, massive glass blocks, were to be used for this Green Village bridge. The glass design was selected to be the first sustainable bridge to the Green Village.

Shallow glass arch

Glass is good choice for a Green bridge because glass is a very sustainable material: it is made from sand (lots of it available worldwide), it is inert (no corrosion/rot) and it is 100% recyclable without loss of quality. And glass is transparent, a beautiful property that makes it shine and sparkle. In the façade of the Chanel shop in Amsterdam the glass blocks were glued together for structural integrity. As adhesive is not a wise sustainable connection method and the bridge has to be dismantled after five years, using glue (adhesive) was not an option. Therefore, the choice was made to create an arch, working under compression at all circumstances. It had to be as shallow as possible arch to prevent people from sliding and slipping on the bridge. Shallowness in arches has a big price: it results in large horizontal forces on the supports of the arch. In combination with the Dutch soil (peat up to 20 m and therefore long concrete piles) this is an unfortunate and possibly dangerous combination: a

prof. ir. Rob Nijsse
Faculty of Civil Engineering and Geosciences, Delft University of Technology

ir. Ate Snijder
Faculty of Architecture, Delft University of Technology

1

2

1 The finished glass arch bridge in the Green Village on the Delft
credits: Frank Auperlé
2 Massive glass block used for the construction of the bridge
3 Artist impression of the bridge

structure with limited lateral stiffness. The Structural Design group of the DUT was however convinced that with a clear awareness of this dangerous combination and the appropriate structural measurements this challenge can be tackled.

The glass arch bridge is designed and engineered by the DUT and the two abutments by the engineering firm RHDHV. The DUT was also involved in the execution of the abutments. It is important to know the state of deformation of this highly experimental bridge to assure its safety, or to be able to take preventive measures when deformations become too large.

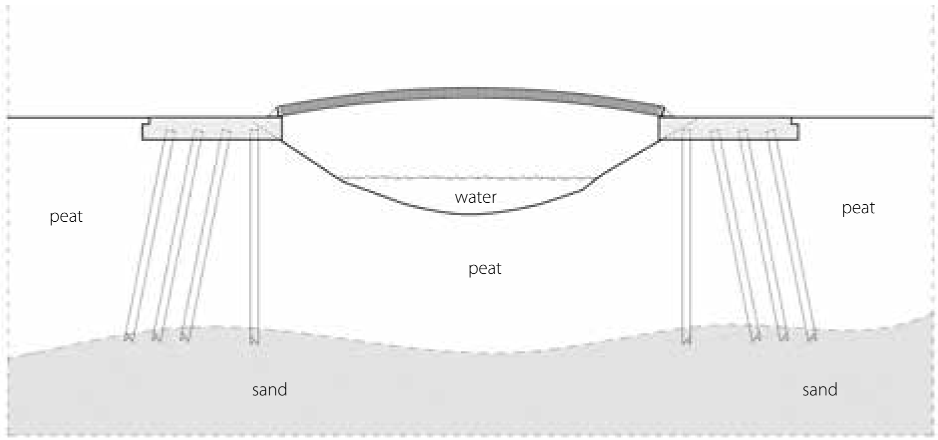
Continuous collaboration between the structure itself and the foundation has to exist, especially in the Netherlands where the soil is so bad/ weak.

Design of the foundation

In close collaboration between Royal Haskoning DHV (RHDHV) and the Structural Design group of Architecture and the Building Engineering group of Civil Engineering of the DUT the following concept was worked out (fig. 4): two big, cast on-site, reinforced concrete foundation blocks (5650 mm

3

4 Bridge, abutments and piles
5 Plan for foundation piles
credits: RHDHV



4

long, 3300 mm wide, 800 mm high) resting on concrete piles. Each concrete foundation block rests on eight piles. Two piles, close to the supports of the glass arch, are placed vertical, the other six are placed under an inclination of 1:5, an angle of about 11°. This has been done to have as much capacity as possible for taking up the huge horizontal forces from the glass arch. The concrete piles had to be 23.75 m long to find a good firm footing in the bearing sand layers under the first 20 m of non-load bearing peat. The piles measured 400 mm by 400 mm and were driven in the soil.

Calculation of the concrete foundation blocks

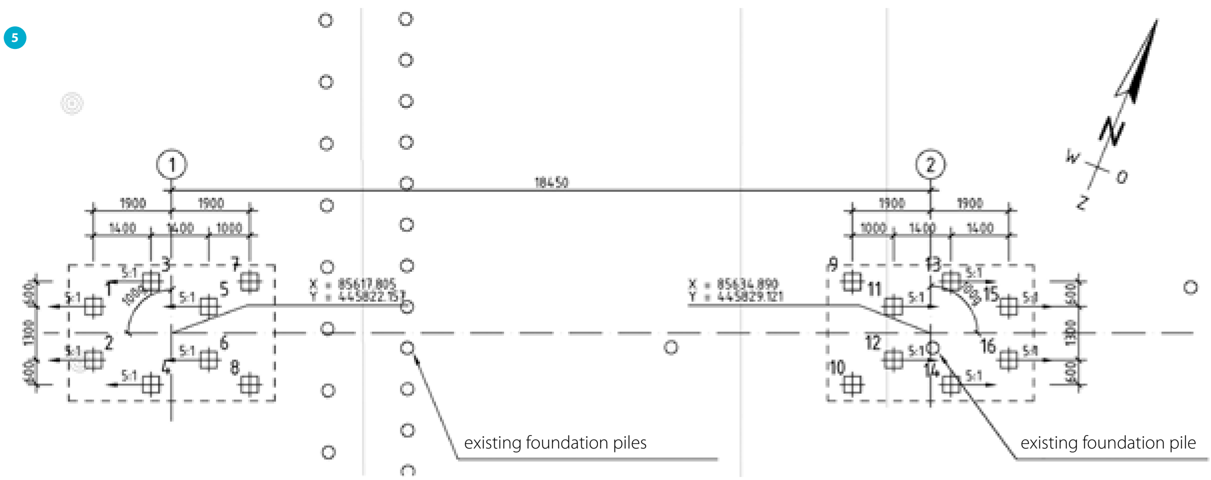
TUD made FEM calculations of the shallow glass block arch composed of loose glass blocks of 400 mm depth. Dictating loads were the dead load of 1000 kg/m^2 due to the glass arch and a live load of 500 kg/m^2 that could be placed eccentrically. As a special load case a maintenance vehicle had to be taken into account.

RHDHV calculated the concrete foundation with a finite element program in which the piles were described as tall columns supported by springs that represent the soil action. The determination of the strength and stiffness of these springs is very difficult. Also the difference between horizontal and vertical components of these spring stiffness's is complicated. The state of the art theory was followed but the real behaviour of (driven!) piles under load is rather unknown. Especially considering that the long term behaviour (plasticity and creep) is completely unknown.

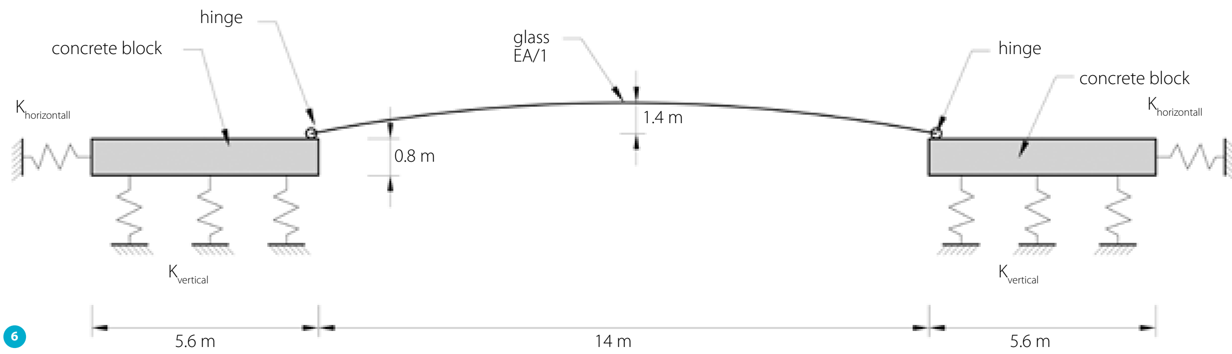
Building Code producing authorities have to be aware that real life tests on piles have to be executed to provide reliable structural properties for this type of calculations. This counts for static, dynamic and long term loadings on piles. If the engineers do not have validated data their calculations are not more than an educated guess, which may result in an unacceptable or unsafe situation.

Just the characteristic dead load of 308 kN of glass blocks already results in a horizontal force of 480 kN on each abutment of the arch bridge. Adding to this the characteristic live load, like pedestrians and cyclist, a maximum vertical load of 443 kN and a maximum horizontal load of 718 kN results.

5



6 Static diagram
7 Piles in place
8 Construction of an abutment



6

The TUD as a client laid down the criterion that a maximum horizontal movement of 10 mm was acceptable. The uncertainty regarding the springstiffnesses, especially horizontally, led to the following precautions. Directly from the delivery by the contractor the situation was carefully measured during the building process and, further on, during the life cycle of the glass block arch bridge, these values will be monitored. If the displacements of the bridge are measured to be larger than the maximum of 10 mm stiff steel cables can be attached horizontally between the concrete foundation blocks.

Execution of the foundation blocks

The real work on-site starts with one of the most critical activities: the layout of the bridge and foundation in the plan of the Green Village. Related to that the next, even more critical, item: the correct positioning of the piles (photo 7).

Only a small wooden stick pushed in the ground indicates the position of the pile to the pile driving company. Despite all the attention asked for this specific item, a misplacement of maximum 500 mm occurred by mistake. To account for this, the size and the reinforcement of the concrete foundation blocks had to be changed. Also the inclined piles were not driven in the same direction, some more to the left, some more to the right etc. Implementing these differences in the computer model showed a decrease in structural capacity of about 4% with a possible rotation of the entire block which will be continuously monitored in future. After this, making of the formwork, introduction of reinforcement (photo 8) and to-be-cast-in anchors were positioned and the casting of the concrete took place.

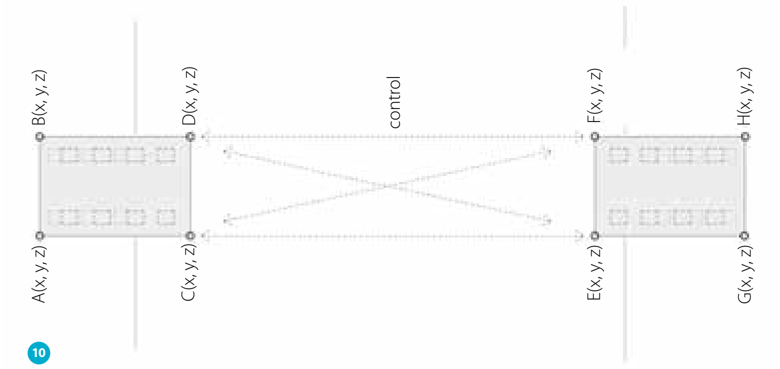
Control of the deformation

All four corners of each concrete foundation block will be, as constantly as possible, monitored and registrated for x-, y- and z-coordinates. Especially during the construction of the actual bridge, the deformation will be monitored since in this period


7


8

9 Temporarily support in a form of steel trusses and glass diagonals
10 Top view of abutments and critical measurements

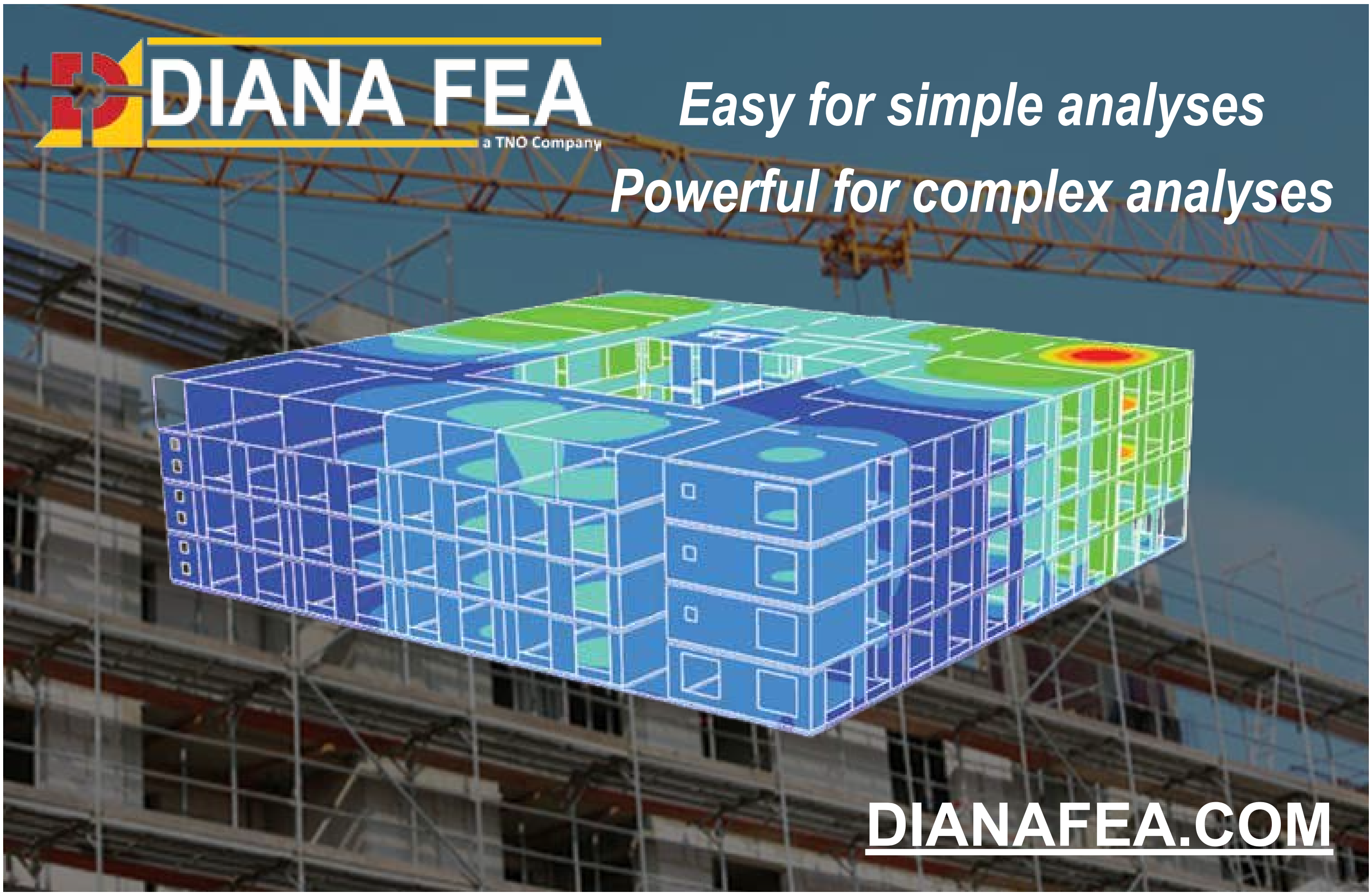


the horizontal and vertical loading on all piles will build up from 3-5% (self weight of concrete) to 100%. To build an arch with loose glass blocks a temporarily support is required. For this reason two steel lens shaped trusses were installed (photo 9). The diagonals in these trusses are another structural innovation; they are made as a bundle of massive glass bars, pre-stressed for structural safety with an internal steel bar. An arch only becomes a structural arch when the last stone is put in place. Hence the steel trusses with glass diagonals will carry up to the last stone of the arch the vertical dead load of the glass block arch; about 308 kN! So during construction of the bridge the vertical load on the tip of the concrete foundation block will grow from about 10 kN to 154 kN per block. It will be interesting to see in what way the foundation will deform during this period. A dramatic change will take place at the moment when the arch is completed and the temporarily trusses will be lowered (with jacks). In one second the, above described, vertical load from the temporarily trusses, combined with the huge horizontal loads due to the glass blocks arch, will be transferred to the two concrete foundation blocks.

It will be interesting to registrate displacements of the two concrete foundation blocks in this phase: both short time and long term behaviour. To predict what will happen with the concrete foundation blocks supported on long, inclined piles, due to the lack of accurate data, is impossible. Hence the measurements will provide more insight in the real behaviour of this experimental structure and its foundation.

Conclusion

The structural designers of the glass arch bridge, are very grateful to be allowed by the Green Village and the Dutch Authorities to construct, and monitor, such an experimental and innovative bridge (and foundation!). It will result in a large increase of structural knowledge and is a big step ahead for Structural Science. The glass bridge is officially opened on May 15 2017. This is only a temporarily version: the composite trusses are covered with stepping stones and grass. The 2200 glass blocks for the final bridge version, still need to be manufactured and put in place. ☒


DIANA FEA
a TNO Company
Easy for simple analyses
Powerful for complex analyses
DIANAFEA.COM


BEDRIJFSABONNEMENT
Toegang tot het online archief met al uw collega's!
ALLE ARTIKELEN OVER BOUWEN MET BETON DOORZOEKBAAR!
VOORDELEN:
Toegang met het hele bedrijf
Aantrekkelijk tarief
Inloggen behoort tot het verleden
KORTING!
Sluit nu een bedrijfsabonnement op meerdere titels af en ontvang een aantrekkelijke korting!
Voor meer informatie over tarieven en kortingen kijk op www.cementonline.nl/bedrijfsabonnement
Scan de code en ga direct naar de website ->
Andere titels zijn: Betoniek, CUR-Aanbevelingen, Energie+, Renda, Toets en Toolkit.
Direct aanvragen of vrijblijvend een offerte op maat?
www.cementonline.nl/bedrijfsabonnement
cement

Designing and Building the new E-line station Den Haag

From architectural vision to reality

The light rail connection between Rotterdam and The Hague was using the heavy rail tracks of the Dutch Railway company as a temporary solution. However, in the final situation, a separate platform was required. This paper describes the design and execution of this new platform which was complicated due to the restricted size of the building site and tight time schedule.

1

Jorrit Blom
BAM Infraconsult

1 Haags Startstation E-lijn
2 Artist's impression
3 Situation

On August 22nd 2016, approximately 2 years and 4 months after the project was awarded to BAM Infra, the 'Haags Startstation E-lijn' (photo 1) was opened to the public.

Prior to the contracting phase the stake holders, such as the City of The Hague, public transport companies of The Hague (HTM) and Rotterdam (RET), ProRail and various advisors, such as ZJA Zwarts & Jansma Architects and Movares consultants & engineers studied various options and conceived an architectural solution that fitted into the masterplan for The Hague New Central. In the proposed solution emphasis was given to transparency and slender curved shapes. This architectural design was the basis for the public procurement of the project. (photo 2).

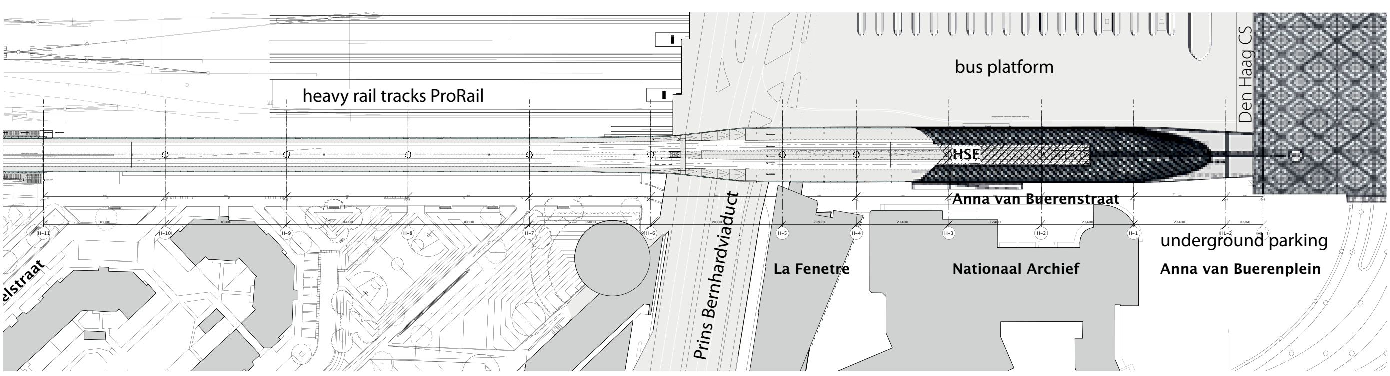
In the phase prior to contracting it was determined that the station should be located on level 2, above the existing train station (level 0) and the bus station (level 1). This decision was driven by the lack of space for a new light rail station in between the existing station, bus platform, underground parking in the Anna van Buerenstraat and other existing structures (fig. 3).

At the location of the station a 323 m long viaduct through the Anna van Buerenstraat passing over the Prins Bernhardviaduct into the Laurens Reaelstraat was required. The passage over the Prins Bernhardviaduct with a 4.70 m clearance determines the level of the station. After this passage the rails lower to ground level with a 3,75% slope. Figure 4 gives an overview of the project. The first part of structure consists of ground works, an U-shaped concrete structure filled with sand on deep foundations and an abutment. The second part consists of a steel deck supported on 10 steel columns supported by concrete bases on deep foundations. The last column is attached to a services building where passengers can access the bus platform or the train terminal. The last part of the steel deck is wider and is provided with a roofing of steel beams and glass. In this part of the deck is the platform where passenger can enter and leave the light rail wagons.

2

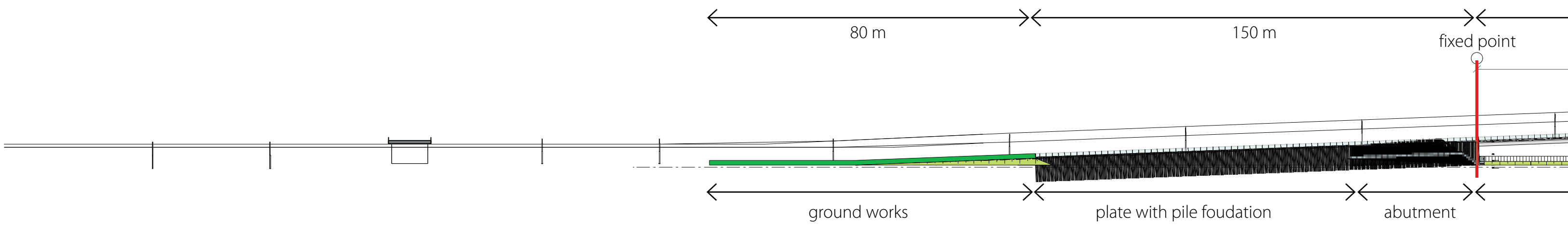

The reference design of the Client contemplated a viaduct with 10 separate steel spans varying from 22 m to 36 m placed on top of steel columns. Especially in the narrow Anna van Buerenstraat various problems had to be overcome. First of all this street had to stay open to traffic during construction because of the exit from the underground parking. Passenger streams towards and from the station had to be allowed and also access for the emergency services had to be ensured. However, a major part of the structure would have to be assembled over this street (fig. 3). Additionally both the bus station floor (rail station roofing) on one side and the underground parking on the other side had limited weight carrying properties which limited heavy lifting possibilities around the Anna van Buerenstraat.

In cooperation with co maker Iemants Steel Structure, BAM Infra opted for an important design change during the tender phase. It was decided to convert the 10 single span Client's design into a continuous bridge deck of 323 m which had to be shoved into its final position and would be prefabricated on the other side of the Prins Bernhardviaduct.



3

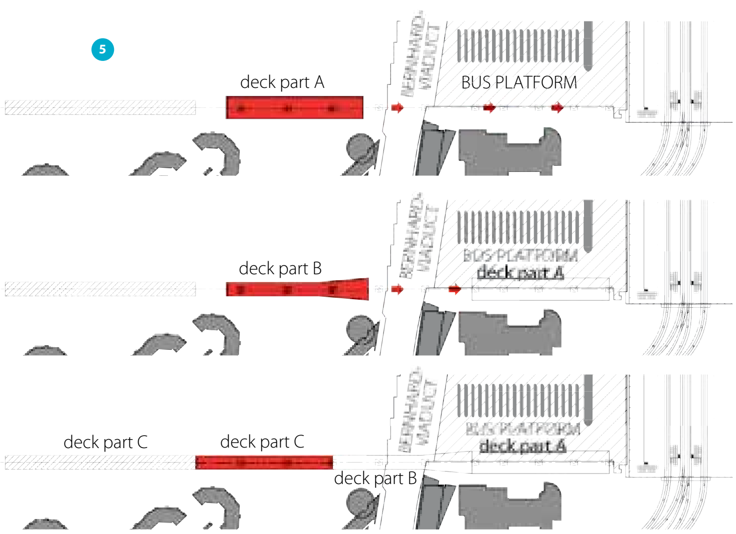
4 Longitudinal section of the project
5 Sequence of deck sections
6 Lifting jacks on right and left side, moving jack in the middle
7 Shoving deck part A over auxiliary structures (blue)
8 Position of bearings in Client's reference design.



4

Through this change the safety could be improved significantly and hindrance during the execution could be reduced. The biggest part of the construction activities for the steel deck were moved towards the Laurens Reaelstraat with more space, less passengers and better options for heavy lifting of bridge deck elements. Only the first two deck sections were shoved towards their final position. The rest of the sections were assembled directly on their final position. In figure 5 the process is schematically pictured. First deck part A was assembled on a prefabrication platform. After part A was finished and shoved to its final position, part B was prefabricated on the same platform. After part B was shoved into its final position deck parts C were assembled directly on their final position.

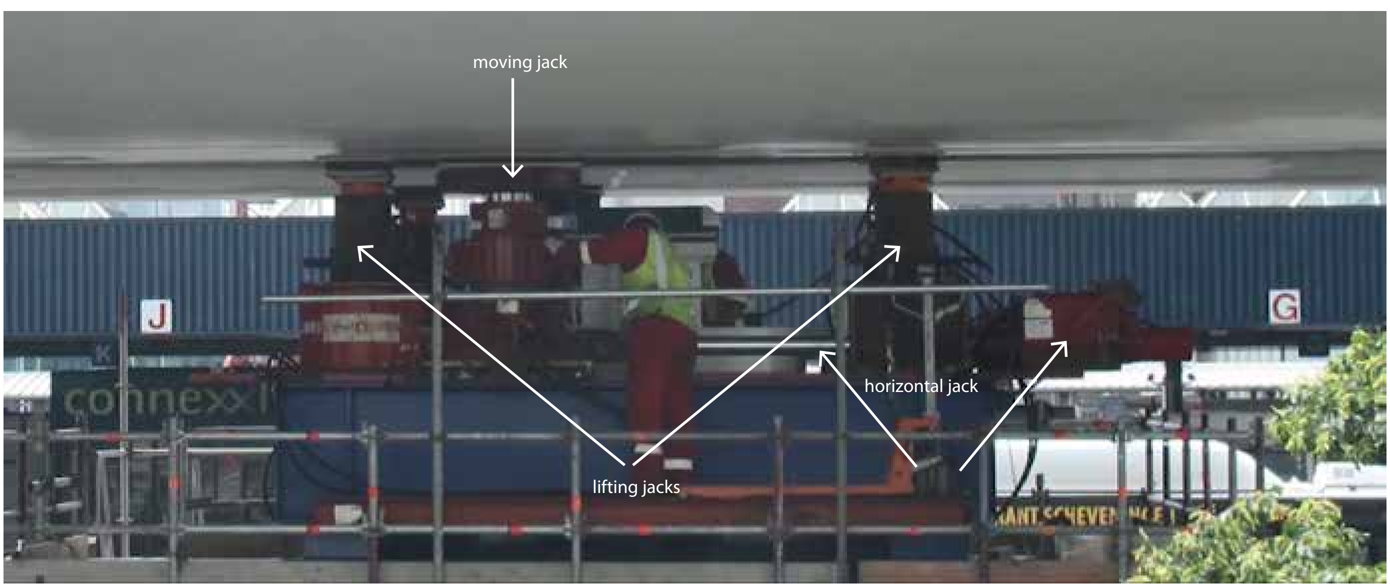
5

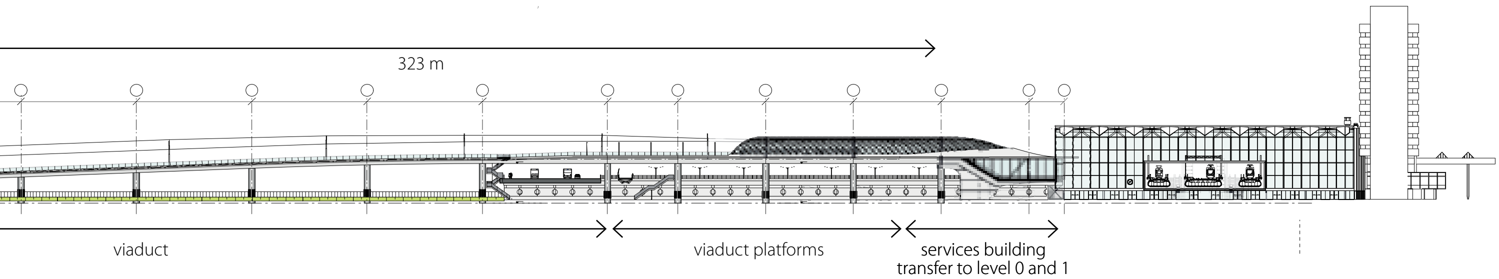


During the process of shoving, the deck was supported alternately by lifting jacks and moving jacks (fig. 6). The jacks were placed on top of an auxiliary structure around each column (fig. 7). When the deck was supported by the jack in the middle the horizontal jacks on each column made a stroke from left to right. When the stroke was finished the jacks on the left and right lift the deck and the jack in the middle was moved back from right to left where the deck was released back on to the moving jack. This way the deck part was moved approximately 2 m during each stroke. During one weekend deck part A was moved over 100 m.

By changing the designed building method some new engineering challenges had to be solved. The continuous span led

6





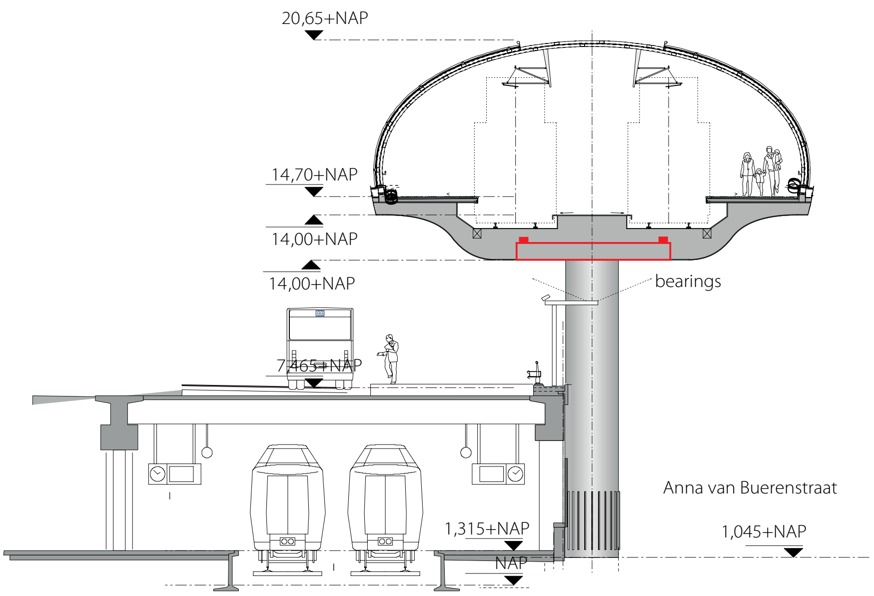
to bigger displacement due to temperature. Also the continuous girder in combination with the slender architectural columns called for special measures to assure stability of the deck. In the Client's reference design the separated decks where supported by a fork shaped beam on top of the columns (fig. 8). In order to shove in the deck, the support width of the deck had to be drastically reduced. Apart from these challenges induced by the design change it was necessary to integrate different parts of the design from different subcontractors into one integrated design solution.

7

Displacements

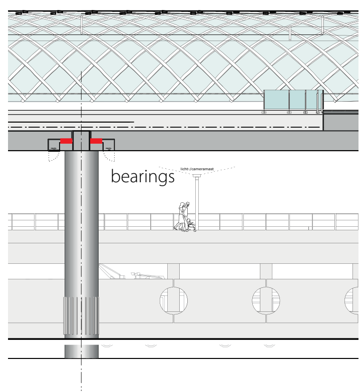
The slender bridge deck is supported by slender columns with a maximum diameter of 2 m. The top level of the rail track in the station is approximately 13 m above ground level. At the start of the viaduct an abutment is projected and on the other end a services building with elevator shaft and automatic stairs are located.

The bridge deck had to have a longitudinally fixed point. Because of elongation due to temperature the other supports had to be provided with sliding bearings. The slender columns with small foundations were not capable of absorbing the summed forces from the friction and breaking forces of the metro. Also the Client did not wish an expansion joint in the railway track. Therefore the fixed point was chosen at the abutment. This choice resulted in ULS-movements of the bridge of approximately 350 mm on the other end where the platforms are located.

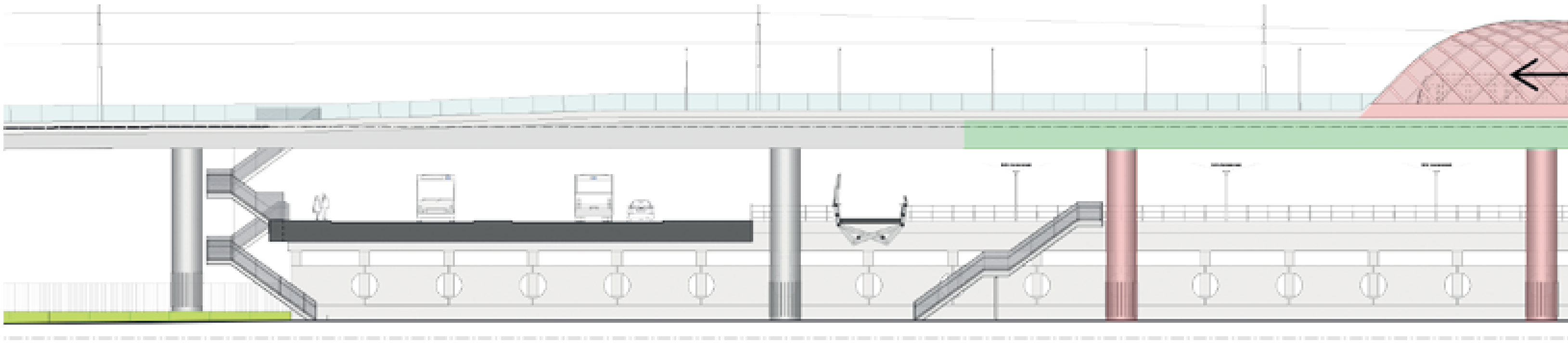


8a

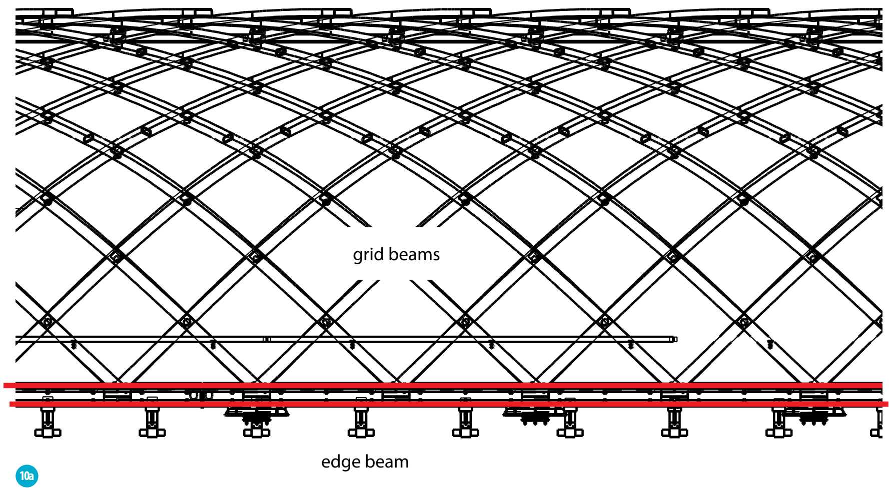
8b



9 Movements of roofing and bridge deck
10 Roofing structure (a) and mock up connection grid beams after testing (b)



9



10a

10b

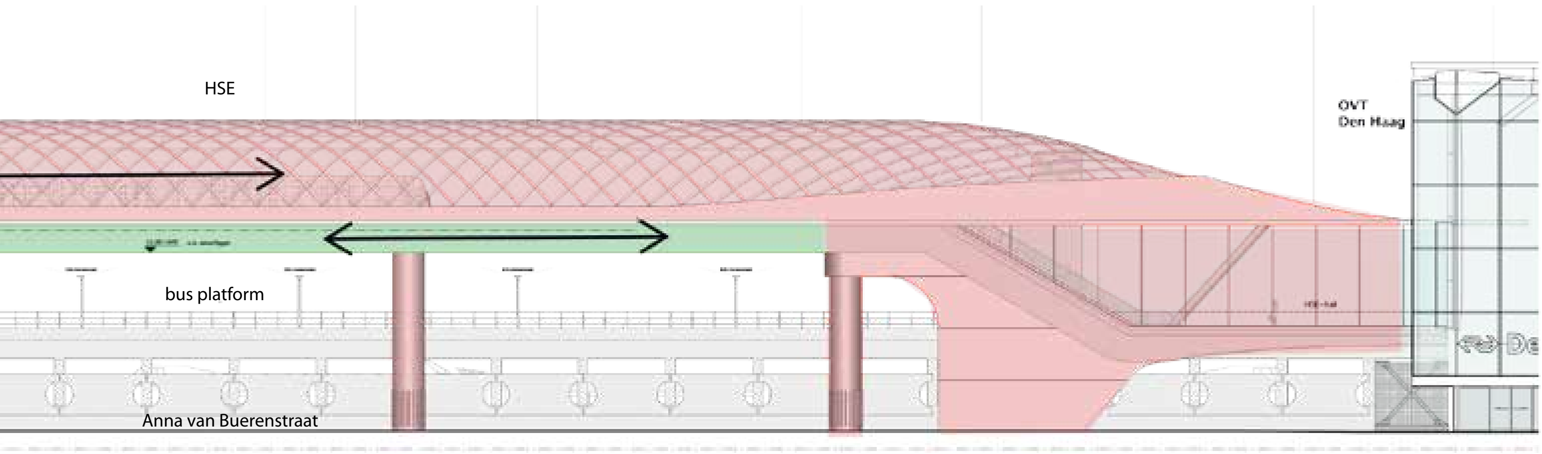
In figure 9 it is illustrated how the bridge deck slides over the supports on top of the slender columns and how the architectural roofing fixed to the services building (red) is sliding over the bridge deck (green). Therefore, with increasing or decreasing temperature the bridge deck and the roofing move in opposite directions.

The glass roofing is supported by sliding bearings placed on top of the bridge deck. The roof structure consists of an edge beam and grid beams. The complete structure is hot dipped galvanized. All bolted connections are made on site. The estimated stiffness of this connection was verified by building and testing a mock-up of this connection (photo 10b).

Integration

Aside the challenges that resulted from the design change, the architectural requirements imposed another challenge. The architect made an 'open' design using slender curved shapes. The Client had opted for a design that should be a landmark and it was contractually arranged that all designs and design stages had to be approved by the architect ZJA. The architect was to make sure that the artistic impressions made at the start would be transformed into reality.

From the beginning it was clear that due to the complex three dimensional shapes and the great variety of disciplines a Building Information Model was needed. The model started with the 3D-Design of the architect ZJA and a point cloud of the existing situation. Each design step was checked against this model. Each company worked in its own software application. The different designs were checked with the original visualization and the interfaces between the different designs were adjusted.



Emphasize was given to sharing models as early as possible to get an early insight in possible clashes and the use of limited space. Also, it was a contractual requirement to keep cables, piping and bolted connections out of sight. This resulted for example in a detail in which the bolted connections of the grid beams of the roofing were hidden behind speakers and light spots.

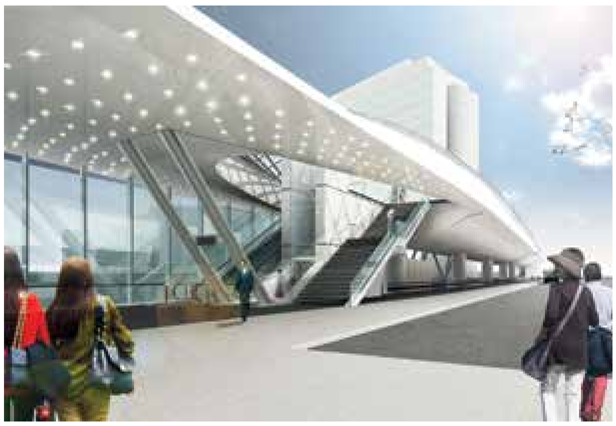
This approach made it possible to stay close to the original visualization (fig. 11, 12 and photo 13), get timely design approvals from the architect and effectively integrate different designs. Also the model was used to gain insight into difficult details preparing the execution of the works.

Conclusions

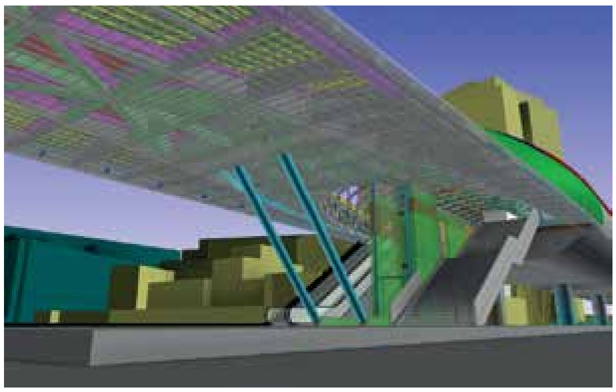
The design change that shifted the construction works from the crowded Anna van Buerenstraat to the more spacious Laurens Reaelstraat was crucial for BAM Infra to get the contract.

The use of BIM-technology and the tight cooperation between client, contractors, co-makers and subcontractors were key factors to complete the project successfully and realize the landmark as desired by the client. ☒

11



12



13

Monolithic pouring of the foundation slab of the 632 m high tower

Shanghai Tower

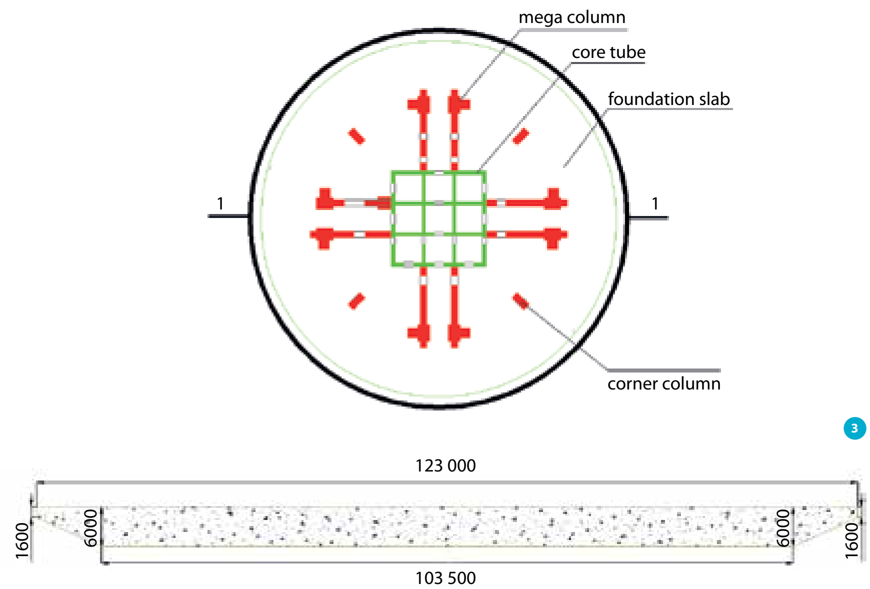
The Shanghai Tower, a 632 m high building, is the highest one in China and the second highest in the world. The foundation raft of high-rise buildings is usually a huge mat with cast-in-place mass concrete. It is a structural member that transfers loads from the building to the foundation base. To keep the integrity of a massive raft is a key issue both in design and construction phase. The key point to obtain a monolithic foundation raft is via continuous pouring of concrete without providing construction joints. This would definitely invoke a problem on how to mitigate hydration heat accumulated in mass concrete during pouring of concrete. Preventing thermal cracks is critical for large concrete members and this project presentation shows how is dealt with this issue in the Shanghai Tower foundation.

To control volume variations due to release of hydration heat, guidelines such as the Code for Construction of Mass Concrete (GB50496-2009) [1] and others provisions (EC 2) [2] give limitations on pouring of concrete, for example, a maximum volume of poured concrete per unit time and a minimum interval between pouring of each batch. Research interests are raised worldwide to explore measures to avoid thermal cracks in mass concrete. Accordingly, many construction approaches aimed at decreasing hydration heat and controling of heat accumulation in a massive raft have been developed in practice. For example, the pre-installed pipe system method was applied in the 13 500 m^3 mat foundation of Jinmao Tower building [3] and layered construction in the 9 m thick foundation raft of the World Financial Center [4]. This method increases project costs and complicates the construction procedure. Some projects such as Petronas Towers in Kuala Lumpur, Malaysia, were executed by continuous pouring in 54 hours [5]. Excel Warehouse Project

Jian Gong
College of Civil Engineering, Tongji University Shanghai, China /
Shanghai Construction Group Co., Ltd., Shanghai, China
Weijiu Cui, Yong Yuan
College of Civil Engineering, Tongji University Shanghai, China

1 Pumping concrete for the foundation of the Shanghai Tower
2 Location and surrounding of the Shanghai Tower
credits: Courtesy to Shanghai Center
3 Foundation raft of main building: (a) plan, (b) section [mm]

[6] and Abu Dhabi's Landmark Tower [7] also used this method. For the foundation of Shanghai Tower, more than 60,000 m^3 of concrete had to be poured continuously, with no construction joints or post-cooling measures, for high-speed construction and with high-quality impermeable concrete. To avoid early age cracks, special measures such as mix design considering hydration heat reduction and a pouring organization suitable for continuous pouring were taken.



3

Outline of the Shanghai Tower

Location

Located in the central area of Lujiazui financial district in Shanghai city, Shanghai Tower is designed for offices, a hotel, a commercial and shopping mall, a conference hall and an exhibition hall, as well as leisure tourism and sightseeing platform. It consists of a tower building and a surrounding podium building. With the footprint of the complex covering an area of more than 30 000 m^2, the total useful area of the complex is about 570 000 m^2. The tower building has 121 floors above the ground, and below ground is 5 storey basement.

Foundation raft of main building

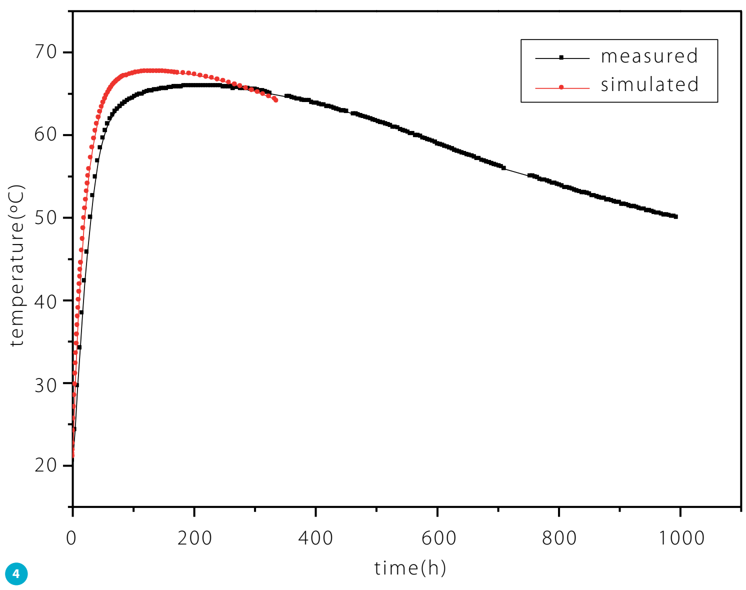
The foundation raft slab of the main building has a disk-like circular shape with a diameter of 123.0 m (fig. 3a). Along the radial direction from the center to the perimeter, the thickness of the slab changes from 6.0 m at the center to 1.60 m at the edge (fig. 3b). Special measures with respect to mix design and the organization of pouring were taken. The key challenge was to ensure continuous pouring with the limited thermal issues.

Mix Proportions

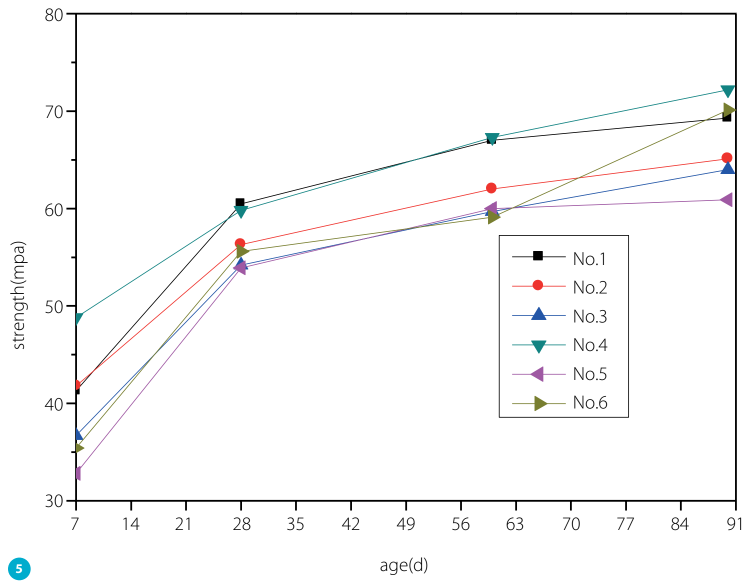
The main issue during the mix design is to minimize the hydration heat of concrete. Therefore, raw materials which assure that the low-heat concrete is obtained, are used. Locally produced cement with hydration heat of 220 kJ/kg and 289 kJ/kg at 3 and 7 days, respectively, is chosen. Beside Ordinary Portland cement, fly ash and slag, as supplementary cementitious materials, are used as a binder. This leads to a decreases of the hydration heat by 22.3% at 3 days and 13.5% at 7 days compared to the cement without addition of supplementary cementitious materials. A polycarboxylic acid

2

4 Temperature comparisons (measured and simulated in the core of the element)
5 Strength developing curve
6 The concrete pouring plan, pouring area and pump arrangement: (a) from top and (b) from side



4



5

Table 1 Mix design of concrete C50 (kg/m³)

W/B	water	cement	slag power	fly ash	sand	gravel	additional-agent
		P.O.42.5	S95	III	middle	5~25	poly carboxylic acid
0.36	160	240	120	80	760	1030	4.4

superplasticizer is added to decrease the water - cement ratio. The addition of superplasticizer resulted also in the reduction of the hydration heat by 37.3% and 24.6% at 3 and 7 days, respectively, in comparison to those without superplasticizer.

The final mix (see table 1) should also comply with the following requirements: the 28 days strength is not below 50 MPa and the slump is around 180 mm (based on pumping requirement). For the purpose of checking the thermal performance, a 6.0 m × 8.0 m × 3.0 m test block is poured prior to casting of the foundation with the same mix proportions. Temperature monitoring is carried out simultaneously. The temperatures reach the peak values within 48~72 h and the peak value does not overpass 65°C. The designed mix proved to be suitable for the construction and it was further used in the actual project.

Temperature monitoring is carried out on the spot during the project. Temperature development is also simulated, and temperature comparisons are shown in figure 4. The temperature-time curves resemble except for peak values. The difference between simulated and measured temperature is 2°C. Strength tests were performed on concrete used for real pouring. The average compressive strength of concrete, obtained from each plant for foundation pouring is shown in figure 5. The strength all passes 50 Mpa at 28th day, so the concrete meets performance requirements.

Pouring Organization

Concrete supply and transportation is the basic issue of the 60 000 m³ pouring. The main principle was to fulfill the pouring work within 60 h. Careful calculation on concrete supply and transportation is combined with extensive project experience. Six pre-mixing plants are chosen with the total supply capacity of 1250 m³/h. 355 mixing trucks with an average size of 8 m³ were in charge of the transportation.

Central flowering pouring

Pouring usually takes place from one side to another side in a traditional pouring method. For this huge mass concrete member the poring distance was too long. Therefore, a new concrete pouring method, called 'central flowering pouring', is used in this project. Unlike with the traditional pouring method, pouring of the concrete begins at the center and concrete flows to the surrounding area.

Mobile pumps have flexibility and high pouring capacity, but its pouring range is limited by its arm. Fixed pumps can overcome this disadvantage. Two kinds of mobile pumps were used i.e. a 56 m long arm concrete pump and a 48 m long arm one. As the maximum pouring distance is 61.5 m long (radius of circular mat), the mobile pumps cannot reach the central area. Therefore, fixed pumps are in charge of that area. Based on their maximum pouring length, the whole slab is divided into three parts in the radial direction. The three areas in horizontal direction are shown in figure 6a. All the area divisions can be explained by the pumps' maximum length. The pouring amounts of the three areas are 880 m^3, 10 150 m^3 and 48 950 m^3 respectively. Firstly, the fixed pump started to pump in the central area. Then, when the concrete from the central area reaches the pouring boundary of other pumps, they begin to pump. Figure 6b shows the concrete is supposed to flow along a slope of 1:12.

Layout of concrete pumps

Four 56 m long arm mobile pumps are installed on the previous soil digging platforms. Other pumps are arranged along the edge of the foundation pit (fig. 6a). Based on experience and the actual situation, six fixed pumps are used for the pouring work of area 1. The theoretical pouring capacity of the three types of pumps is 40 m^3/h (fixed pump), 80~100 m^3/h (56 m mobile pump) and 60~80 m^3/h (48 m mobile pump) respectively. The average pouring speed should not be lower than 1000 m^3/h. Therefore, eight 48 m-long arm pumps are chosen. The pouring speed is:

$$V_s = Q_1 \cdot N_1 + Q_2 \cdot N_2 + Q_3 \cdot N_3 = 40 \cdot 6 + 80 \cdot 4 + 60 \cdot 8 = 1040\ m^3/h$$

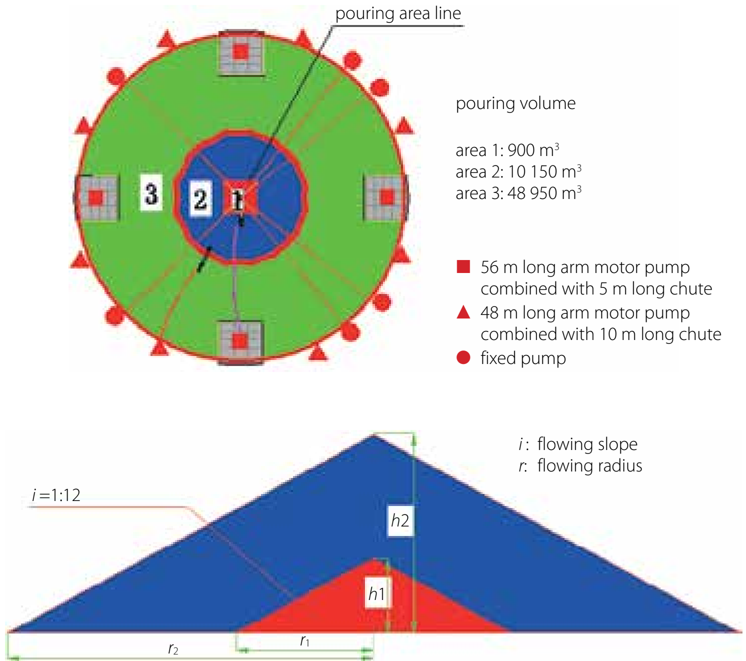
In other words, the 60 000 m^3 of concrete can be poured in 60 hours theoretically under such pump arrangement.

Pouring on the spot

All the 18 pumps are placed along the perimeter of the foundation pit. The whole pouring scene is shown in photo 1. During the actual pouring process, the fixed pumps showed some disadvantages such as inflexibility and low pouring speed. Therefore, another four 48 m long arm pumps were put into use instead of the fixed pumps. Because of the traffic jam on the spot, the pouring speed was lower than the theoretical value. Curing is carried out after concrete initial setting for each area. The main curing materials were film and sack. The coverage contained four layers: film, sack, film and sack from the bottom to the top. By this the temperature difference between the surface and the central area of the concrete was reduced.

Conclusions

The foundation raft of Shanghai Tower was poured continuously in 63 hours without construction joints and without post cooling measures. The continuous pouring for such huge volume concrete was never reported before. Mix proportion design and pouring organization are important parameter for this construction method. According to the data obtained by temperature monitoring, the used procedure resulted in the desired effect.



6

REFERENCES

1 GB50496-2009.: Code for Construction of Mass Concrete. Beijing, China Plan Press, 2009.

2 Eurocode 2, Design of Concrete Structures, Part 1-1, General Rules and Rules for Buildings. British Standards Institution, 2004.

3 Wang, T.M. (1997). *Control of Cracking in Engineering Structure* (fifth edition). Beijing: China Architecture and Building Press.

4 Gong, J. , Zhang, Y., & Yuan, Y. (2006). Construction Study on Deep Mass Concrete Foundation Slab for Main Tower of Shanghai World Financial Center. *Building construction* 28(4), pp. 251-256.

5 Nadiu, K.G. (1995). The Petronas Tower: The world's tallest building. *Modern technology in concrete construction*, 03/1995. Bangkok, Thailand.

6 Continuous cast: Exxcel Contact Management oversees record concrete pour. *Concrete Products*, March 1998; 101, 3; ABI/INFORM Complete p. 32.

7 Landmark Tower has a record-breaking pour. Abu Dhabi: Al Habtoor Engineering, September/ October 2007, p. 7.


1

Challenging Indian project in prefabricated and in-situ concrete

Infosys multi level car park

With the steady rise in the Indian economy, the construction industry has become the second largest industry which contributes to the country's growth. This growth magnified demands for quick, clean and hassle free construction technologies and has thus laid 'concrete' foundations for the precast technology in India. An example of this trend is the Infosys multi level car park in Pune (photo 1). The challenging project is constructed fully in concrete, without any other materials, with the combination of prefabricated elements and in-situ concrete.

Precast India IPL built the Multi-Level Car Park for their client, Infosys Ltd, in their Pune campus. This eye-catching, half a million square feet multi-level car park was used for the cover page of fib Bulletin 78 and was also named as 'The Outstanding Concrete Structure of India-2016' by Indian Concrete Institute (ICI). Special awards were also won in architectural, structural and precast construction categories.

In total there were 3500 elements in the building of the Infosys multi level car park, consisting of: 16 m span precast TT slabs, 15 m precast concrete columns, and 1300 façade elements each around 7 m long and weighing 2.5 tons, triangular in profile, fixed in around 160 days on-site, with a self-designed and electronically managed factory and construction site coordination, culminating in the installation of all elements within an erection tolerance limit of +/- 10 mm.

Er. Ajit Vasant Bhate
Precast India Infrastructures Pvt Ltd

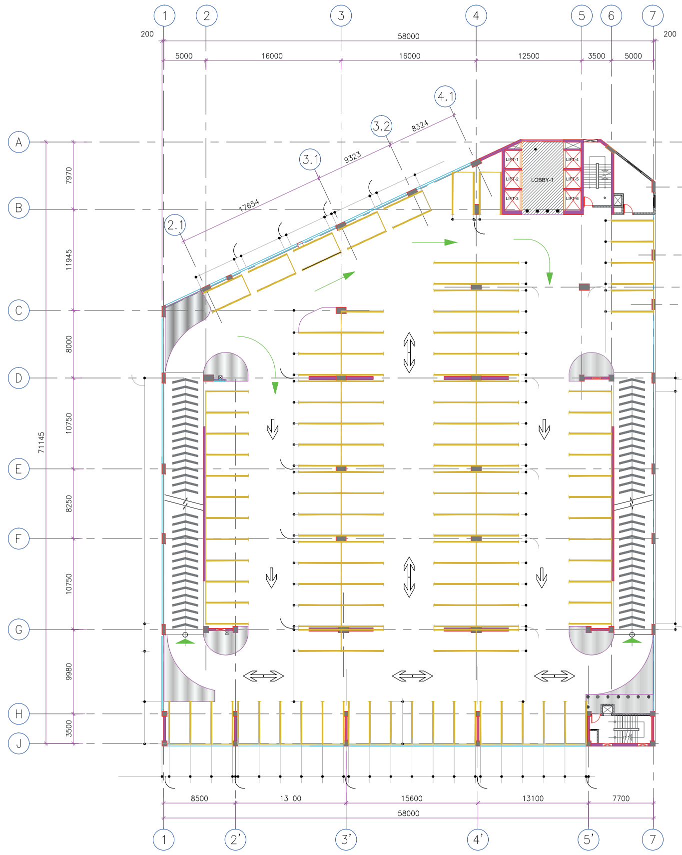
1 The Award winning façade elevation of the completed structure of the Infosys multi level car park
2 Typical architectural floor plan
3 The architectural design is inspired on a woolen weave pattern
4 The woolen weave concept elevation

The precast concrete elements in the building speak volumes about the innovations and the cutting edge technologies that were used for the construction of this magnificent structure. Concrete was chosen over glass, which resulted in excellent natural lighting and ventilation factors, minimizing future maintenance costs, and enhancing safety.

International architectural design competition

To meet the client's requirements, Precast India conducted an international design competition for the façade. From a number of designs, the entry from Arch Domingo Seminario was selected. The architecture of this structure is a perfect example of how uber modern structures can be designed to blend with the basic architectural concept evolved from and resembling the warmth of woven wool, with a virtue of unharmed air circulation along, with sufficient mild sun light entrainment in all the area.

The parking structure comprises of a basement and ten floors with a parking capacity of around 1300 cars. In addition to precast concrete columns, beams, double tee slabs, hollow core slabs, staircases, and spandrels, the uniqueness of the building lies in its 1300 odd, 100 mm thick precast concrete façade elements (photo 4) [5], all of which were produced and transported from Precast India's plant (photo 5) 45 km away from the site.
This is a unique structure of its kind in India, where, instead of glass or aluminum composite panels, structural concrete prefabricated elements are used for the façade.



2

Structural design

The structure lies in seismic zone 3 [1] and was designed and analyzed by a team of experienced structural designers, led by prof. Sypros Tsoukantas and mr. Tryfon Topintzis. The structural design of the precast elements [4] was complex as it involved intricate connections between prefabricated basement walls-columns/rafts, and floor to façade elements. The

3

4

5 Precast India plant which made intricate prefab projects possible
6 Bare façade element (a) and a mock up façade-spandrel connection (b)
7 Spandrel element with dowels for connection to column corbel (a) and column sides (b)

5

6a 6b

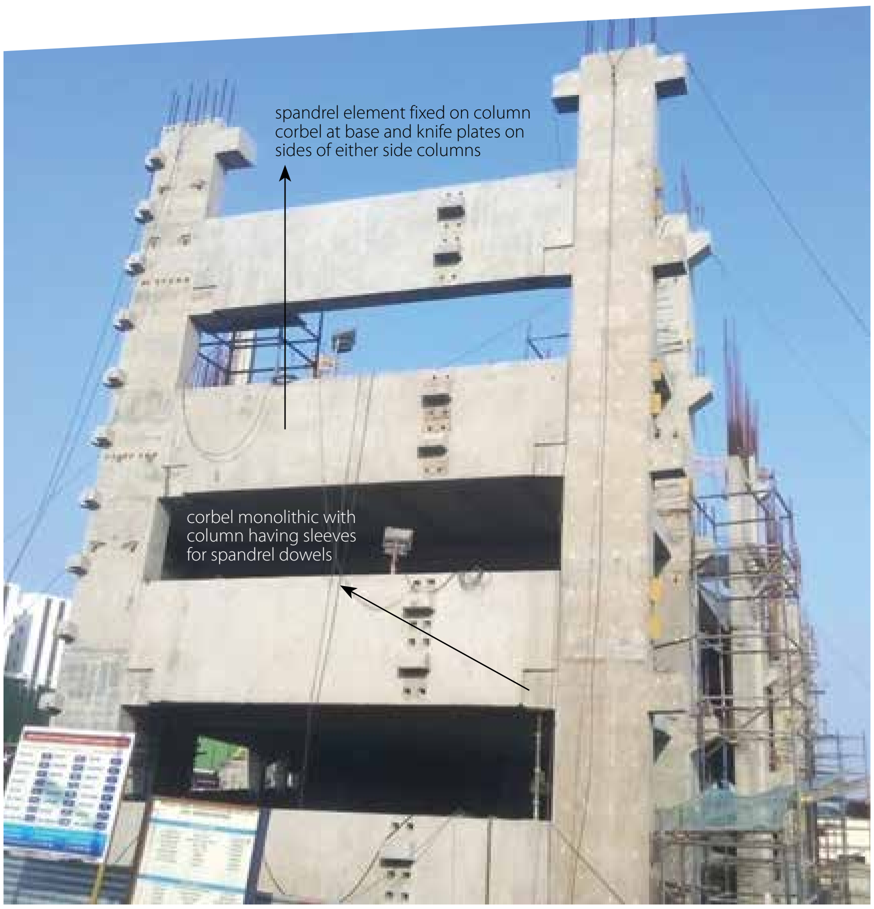
connections are ductile [3], sturdy and unyielding. Shear forces between spandrel and façade elements are transferred by the monolithic corbels in the spandrels which are also interlocking each façade element against overturning (photo 6). The ductility [3] of connection between the façade element and building spandrel was achieved by use of mild steel bolts to offer moment of resistance for 1.50 m cantilevering façade element. The use of mild steel bolts offers sufficient warning before failure in the seismic event.

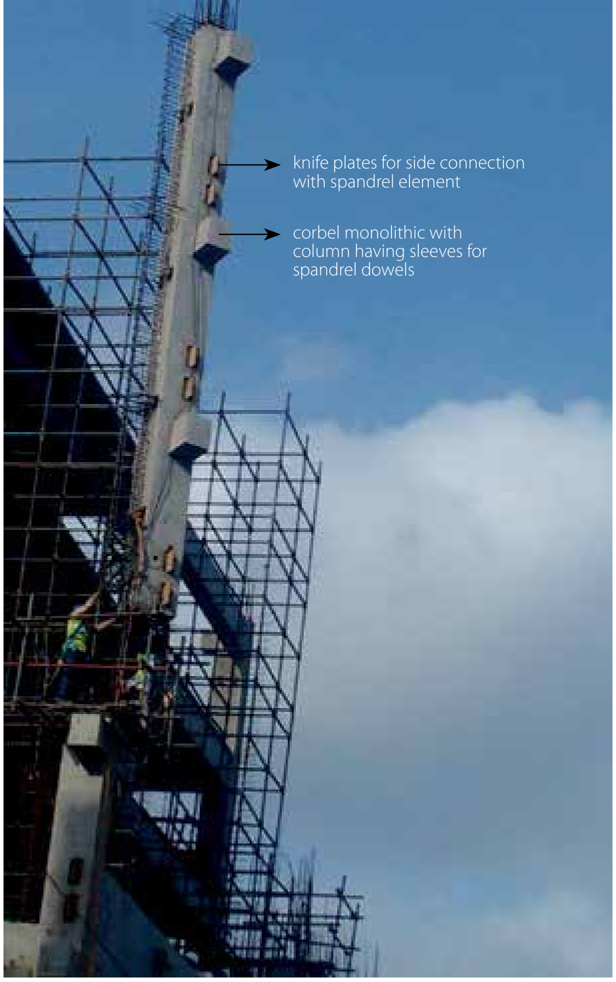
Integrated it systems

A 3D Tekla model including all reinforcement and precast connections was made. Highly accurate production drawings were generated from this model and the production of the elements was carried out accordingly.
Completely in-house developed, cloud based planning and tracking software, generated production and erection schedule along with sequence for installation and a full-fledged data base of elements. Other IT systems facilitated online booking of

7a

7b





8


9a


9b

8 Three-storey column erection
9 Lifting frames (a) and stacking (b) of façade elements
10 One of the many innovations: the double decker transport system

resources (RMC, trailers, cranes), logistics management purchase orders, material management and budget tracking.

Production and quality control systems

Even the smallest details and activities were planned well in advance before production drawings were made and unique element IDs were generated to be able to pinpoint the life cycle stages of each individual element from design stage up to final installation on site.
The state of art production facility, consisting of a hollow core slab unit, Carousel unit (for solid slabs, walls), mechanically and hydraulically operated adjustable beam and column moulds giving smooth surface finish, special batching plants, tilting tables, shear cut bend lines (steel cutting), gantry cranes, steam curing facilities, etc., produced all precast elements in the project, with a concrete grade of M40 MPa.

All the risks and challenges like demoulding, lifting, installation, anchoring etc. were eliminated and structurally approved by mockup tests. Sample prototypes were cast multiple times, and representative elements were tested till failure.
Multiple tests on raw materials, load tests for representative elements were done and certified by third party testing agencies, before full scale production began. Each test and control greatly helped in achieving the high quality and consistency with regards to strength and finish, resulting in the good durability.

Stacking

Long spans and critical geometry of elements (photos 9a and 9b) were a concern while stacking. A separate special support lifting system was designed to de-mould and stack the elements, avoiding lifting and handling stresses thereby lifting the façade elements delicately, but quickly.

Transportation to site

Transporting fragile elements 45 km through narrow and choked roads was a challenge, considering the road conditions, lifting and handling stresses. This prompted attention to

10

11 Intermediate stage of façade installation, showing a corner column
12 Interior of the fully finished structure
13 Exterior of the fully finished structure

the economical considerations and resulted in the decision to transport multiple elements simultaneously. The transportation experts innovated a double decker transport assembly system designed and fabricated as a removable attachment, ready to shift eight façade elements per trip (photo 10). This improved installation speed and enabled meeting the target dates.

11

12

13

Installation on site

The entire erection of the 3500 elements was a pre-planned activity, even before the production planning, considering all the major constraints at site like restricted side margin availability, limited access, availability of special tools and tackles etc. The installation of the odd façades was accomplished with an average erection time of 25 minutes per façade element.
For installation of TT slabs, columns, beams and other elements of the structure, a 50MT rail mounted Liebherr tower crane was set up on rails in the center of the building, considering element weight and available crane capacity at the required radius (16 tons at 48 m). 50 ton and 120 ton mobile cranes were also used.

Accuracy, tolerances and precisiom

An overall accuracy up to 15 mm without any adjustment anywhere was achieved despite the complex nature of all the precast elements. The 20 mm bolts in the façade element passing through the spandrel sleeve of 40 mm, had overall tolerance of 20 mm only, including also other tolerances during erection of foundations, columns, beams etc. This came as a result of constant coordination and exchange between all the stakeholders [9] including design, planning, production, transportation and installation of all the elements. Finally, perfect coordination between all stakeholder teams, ensured the creation of milestone and a unique game changer in the Precast industry in India. ☒

REFERENCES

1 BIS, I. (2000). Indian Standard (IS), Plain and RC-Code of Practice. Bureau of Indian Standards.
2 Standard, I. (1893). Criteria for earthquake resistant design of structures. Indian Standards Institution, IS, 1975.
3 BIS, I. (1993). 13920 Ductile Detailing of Reinforced Concrete Structures subjected to Seismic Forces–Code of Practice. India: Bureau of Indian Standards.
4 PCI Design Handbook, Precast and Prestressed Concrete (1985). Chicago: Precast and Prestressed Concrete Institute. pp. 11-18.
5 BSI, B. S. 8110-1: 1997 Structural use of concrete–Part 1: Code of practice for design and construction.
6 Building code requirements for structural concrete (ACI 318-08) and commentary (2008). ACI Committee, & International Organization for Standardization.
7 Ajit V Bhate, Jt. MD, Bhate & Raje Construction Co. Pvt. Ltd and MD, Precast India Infrastructures Pvt. Ltd., 2010-present.
8 Precast India QAQC Manual, 2015.
9 Precast India MPFC (Meta Process Flow Chart) Document, 2015.


1

Elevation and refurbishment of four lock bridges on the Albert Canal in Belgium

Modernising the Albert Canal

In order to facilitate large containerships through the Albert Canal, the four lock bridges of Diepenbeek, Hasselt, Kwaadmechelen and Olen needed to be elevated and refurbished. Through the use of state of the art techniques the existing reinforced concrete structure could be upgraded to the required level of performance.

The Albert Canal is an important waterway that links the region around Liège and the North East of Belgium to the port of Antwerp. In order to facilitate the adequate shipment of containers via this canal, all bridges need to have a clearance for at least four layers of containers (9.10 m). For that matter, a number of bridges have to be rebuilt while others are subject to elevation and refurbishment. Among the latter are the four lock bridges of Diepenbeek (photo 1), Hasselt, Kwaadmechelen and Olen. The bridges have been in use since the mid nineteen seventies. Except for the bridge at Kwaadmechelen, they only connect local roads across the canal.
The four bridges are very similar to one another. They are all triple span (12 m – 24 m – 12 m) with a central span above the lock. The side spans are incorporated in the lock walls. Viewed from above, the bridges have a shape composed of a rectangular and triangular part representing a pedestrian platform (photo 2).

The bridges are multiple box girder reinforced concrete structures with a construction height of 2.4 m for the rectangular

Robert Somers
Department of Mobility and Public Works, Flemish government

1 Lock bridge at Diepenbeek
2 Top view on the lock bridge at Diepenbeek
3 Typical transverse cross section

2

part, gradually increasing to 3.5 m for the triangular part (fig. 3). They are constructed over the lower head of the lock with which they form an integrated part.

As the canal is a busy waterway an elevation and refurbishment solution based on intensive on-site activity was not an option. On the other hand, since the roads across the bridges were only of local importance, closing the bridges for all motorized traffic during the construction works was not a problem. Bridges remained accessible for pedestrians and cyclists.
Periodical inspections of the existing bridges proved a fairly good condition from a structural point of view, which justified a partial re-use of the existing structure.

Overview of the proposed solution

Since the bridges form an integrated part with the lock, dismantling them would disrupt the stability of the lock walls. The following phased approach proved to be a workable concept (fig. 4, 5, and 6):

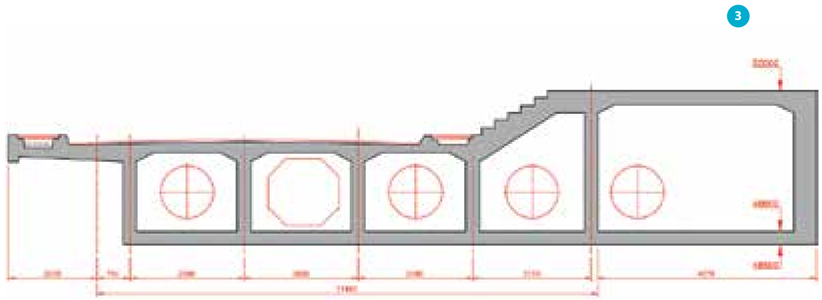
- at first, parts of the inner construction of the box girders, in particular the lower part of the transverse beams are dismantled (preparatory phase);
- the new bottom flange at the required level above the existing flange within the box girder, including the deviator blocks for the longitudinal post-tensioning, is constructed;
- subsequently, the existing bottom flange and the lower part of the girders are removed;
- as some parts the existing structure could not resist forces imposed by external post-tensioning, externally bonded Carbon Fiber Reinforced Polymer (CFRP) sheets were applied at girder webs;
- The end anchor blocks are constructed (located above the lock walls) and transverse and longitudinal post-tensioning on the new structure are applied in order to sustain the required traffic loads by a bridge deck with a reduced construction height.

Study phase

With regard to the construction phases, the following major issues had to be addressed:

- the capacity of the existing structure;
- the design of the new bottom flange;
- the post-tensioning forces, their anchoring and the potential additional strengthening measures can be without this.

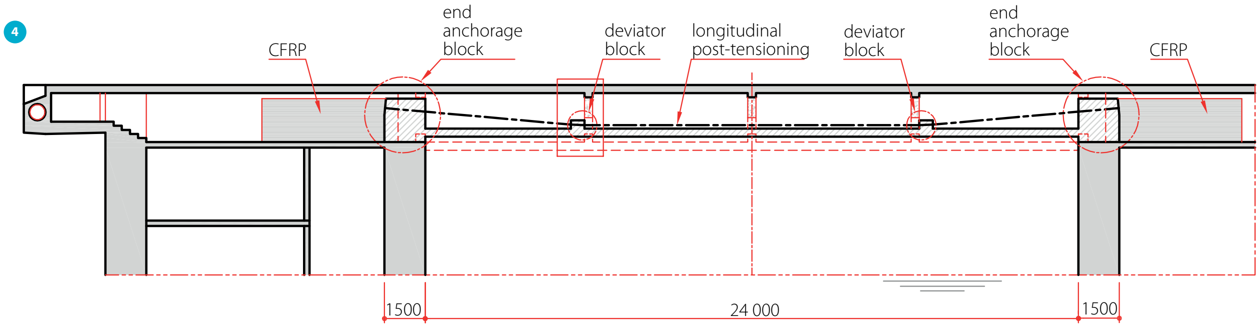
3



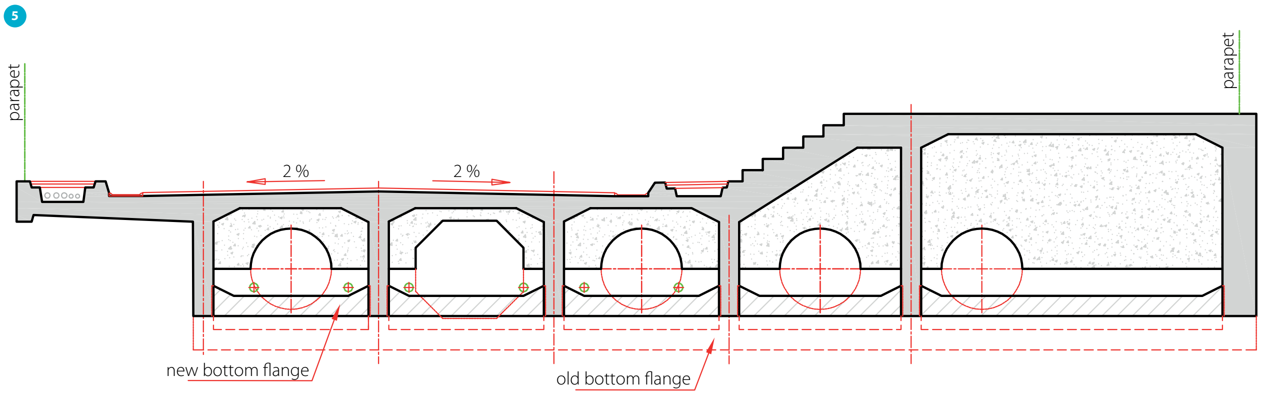
4 Longitudinal cross section of the future situation (Diepenbeek)
5 Transverse cross section of the future situation (Diepenbeek)
6 Horizontal cross section of the future situation (Diepenbeek)

7 3D Model II bridge deck (rendered view)
8 3D Model II bridge deck (structural view, six longitudinal tendons)
9 Transverse post-installed chemically anchored rebars, with underneath the formwork for the new bottom flange

4



5

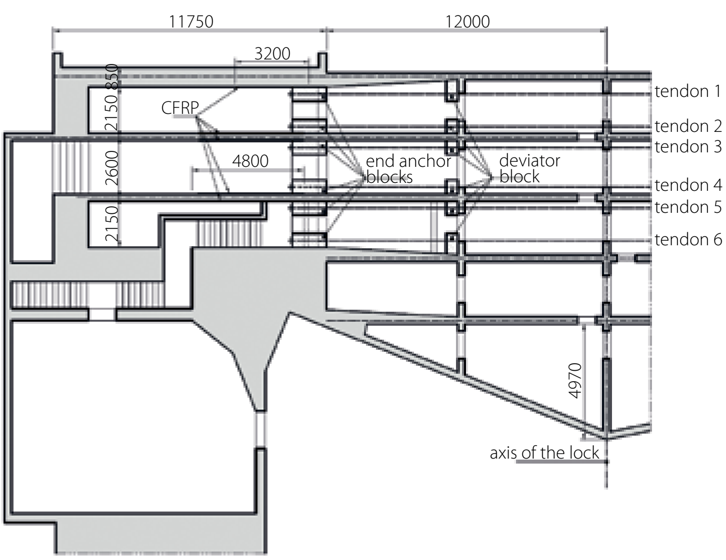


Existing structure

Although calculation notes were very concise, drawings of the existing structures revealed that concrete was equivalent to C25/30 and that two types of reinforcement with the characteristic yielding strength of 220 N/mm² and 400 N/mm² were used.

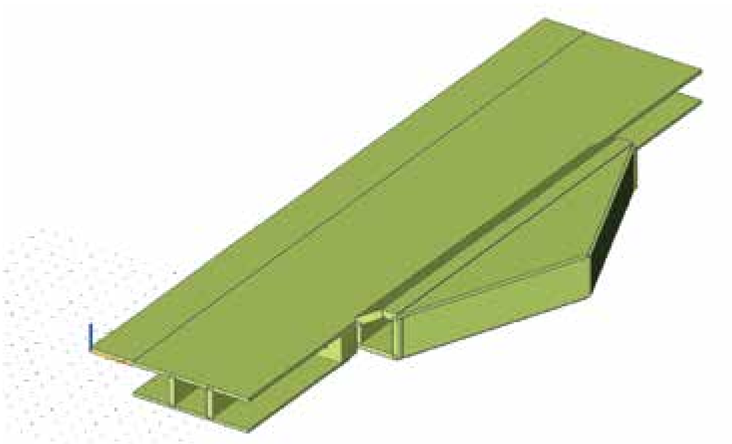
Results of cone penetration tests showed an acceptable value for the vertical modulus of subgrade reaction under the lock floor (20 MN/m³).
Available design information proved sufficient and reliable, likewise the concrete structural elements which proved to be in good shape.

6

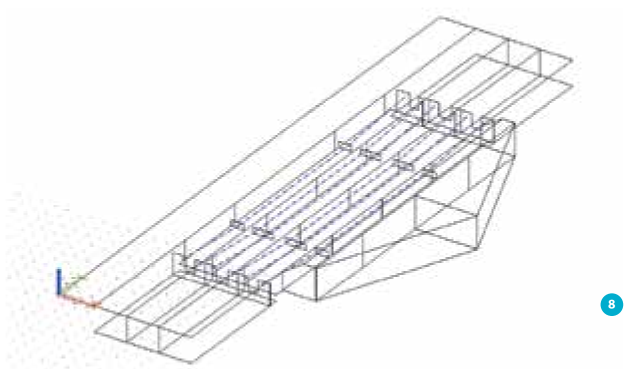


Acting loads on the integrated structure of lock and bridge

The main loads, other than the self-weight, acting on the 'lock' part of the integrated structure are the ground and water pressures and the bearing reactions by the lock doors (miter gates) in closed position, as well as service loads on the inner structure of the lock walls.
The main loads, apart from the self-weight, acting on the 'bridge' part of the integrated structure are thermal actions (uniform temperature change and gradient) and traffic loads on the bridge deck. These are based on load model 1 and load model 3 of EN 1991-2. For load model 3 a special vehicle of 1200 kN (6 consecutive axles of 200 kN, interdistance of 1.5 m) had to be used for the bridge at Kwaadmechelen as it is situated in a trunk road, but it was decided to extend this requirement to the other three bridges. Load model 1 was multiplied by a factor $\alpha = 0.80$ to take into account the fact that the bridges, although strengthened, are in fact existing structures.



7



8

Finite element modelling

In order to get a good understanding of how the various permanent and variable loads would act upon the elevated structure and which internal forces they would generate, three separate finite element models were set up (fig. 7 and 8).
The combined results of all the models were used to balance the acting forces with the resisting forces by the passive reinforcement (rebars, stirrups) and the longitudinal post-tensioning. The main 'variable' in this 'equation' was the necessary longitudinal post-tensioning force to obtain the equilibrium, as the passive reinforcement could not be altered.

New bottom flange

Before the longitudinal post-tensioning could be applied, a new bottom flange had to be constructed. It had to be strong enough to carry the self-weight and permanent loads of the mid span until the post-tensioning is applied. The new bottom flange consists of reinforced concrete C35/45 and was built using transverse post-installed passive rebars, chemically anchored to the existing girder webs (photo 9). Beside resisting loads during transient situation (prior to application of post-tensioning), this longitudinal reinforcement will also be active in persistent situation, together with the longitudinal post-tensioning.

Longitudinal post-tensioning

Based on the results of the finite element models, the following longitudinal post-tensioning had to be implemented at the central span: 6 tendons (2 per box) with 22 monostrands (150 mm^2, f_{yk} = 1860 N/mm^2). The tendons were prestressed at 0.75 f_{yk}, yielding 4605 kN per tendon.
The level of prestress was determined in such a way that, together with the longitudinal passive reinforcement, it balanced all the acting permanent and live loads on the bridge deck.
The tendon profile was such that the tendons were close to the bottom flange at mid span and near to the top flange at their extremities (anchorage blocks). In between they were deflected by deviator blocks. In this way a maximum benefit was genera-

9


10


11

ted regarding bending moments, both positive (mid span) and negative (supports) as well as shear forces. As the original reinforcement (longitudinal and stirrups) near the supports (the lock walls) had a limited capacity and could not be increased without considerable effort, the longitudinal post-tensioning had to resist both the negative bending moments and the shear forces at that location.
The stress losses due to friction were limited, both due to the low friction coefficient of the monostrands and a fairly straight tendon profile with a limited length (just over 27 m), deviated at only 2 points. After all short and long term losses, a net capacity of just over 3900 kN (85% of 4605 kN) per tendon was available.
The monostrand feature created the possibility to apply additional post-tensioning, if ever necessary in the future.

Deviator blocks

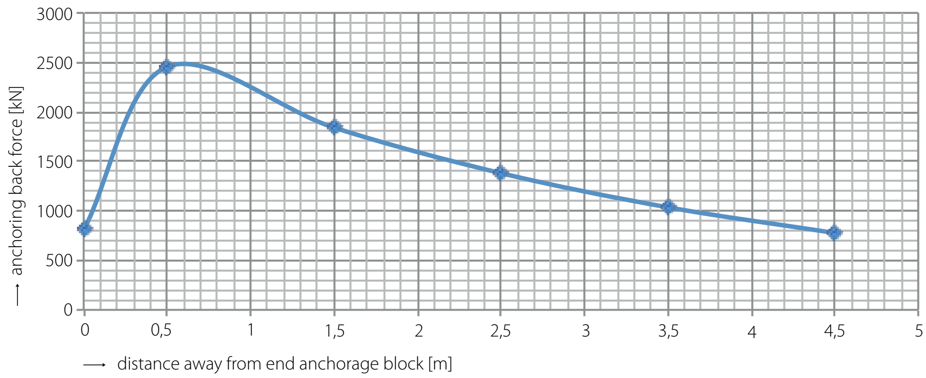
At the points where the tendons deflected, deviator blocks (photo 10) had to be constructed. These were reinforced concrete blocks C35/45 integrated into the new bottom flange. Based on the equilibrium of forces at the node represented by the deviator block, an upward force per tendon of approximately 600 kN at ULS had to be absorbed. In order to control the crack widths within this critical part, the reinforcement steel was designed with a stress limited to 300 N/mm^2.

Located at the extremities of the tendons the end anchorage blocks ‘nail’ the longitudinal post-tensioning forces to the structure (photo 11). Because of the magnitude of these forces and the limited capacity of the existing structure, both vertical as well as horizontal anchoring of the blocks was necessary. Given the height of the anchorage blocks and the confined space within the box girders, the project specifications required the use of self-compacting concrete. A strength class of C45/55 was prescribed to cope with the bursting and spalling forces of the post-tensioning. As the end anchorage blocks were located above the massive lock walls. This was an advantageous situation for the vertical anchoring of these blocks.

CFRP strengthening

As the longitudinal post-tensioning was not applied at the extremities of the structure, but through interior anchorages, a considerable “anchoring back” force had to be dealt with. Depending on the source (see reference VSL [2] and Stufib [4]), this force should be estimated at 1/4 to 1/3 of the post-tensioning force as an average. A finite element model of the anchorage and the surrounding structure showed that this force was in our case even close to 50% (peak value) of the post-tensioning force, with its maximum at 0.50 m behind the anchorage block and then decreasing by 20 to 25% per meter away from the end anchorage block (fig. 12).

10 Deviator block
11 End anchorage block (showing longitudinal and transverse post-tensioning; left and right corners CFRP partly visible)
12 Anchoring back force (inner webs, Diepenbeek/Hasselt)



As the existing structure proved to be insufficiently strong to cope with this post-tensioning force, the externally bonded CFRP sheets were applied. This reinforcement was attached to the girder webs behind the end anchorage block. The project specifications stated that CFRP sheets with an elastic modulus of 640 GPa, strain at break equal to 0.4% and a base weight of 400 g/m^2 had to be used. The ultra-high E-modulus was necessary to mobilize the required anchoring force. For the calculation, the CUR guideline 91 [3] was adopted, with the governing criterion being 'peeling off'. The bond strength of the concrete of the existing structure should be at least 1.8 N/mm^2 (verified by on site pull-off tests). If not, corrective measures were to be taken. The required number of CFRP sheets was dependent on the web and the location. The maximum was at the inner webs at Diepenbeek and Hasselt: starting with ten layers over the first 1.8 m, over six and three layers for the next 1 + 1 m and finally two layers for the last meter (assumed layer thickness is 0.235 mm).

Execution phase

Early 2012, the refurbishment and elevation started with the most upstream bridge, i.e. Diepenbeek. By the end of 2014, the last bridge (Olen) was elevated and refurbished. Together with the elevation, the opportunity was taken to refurbish the bridges in terms of concrete repairs, crack injections and enhancement of water disposal. To proof the elevation and refurbishment at each individual bridge, static load tests were performed. In Belgium, a static load test is a standard procedure for a final check of a newly built or strengthened bridges before it is (re) opened for traffic. Six lorries representing up to 85% of the design load, were placed around the mid span and the support to invoke the maximum positive and negative bending moments respectively. The elastic deflections were very small (less than 3 mm) and residual deflections (after unloading) were close to zero. As such, the load test results were satisfactory.

Conclusion

Based on a viable phased concept, the four lock bridges were elevated by 0.43 m to 0.65 m (depending on geographic location) and refurbished over a period of just under three years, without having to build a completely new structure and without compromising the stability of the existing lock structure at any time. Through the use of state of the art techniques, like post-tensioning and CFRP, the existing reinforced concrete structure could be upgraded to the required level of performance. ☒

REFERENCES

1 SCIA Engineer, www.scia.net.
2 Rogowsky, D. M., Marti, P. (1996). *Detailing for post-tensioning*. Bern, Switzerland: VSL International Ltd.
3 CUR Recommendation 91 (2nd revised edition) – Strengthening of reinforced concrete structures with externally bonded CFRP (2007). Gouda, The Netherlands: Stichting CURNET.
4 Stufib Report 9 – Strenghtening of existing bridges by external post-tensioning (2006). Nieuwegein, The Netherlands: Studievereniging fib-Nederland.
5 Circular letter 576-58, Regulations for load tests on bridges (24 April 1981). Brussels, Belgium: Ministerie van Openbare Werken van België.

Strengthening of 100 year old unreinforced concrete arch bridge

Kuhbrücke/ Hildesheim bridge

The Kuhbrücke/Hildesheim bridge is an unreinforced concrete arch bridge near the city of Hildesheim, dating from 1910. When recalculation showed that the bearing capacity was no longer sufficient, the bridge had to be strengthened.

The Kuhbrücke/Hildesheim bridge has an important function, because it is the single access to an agricultural area owned by the City of Hildesheim. Recalculation has shown that the bearing capacity of the bridge is not sufficient to carry ordinary agricultural machines and traffic loads had been limited to 3 tons maximum for vehicles. Furthermore the bridge and its equipment showed a lot of damage related to ageing.

The City of Hildesheim investigated several alternatives, e.g. building a new bridge at the same place, building a new bridge at an alternative place and strengthening of the existing bridge. Because of limited financial capacities, strengthening was the preferred solution. Structural engineering firm 'matrics engineering' was chosen to search for a technical solution that
- upgrades the bridge for load model 'Brückenklasse 30' according DIN1072 (1985);
- minimizes effort and costs for the structural measures;
- allows use of old bridge during harvest before strengthening is done.

The bridge was strengthened in 2016 to upgrade the capacity for carrying vehicles with maximum weight from 3 ton to 40 ton. The historic arch (fig. 3) and the foundations are further used. New webs were added by horizontal prestressing to the arch (fig. 4). A bar post-tensioning system (50 mm) was used

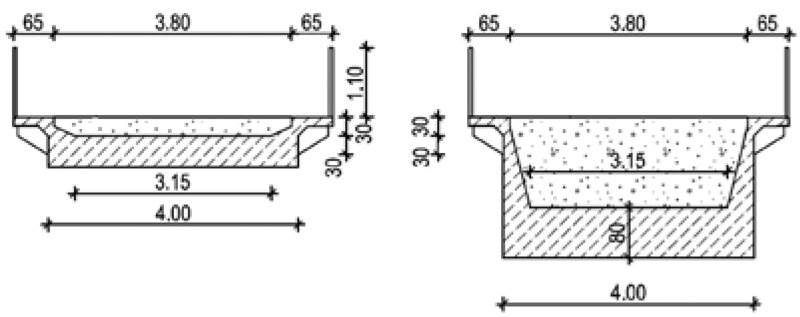
Hermann Weiher, Katrin Runtemund
matrics engineering GmbH
Andreas Praus
City of Hildesheim

with innovative and very durable Ultra High Performance Concrete (UHPC) anchor plates (Hybridanker). Finally a reinforced concrete deck slab was added to create a kind of box section. To reduce thermal stresses in integral bridges it is planned to develop a bridge deck cooling system in a future research project, using the Kuhbrücke/Hildesheim as trial project. To verify the efficiency of the cooling, many temperature sensors were placed. The operation was finished in June 2016.

2

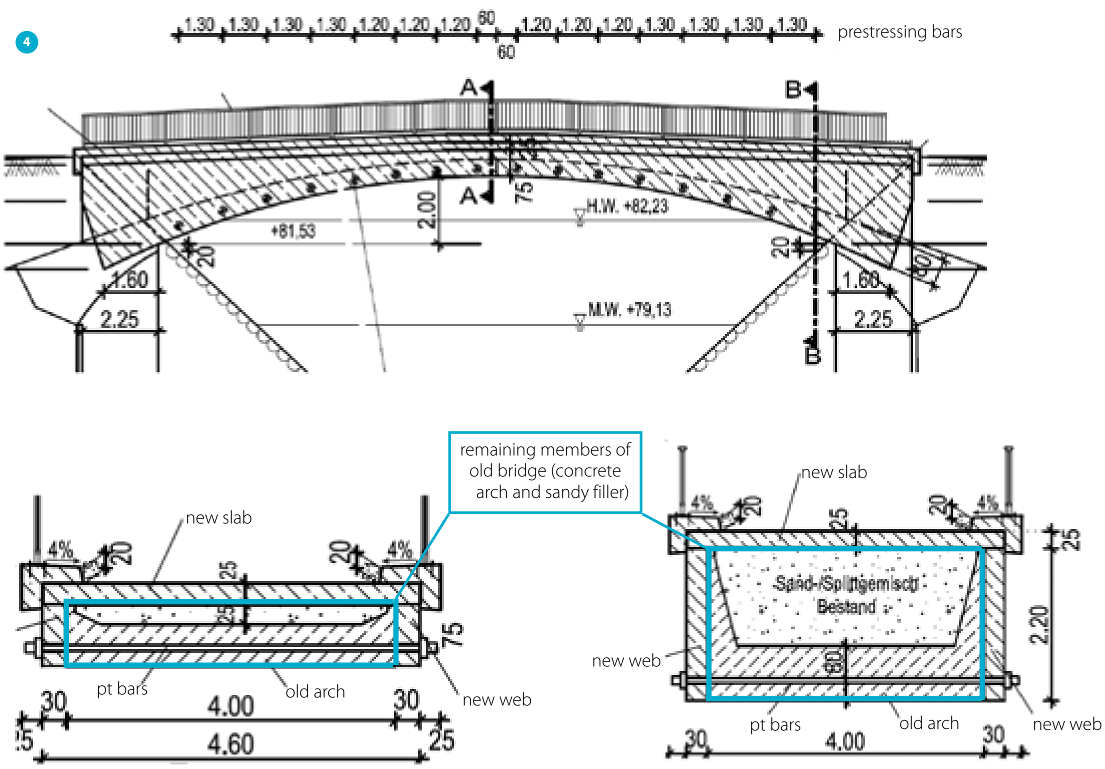
Kuhbrücke – 100 year old structure

The bridge was built as unreinforced concrete arch. The arch is continuous and supported by two massive abutments with a span of nearly 25 m. With a thickness of just 500 mm the bridge is very slender (1:50 ratio of midspan height to span length) and the arch very flat (1:10 ratio of rise of the arch to span). Although more than 100 years in service the bridge showed only minor deficiencies, e.g. a transversal crack of about 50 mm depth at the bottom side of the arch in its centre along total width. This might have come from overloading by traffic, temperature, shrinkage and horizontal movement of abutments. Concrete testing was done to determine the concrete strength. Class C16/20 according to Eurocode 2 (2011) was finally found. Based on that strength calculations



3

4



1 Strengthened bridge, finalized in June 2016
2 Kuhbrücke (1910) before strengthening
3 Sections of the original structure
4 Side view and sections of the strengthened structure

5a

5b

was done and traffic loading finally was limited to vehicle loads of maximum 3 tons. For the upcoming harvest in autumn 2015, when thousands of tons of sugar beet root were expected, an urgent solution was needed. If using this bridge with its limited capacity, only a solution with conveyor belt and small equally distributed loads was allowed. Finally the City of Hildesheim created a temporary access by concrete cylinders thrown into the river and filled up with earth.

Strengthening concept

The main structural deficiency of the arch bridge is the limited bending resistance of the arch both in longitudinal and transversal direction. It must be assumed that the sandy filler above the concrete arch does not act as resistance although it seems that it has some strength.

New cast in-situ webs with height from lower bound of the arch to the traffic lane level are added on both sides of the arch. For monolithically connection to the existing arch, the webs were cast against roughened surface and stressed together by 50 mm prestressing bars (some 1.5 MN for each bar stressing force). Time depending losses were very small because of the age of the existing arch. This prestressing force also solved the deficiencies in transversal direction. Finally a reinforced concrete deck slab was added to further help distributing the loads and to improve durability. The strengthened bridge still needs the high compression resistance of the arch and actively uses it by transferring the forces through the webs into the arch which acts like a bottom slab. Of course dead weight of the bridge is fully transferred by the old arch. The old arch and new members webs and slab act fully together, similar to a box girder/arch.

The bridge is far away from public road network and subsequently de-icing salts are not used. To keep costs small, no sealing was applied. To improve durability, the calculated crack width in the slab was limited to 0.2 mm instead of 0.3 mm.

The arch bridge has no hinges and hence very high stresses can occur due to temperature loading. To avoid massive reinforcing because of the high stiffness of the new box arch girder the concept was to allow cracking and limit the crack width to 0.2 mm in the webs. Further limitation of stresses due to restraint deformation is planned to achieve by actively cooling and heating the bridge deck (see chapter 'Tempering of bridge deck'.)

Construction

Construction works began in February 2016 with installation of scaffolding (photo 5a and 5b). The old bridge deck was completely rebuilt. Only the arch and the filling material

5 Erection of scaffolding
6 Drilling of tendon holes and detailing of webs (from left)

remained and could be used as formwork for the new members. The arch was bored horizontally in transversal direction at the length of 4 m to house the prestressing steel bars (photo 6a and 6b). Vertical and horizontal deflection of borings were very small. After reinforcing the webs and closing the formwork webs were poured with C30/37.

Deck slab was poured in second stage and monolithically connected to the new webs. For transversal prestressing of the arch a bar system of BBV Systems GmbH was applied according to ETA-16/0286 (2016) using Macalloy prestressing bars and 'Hybridanker'-anchorages.

It was the first time that Hybridanker-plates were applied for these prestressing bars. Hybridankers are anchorages made of ultra-high performance concrete (UHPC). Using these anchorages was beneficial from durability point of view (no steel parts exposed outside stainless steel cap, photo 8a and 8b) and also because of very small edge distances. This was proved by special tests which showed that, due to its stiffness (large thickness), when applied on concrete no extra confinement is needed (e.g. spiral).

6a

6b

7

The Hybridanker-plates for this project consisted of a force transfer unit made of ductile cast iron, confinement with rebar spiral and precast with Ultra High Strength Concrete with a compressive strength around 200 MPa. Further features were: grouting inlet, threads to connect the cap, trumpet made of polyethylene. The technology is still new; its first application was in 2011. See Weiher et al. (2012) for more details about general principles.

The construction was finished in June 2016 with a fully strengthened arch bridge (photo 1). The position of the arch is still visible by following the anchorages.

8

7 Reinforcement of deck slab and installation of prestressing bars
8 Hybridanker-plate for anchoring 50 mm prestressing bar
9 Temperature sensors (a) and plastic hoses (b)

9a

9b

Tempering of bridge deck

The City of Hildesheim was very open-minded for a planned research project. The restraint stresses due to temperature shall be limited by tempering the bridge deck. For that purpose plastic hoses were installed (photo 9a and 9b). The idea is to send tempered liquid in order to cool down in hot periods (e.g. summer) or heat up the concrete deck in cold periods (e.g. winter). By doing so, one may decrease stresses due to temperature significantly. The project shall be used as trial project and permanent use is not foreseen. Therefore, the design was done without considering the benefits of such a tempering. Even higher effects of tempering can be achieved by this method for large continuous girder bridges and integral bridges.

Conclusion

A very old concrete bridge built in 1910 was strengthened at little costs (< 30% of building a new bridge) to meet modern goals. For this purpose it was beneficial that the bridge was unreinforced and furthermore had a static system (arch) that offered hidden resistance.
The strengthening concept was chosen in such a way that the load on the bridge during construction was not large. Innovative aspects were applied, such as the high strength concrete anchorages of prestressing bars and the bridge tempering trial to limit stresses from temperature load. ☒

PROJECT DETAILS

client City of Hildesheim
contractor Hoch- und Industriebau Celle GmbH, Hambühren
PT-system BBV Systems GmbH, Bobenheim-Roxheim
design matrics engineering GmbH, München
tempering matrics engineering GmbH, München; tripleS GmbH, Mülheim an der Ruhr

REFERENCES

1 DIN 1072. (1985) Straßen- und Wegbrücken. Lastannahmen. Berlin,Germany: Beuth Verlag GmbH.
2 DIN EN 1992-1-1 Eurocode 2. (Jan 2011) Bemessung und Konstruktion von Stahlbeton- und Spannbetontragwerken – Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau.
3 ETA-16/0286. (2016) BBV 1030 post-tensioning bar tendon system, nominal diameter 32 to 50 mm. Bobenheim-Roxheim, Germany: BBV Systems GmbH.
4 Weiher, H., Tritschler, C., Glassl, M., & Hock, S. (2012): Hybridanker aus UHPC - Erstanwendung bei der Verstärkung der Rheinschleuse Iffezheim mit Dauerlitzenankern. *Beton- und Stahlbetonbau* Vol. 107, Nr. 4, Ernst & Sohn.

1

Bridge collision protection ramp Kampen

This paper describes the design and construction of a unique and innovative bridge protection structure: the 'collision protection ramp' that is placed in front of an existing bridge. This innovative concept combines the capacity of a ridged structure with the benefit of minimal ship damage and limited visual impact on the bridge.

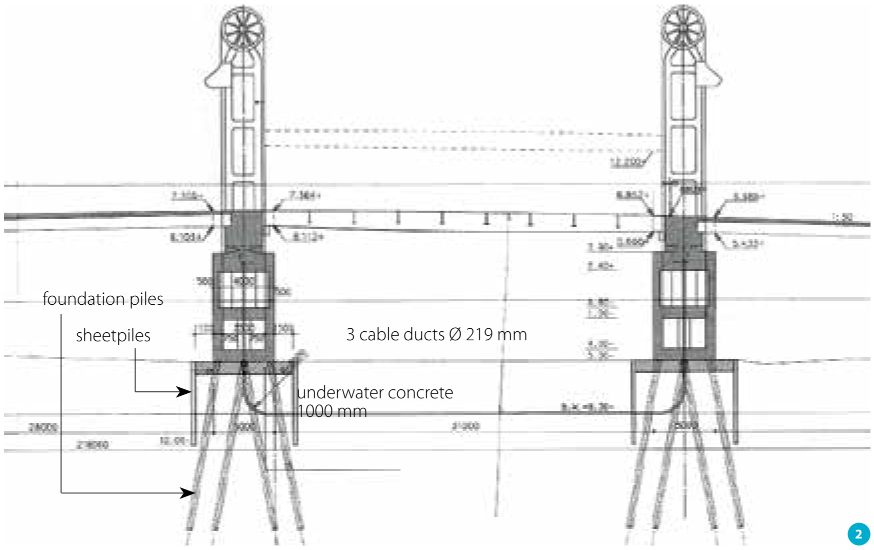
Preface

As part of the project 'Ruimte voor de rivier IJsseldelta' the river IJssel near the city of Kampen is dredged to a depth of NAP -7,20 m (existing riverbed level is NAP -4,40 m). The bridge and its guiding structure are not able to withstand the loads of a ship collision after the dredging works [1].

Consequently Rijkswaterstaat and the city of Kampen decided to protect the bridge with two collision protection structures on the upstream (south) side of the bridge and three on the downstream side. The difference in the numbers is explained by the fact that ships pass the bridge only through the main channel when sailing downstream and can pass in two shipping lanes when sailing upstream.
The contract of the project was awarded to 'Isaladelta', a joint

ir. Alex van Schie
Volker InfraDesign bv

1 City bridge of Kampen, installation of the sections with a sheerleg
credits: Wagenborg
2 As built drawing city bridge
3 Collision protection structure



2

venture of Boskalis and VolkerWessels. The design of the protection structures was made by Volker InfraDesign and the construction and installation was done by Van Hattum en Blankevoort.

Requirements and boundary conditions

The main requirements for the design of the five collision protection structures were:

- the structures need to protect the bridge from the frontal impact of a sailing ship of the CEMT Va class at all water levels between low water (NAP -0,35 m) and high water (NAP +1,80 m); the energy of a sailing ship on the upstream side of the bridge is 55 MNm; the energy of a sailing ship on the downstream side of the bridge is 21,6 MNm;
- the structures need to withstand the side impact of a ship colliding at an angle of 10° calculated according to the Dutch design code Richtlijnen Ontwerp Kunstwerken (ROK). The structures are able to withstand the loads of the above side impact at all levels between 4 m below low water and 1 m above high water;
- the structures need to be separated from the bridge, with a maximum distance of 16 m;
- the structures need to be designed as ramps.

Important boundary conditions for the design and construction were:

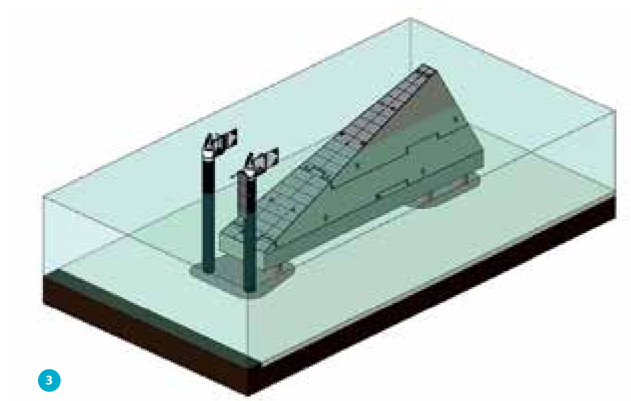
- passing ships (safety during construction);
- river discharge (governing for foundation installation and diving works);
- bridge structure (fig. 2);
- scour protection (had to be removed and replaced).

Construction sequence

After studying the requirements and local boundary conditions, different alternative types of structures and construction methods were investigated and compared in a Multi-Criteria Analysis. The structure that was selected is a prefabricated concrete structure that consists of three parts placed under water on top of six steel foundation piles that are installed at NAP -6 m, just above the dredged riverbed (fig. 3 and 4).

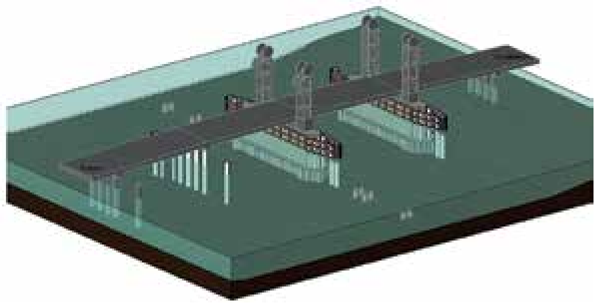
Design

One of the most difficult parts of the design of the collision protection ramps was the calculation of the magnitude of the horizontal and vertical loads on the structure [2]: a colliding ship hits the ramp structure (fig. 5) with a high level of kinetic energy and the front of the ship starts to travel along the surface of the protection structure (the rear of the ship will sink deeper in the water). As a consequence a part of the kinetic energy is converted into potential energy and another part is lost as friction (one of the design requirements was to neglect the energy that is lost by deformation of the ship hull). During this conversion the forces on the structure increase and reach a maximum just before the location where, and the moment when, the ship stops. Figures 6, 7 and 8 illustrate the relation between the forces and the conversion of the kinetic energy when a ships sails against the ramp. Important variables in the equations are the angle of the ramp and the coefficient of friction. These have been varied and the design is based on 20°. A steeper slope gives higher horizontal loads (bigger piles) and a more gentle slope gives a longer concrete structure. For the upstream ramp the energy analysis results in loads of 12 270 kN vertical and 9140 kN horizontal caused by the collision. The loads caused by the side impact are 2510 kN perpendicular and 1255 kN parallel. For the loads and dimensions of the downstream ramps one could say that these are a factor 3 smaller.

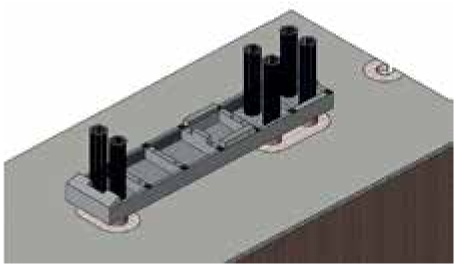


3

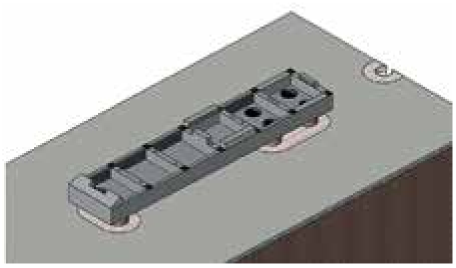
4



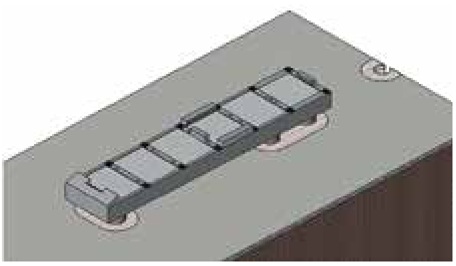
(a) installation of steel foundation piles



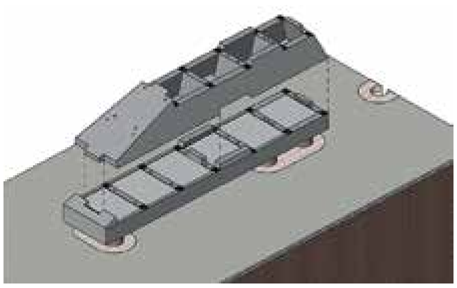
(b) installation of bottom section with rebar cages



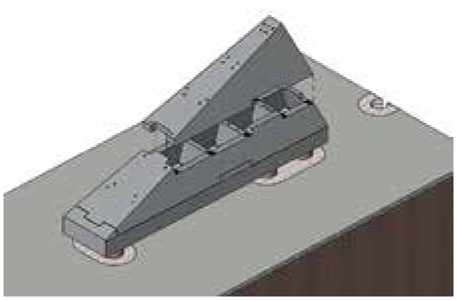
(c) lowering of rebar cages into piles



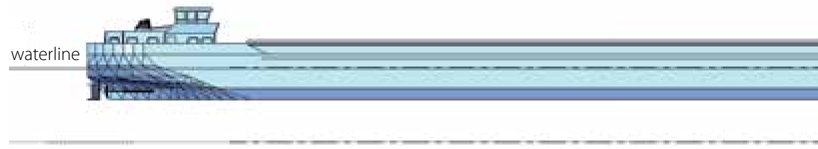
(d) pouring underwater concrete



(e) installation middle section



(f) installation top section



5

The structural design of the bridge collision protection ramps [3] resulted in:

- five foundations of six steel piles Ø 1626 mm x 20 mm of 20 m S355J2;
- two upstream protection ramps of ($l \times w \times h$) 24,80 × 5,00 × 9,40 m^3 and three downstream - protection ramps of 19,70 × 5,00 × 7,80 m^3 concrete C30/37;
- a thickness of the bottom sections of 1450 mm (450 mm prefab + 1000 mm underwater concrete);
- a thickness of the external walls of 600 mm and 500 mm for the internal walls;
- thirty pile connections with rebar cages of 5900 mm length.

The most critical part of the structural design were the connections between the three parts and the load transfer to the foundation piles (see chapter 'details'). Also the fact that the lifting capacity of the sheerleg capable of sailing to the site was limited to 300 tonnes was a challenge during the design (the middle part came close to 285 tonnes).

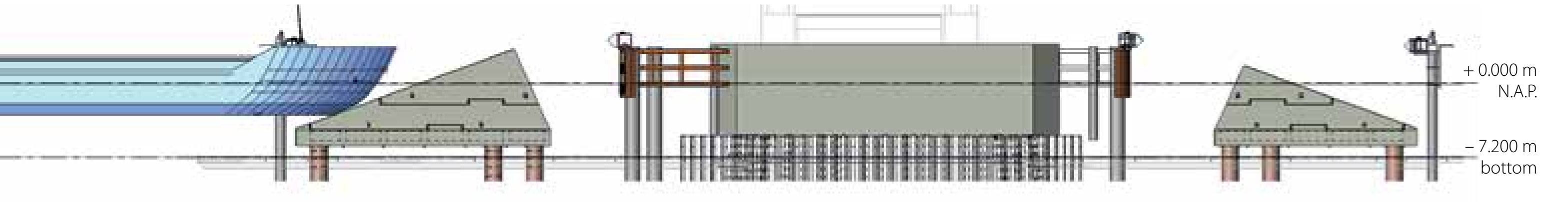
Interfaces

Although deformation of the ship hull is neglected in the energy equation, it is still present in practice. An analysis of the ship damage after a frontal collision on the ramp was made by Marin [4]. Their conclusion is that damage of an unloaded ship is less than that of a loaded ship and therefore the ramps work better for unloaded ships. For loaded ship the ramps perform more like a conventional protection barrier and will damage the ships hull; the damage is limited to the front 10 m (fig. 9) and will therefore not affect the cargo.

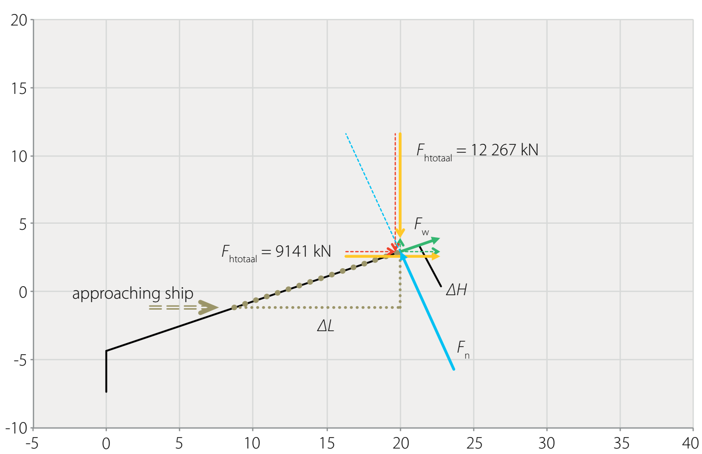
Details

The fact that all structural connections had to be made under water (by divers) asked for a set of details that had to be developed specifically for the project. In particular the two details in figure 11 took a lot of engineering before they could be finalised. The lifting points of the bottom section were made of cast in pad-eyes and the lifting points for the middle and top section were openings (panama chock) in the walls. The lifting points have no parts sticking out, to keep the surface of the ramp smooth. The disconnection of the lifting points could be done without divers.

The three concrete parts are coupled by tension bars that are put in vertical ducts in the structure. The bars and bolts fit into recesses in the roof, to keep the surface smooth. Under the bottom section bolts are connected and fastened by divers. The



6



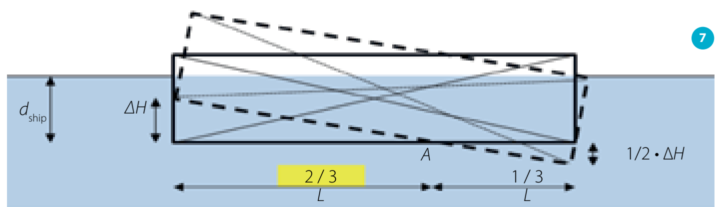
$$E_{\text{kin}} = E_{\text{pot}} + E_{\text{friction}}$$

$$E_{\text{kin}} = \frac{m_{\text{ship}} \cdot g \cdot \Delta H^2}{8\, d_{\text{ship}}} + \frac{m_{\text{ship}} \cdot g \cdot f \cdot \Delta H^2}{4\, d_{\text{ship}} \sin 2\alpha} = \frac{m_{\text{ship}} \cdot g \cdot \Delta H^2}{8\, d_{\text{ship}}} + \frac{m_{\text{ship}} \cdot g \cdot 2f \cdot \Delta H^2}{8\, d_{\text{ship}} \sin 2\alpha} =$$

$$E_{\text{kin}} = \left(1 + \frac{2f}{\sin 2\alpha}\right) \cdot \frac{m_{\text{ship}} \cdot g \cdot \Delta H^2}{8\, d_{\text{ship}}}$$

It follows that:

$$\Delta H^2 = \frac{8\, d_{\text{ship}} \cdot E_{\text{kin}}}{m_{\text{ship}} \cdot g \left(1 + \dfrac{2f}{\sin 2\alpha}\right)}$$

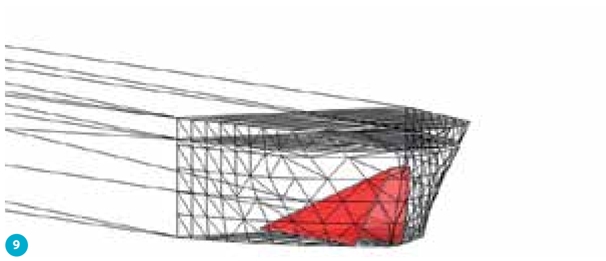


7

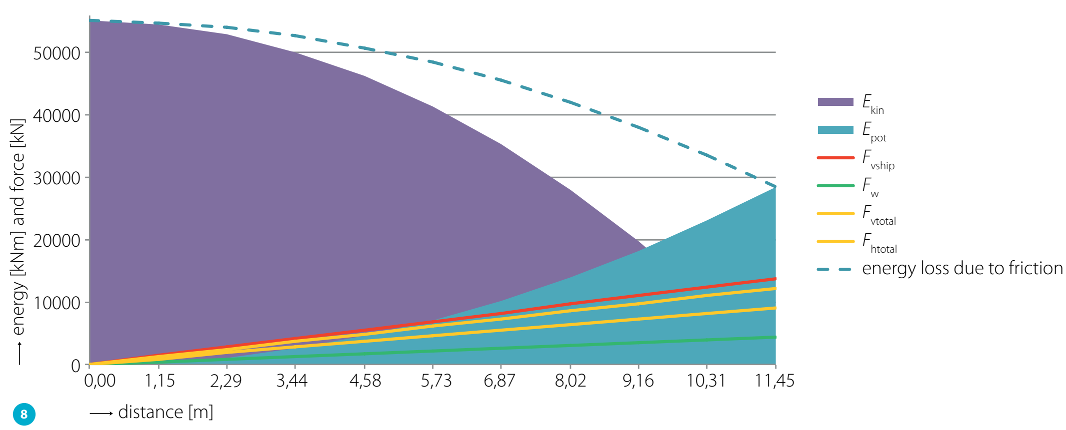
fact that the openings in the wall are permanent and all tension bars can be removed has the advantage that the two top elements can be lifted of the bottom element after a collision (for repair).

Construction

The thirty foundation piles were installed in November 2015 from a pontoon using both a vibrator and a hydraulic hammer. The installation tolerances for the piles, that were determined in collaboration with the contractor were very strict (x, y +/- 50 mm and z +/- 5 mm). The work sequence and schedule allowed the contractor to adjust the position of the openings for the rebar cages in the bottom section to the as built location of the piles. The concrete sections were casted on a quay in the Zuiderzee-haven in Kampen, relatively close to the bridge. As all sections had to be lifted by the floating sheerleg Triton they all had to be casted close to the waterway. To save room along the quay the middle and top section of the five ramps were cast on top of each other. This casting method also made sure the parts would fit smoothly on top of each other when placed in the river. Between December 2015 and May 2016 the fifteen sections were casted.



9



8

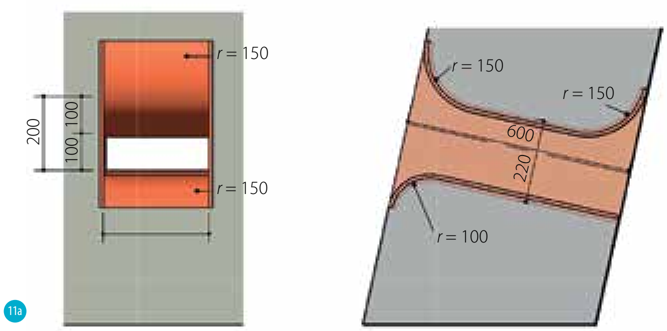
4 Construction sequence
5 Relation between the forces and the conversion of the kinetic energy when a ship sails against the ramp
6 Forces on Collision Protection Ramp
7 Parameters in energy Equation
8 Energy conversion
9 Calculated ship damage after collision

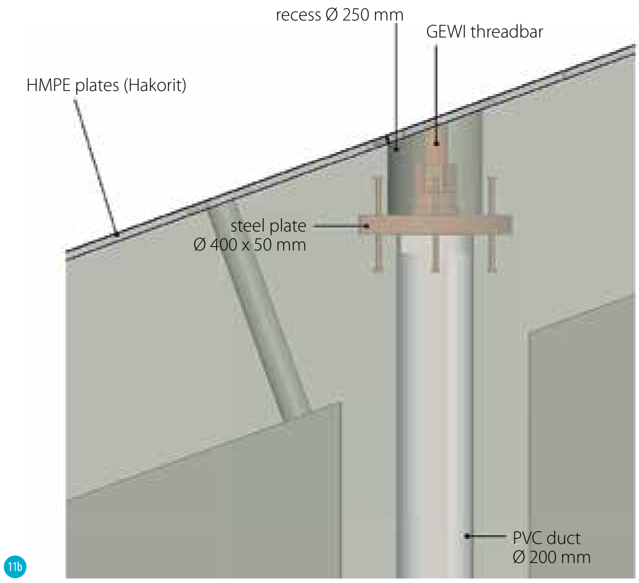
10 City bridge Kampen with the five protection ramps
credits: Maarten van de Biezen

11 Details: (a) lifting point; (b) upper part of tension rod

The installation and connection of the sections was planned in two phases: first the bottom sections were placed and poured with underwater concrete and afterwards the middle and top sections were installed and connected (fig. 1). During the works a delay occurred because of the high river currents in July 2016. Finally all five ramps were placed in August 2016.

After completion of the structures seven steel piles with navigation signs and lights were placed in front of the bridge collision protection ramps (photo 10). The lights are powered by solar panels and battery.





Conclusion

The bridge collision protection ramp is a relatively small structure with the capacity to transfer very large amounts of ship energy to a pile foundation without effect on the structure it is protecting. For the first five structures that are placed in front of the piers of the city bridge in Kampen all technical challenges have been overcome. With this experience it is just a matter of time before other objects in rivers will be protected by ramps like these. ☒

REFERENCES

1 Zomerbedverlaging Beneden-IJssel Deelrapport 1B Inventarisatie Constructies en Kunstwerken. LW-AF20120608/RK versie 4.0, RoyalHaskoningDHV, 2013.
2 Bergwerf, D. (2015). Addendum Ontwerpbasis Civiel Schanscaissons, IJD-SBK-RAP-1004 revisie 2.0.
3 Bergwerf, D. (2015). Berekeningsrapport DO Schanscaissons, IJD-SBK-RAP-2007 revisie 1.0.
4 Marin (2013), Review van Schanscaissons voor Stadsbrug Kampen. 27106-001-HSS.
5 Schoffelmeer, M. (2015). Geotechnisch ontwerp DO Schanscaissons, IJD-SBK-RAP-2006 revisie 1.0.
6 Schoffelmeer, M. (2015). Beoordeling invloed zomerbedverlaging op Stadsbrug Kampen, IJD-OIJ-RAP-2005 revisie 1.0.

Almonte River Viaduct

1 Viaduct over River Almonte in Spain almost finished

High performance self-compacting concrete arch bridge for HSR-line Madrid – Lisbon

The high speed railway line Madrid-Lisbon crosses over River Almonte with a great 384 meters arch made of high performance self-compacting concrete (C80). The construction of the viaduct, with a total length of 996 m, started in April 2011, and its loading tests were undertaken late 2016.

Guillermo Capellán, Javier Martínez, Emilio Merino
Arenas & Asociados, Ingeniería de Diseño
Pascual García-Arias
Idom

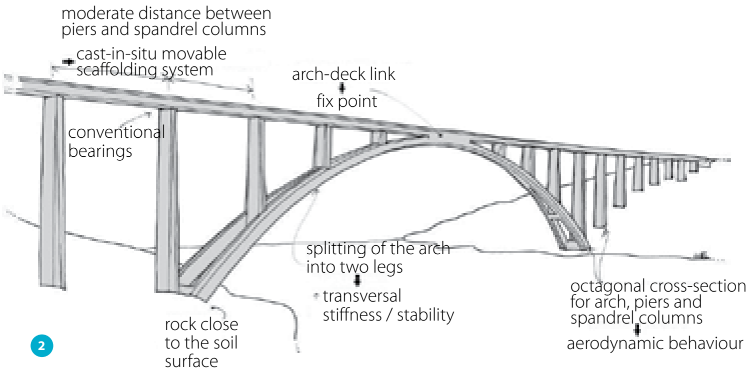
During the initial stages of the project, different structural alternatives for the Almonte River Viaduct were analyzed in a detailed typological study, considering simultaneously its final behavior as well as its erection procedure. Some of these alternatives included cable-stayed, frame type and variable depth truss deck options. The multi-criteria analysis highlighted the concrete arch solution as the most economical, the best in terms of durability and maintenance conditions, and the one that guaranteed a better structural behavior against dynamic trainloads and wind.

The exceptional span of the arch and the specific considerations of high speed rail (HSR) bridges (dynamic effects by passing trains significantly larger than road traffic, significant horizontal loads and fatigue) led to the design of an innovative structural scheme in HSR arch viaducts, using two separate hexagonal sections in the arch springing that join into an octagonal one in the central stretch of the arch (fig. 2).

In order to verify the structural behavior for static and dynamic loads (deflections, accelerations), specific verifications and advanced nonlinear structural models, for every stage of the construction, were carried out. The stability of the arch was verified for all the critical loading combinations, making a geometric-and-material nonlinear analysis, and using step-by-step iterative techniques.

The complex erection procedure required unique and specific auxiliary members during construction. The arch was erected by the cantilever construction method with the aid of temporary

2 Conceptual sketch of the designed bridge
3 Arch (a) and deck (b), typical cross section
4 Cantilever construction of the arch



cable-stays from two temporary steel towers, using form travelers specially designed for this bridge; while the deck was constructed using an overhead movable scaffolding system (commonly used method in Spain for HSR concrete box-girder viaducts).

A detailed analysis of all construction phases was performed together by designers (DJV Arenas&Asociados-IDOM) and contractor's technical services (CJV FCC-Conduril) during the construction-stage, and a complete monitoring program was developed to control every step of the building process.

Construction procedure

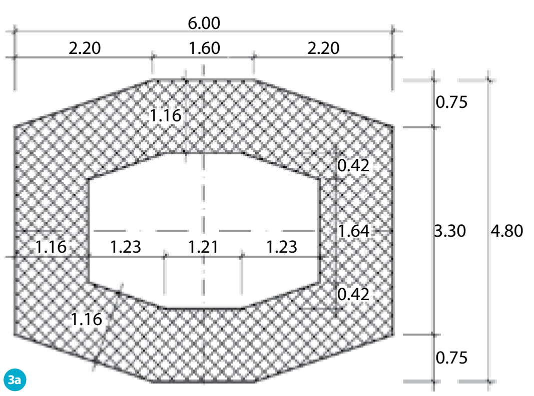
The construction procedure was developed such that the impact and hinder on the Reservoir of Alcántara is minimized. The main arch crosses over it at a height of almost 60 m, with two access viaducts at both ends completing this scheme. These access viaducts, with moderate typical spans of 45 m, have the same deck geometry over the arch (fig. 3b), in order to use the same overslung movable scaffolding system.

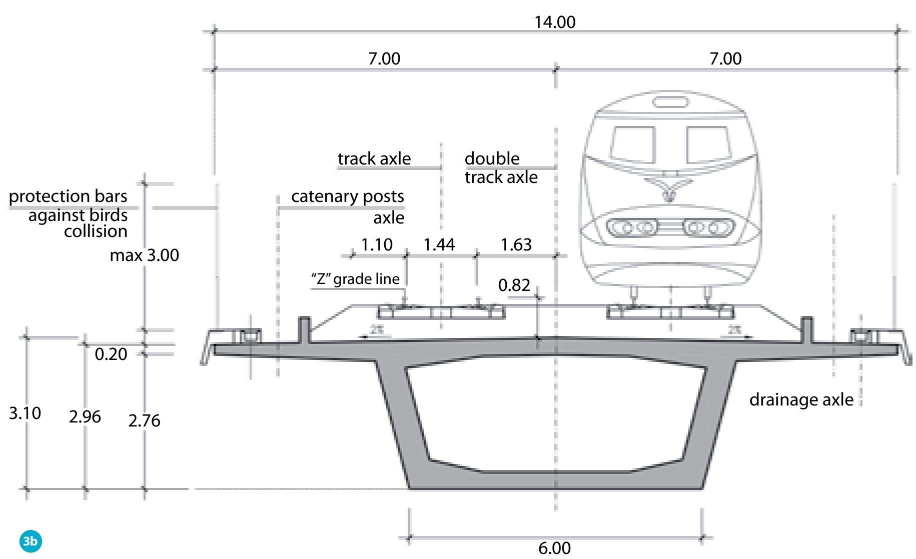
The arch is erected with a cable-stayed cantilever method (photo 4). The total length of the arch is divided into 33 cast in-situ segments on each half, with an approximate length of 6.70 m, plus the key central segment. A cantilever formwork traveler that fits to all geometries of the arch allows its concreting segment to segment.

In order to ensure that stresses are lower than allowable and the optimal geometry is maintained, the segments are supported by stayed cables during the construction.
There are 26 pairs of stay cables on each side with their corresponding back stays. Cables 1 to 8 (shown in yellow on fig. 5) are anchored to the piers rising on both riverbanks.
The anchorages for cables 9 to 26 (blue on fig. 5) are set on the temporary steel pylons built over piers P6 and P15. Cable forces are adjusted during the construction whilst some cables at intermediate construction stages are being released for avoiding excessive stresses.

After the arch closure is reached, cranes and temporary towers are dismantled to continue with the spandrel columns erection. Subsequently, the last 42 m deck spans over the arch are concreted with the same standard movable scaffolding system (fig. 6 and photo 1).

Other special construction features also deserve to be mentioned:
- Arch's foundations:
  The arch abutments are two reinforced concrete blocks of 7400 and 6300 m³ that spread the compression loads to the bed rock. The rock around the blocks is heavily injected with 255 tons of cement in order to fill all cracks and discontinuities.
- *Retaining foundations:*
  The global equilibrium of the 192 m half-arch cantilevered structure is achieved with multiple anchors placed at the retaining foundations adjacent to the riverbank piers. These anchors have a length of between 22 and 26 m, with a prestressing load of 2000 kN.

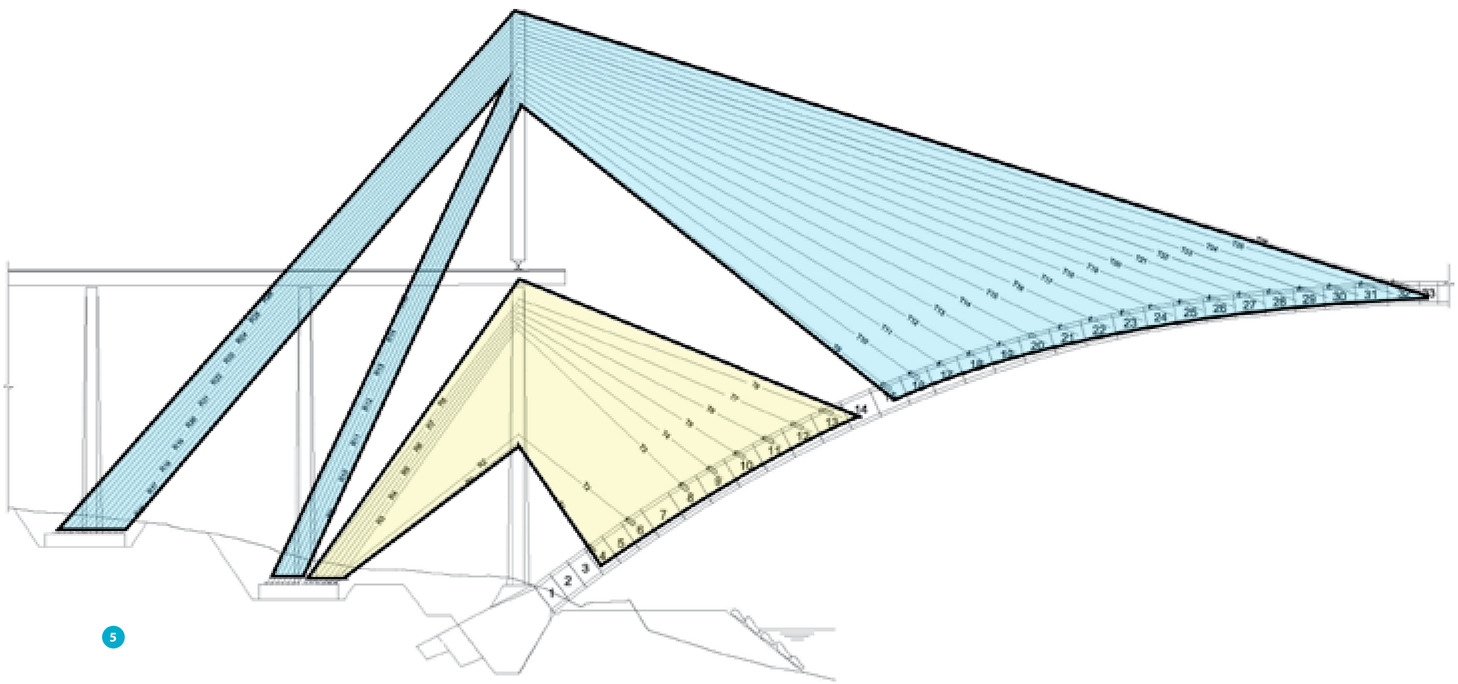




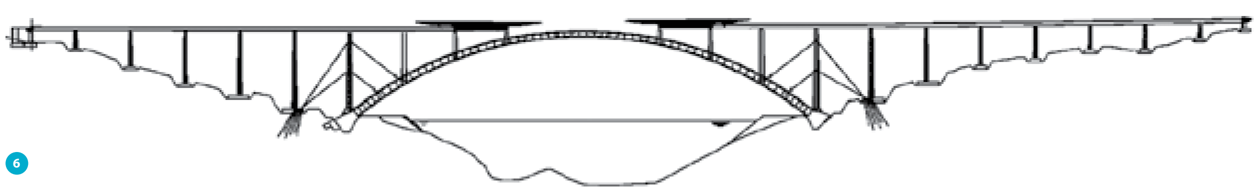
5 Numbered scheme of the arch segments and temporary stay cables
6 Last construction stages of the deck
7, 8 Cantilever formwork traveler

- *Cantilever formwork traveler*:
  The concreting of the arch is made segment by segment with a cantilever formwork traveler (photo 7 and 8) that fits to all geometries of arch: from segment 1 to 15 the arch is two legged and from 16 to 33 is only one piece varying in width and depth.
- *Temporary towers*:
  The articulated temporary steel towers were placed on the arch's edge piers. A rotation operation was undertaken in order to raise both towers from their horizontal position over the deck (fig. 9 and photo 10). This procedure was composed of four different erection stages:
  1. hinge placement;
  2. tower assembly and auxiliary members' installation;
  3. rotating operation;
  4. disassembly of auxiliary members. This system allowed execution time savings.
- *Temporary stay cables*:
  The stay cables are individually-protected multi-strand cables, identical to permanent stay cables (steel type Y 1860 S7; 150 mm^2 section). The number of strands varies from Ø 15 to 53 (15.2 mm each). The strands are not galvanized as enough corrosion protection is achieved by a semi-bonded individual HDPE sheath extruded into the strand after the interstices were filled with wax.
  Usually both ends of the stay cable were articulated in vertical direction in order to facilitate their installation.

4



5



6

7

Monitoring of the bridge

The erection of a bridge with such particular construction features requires permanent structural monitoring, starting during its execution and continuing throughout its entire service life. For a perfectly controlled and functioning structure, it is essential to know the behaviour of the different sections, which will enable monitoring of its service life conditions. For this reason a full scale measurement program was implemented. Staff was organized at three levels that influence each other and interact continuously:

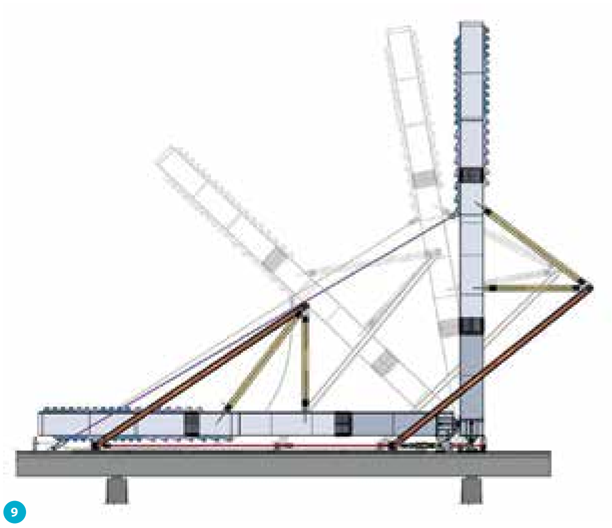
1. A surveyor company records, maintains and presents the data showing the behavior of the structure.
2. A primary analysis makes an immediate coherence evaluation with theoretical predictions providing them to the bridge designer, and simultaneously assesses the perfection of records.
3. At this level, the total station survey is compiled with an automatic data-acquisition measurement system.
4. This primary analysis is developed by an independent engineer, different to the staff of levels one to three.
5. A secondary analysis evaluates in depth the correlation with theoretical predictions and makes corrections to model calculations in order to improve the accuracy of forecast and appraise the origin and consequences of divergences. The installed system initially included 93 points of recording in each side of the bridge, listed in table 1.

Table 1 Recorded parameters of Almonte Bridge (only one semi-arch)

N°	parameter	points of recording
1	wind direction	1
2	wind speed	1
3	external air temperature	1
4	internal arch air temperature	1
5	stay cable temperature	6
6	concrete arch temperature	12
7	concrete pylon temperature	4
8	steel tower temperature	4
9	foundation clinometer	4
10	concrete pylon clinometer	1
11	steel tower clinometer	2
12	arch clinometer	2
13	concrete pylon clinometer	1
14	arch rebar strain gauge	12
15	steel tower strain gauge	4
16	stay cable strain gauge	40

8

9, 10 Elevation of temporary steel tower
11 Geometry control points of cantilever formwork traveler



9

10

The system was further improved with the addition of 5 accelerometers on the arch to analyze the dynamic behavior with the following purposes:

1. Continuous measurement and recording of the vertical and horizontal accelerations;
2. Empirical evaluation of vibration modes of arch in construction stages;
3. Evolution of vibration modes of the bridge in time.

Data was automatically transformed to the engineering units on site, and presented via website to the three supervision levels.

Geometry control of arch construction

For an optimal structural performance, the arch's geometry should match the best as possible with the geometric thrust line axis for all load combinations. It is concluded that the best practice and construction philosophy, is to achieve structure's overall geometric control, by performing field survey work and erection operations (forces of stays and cantilever formwork placement) to a meticulous degree of accuracy.

In this sense, it was necessary to carry out continuous and comprehensive studies of the structure under each erection stage, determining the corresponding stress and geometric data, preparing a step-by-step erection procedure plan and incorporating any checked measurement that was desirable.

Under certain construction load conditions (wind, temperature, gravity loads), it was necessary to check the structural integrity of arch, stays, piers and foundations.

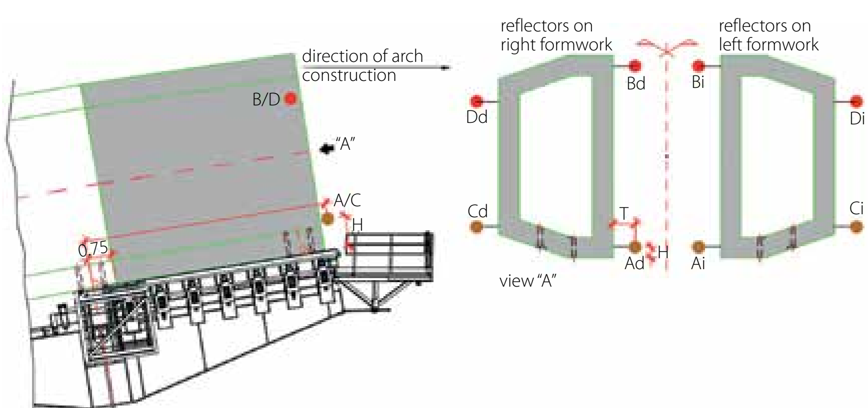
The placement of cantilever formwork travelers was controlled by four reflectors fixed at the end of the formwork (fig. 11). It was then possible to determine the position of the end of the cantilever arch at any stage and compare it with theoretical calculations. This way the structure could be controlled along its whole length and at any time during erection.

Since movements of the different stages grew and the system became more flexible, and therefore more susceptible to other effects (e.g. temperature on the stay cables), alarms were defined in sections of segments. The allowed tolerances were: segments 1 to 15 (±50 mm), segments 16 to 13 (±100 mm) and segments 24 to 32 (±180 mm).

It should be noted that the final construction errors were never greater than 88 mm.

Conclusions

The use of vanguard current technology and construction techniques, has allowed the execution of this engineering challenge. Among them all, it must be highlighted the high performance concrete allowing to adopt a more slender arch section, the four legged arch configuration, the nonlinear and evolving calculation software and techniques, the aeroelastic wind tunnel modelling, and the semi-probabilistic normative treatments, as key elements for the design and structural validation of the Viaduct over River Almonte. ☒



11

1

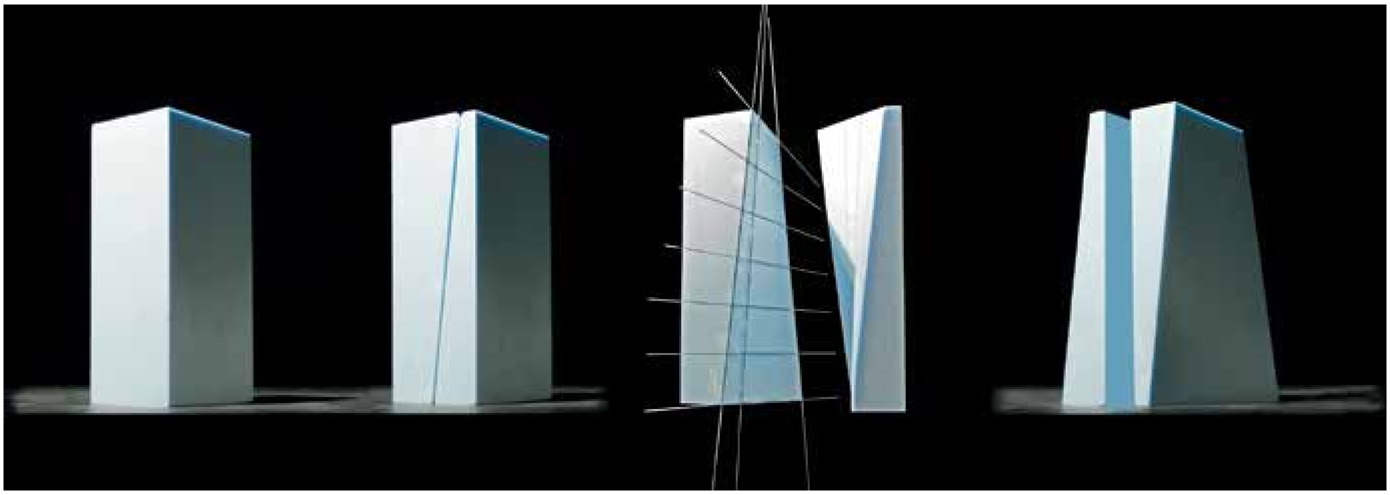
Dynamic and exciting structure with twisted towers connected by diagonals

The European Central Bank

The new building for the European Central Bank (ECB) (fig. 1) consists of two peculiar towers connected by diagonals. It is located in the east of Frankfurt, between the historic wholesale market, the Großmarkthalle and the Main River. The Großmarkthalle, an impressive brick building and monument from 1928 designed by Martin Elsaesser, was restored as part of its integration within the construction of the new ECB towers. Due to the twist of the towers, also torsion effects had to be considered. The locations of the diagonal elements were generated parametrically in an evolutional process.

Alexander Berger,
Manfred Grohmann,
Klaus Bollinger
B+G Ingenieure, Bollinger und Grohmann GmbH

1 The Großmarkthalle and the ECB towers
credits: Paul Raftery
2 Design of the towers
credits: Coop Himmelb(l)au
3 Element of the structural system: the cores
credits fig. 3 t.m. 10: Bollinger + Grohmann
4 Horizontal platforms and bridges and diagonal struts



2

3

4

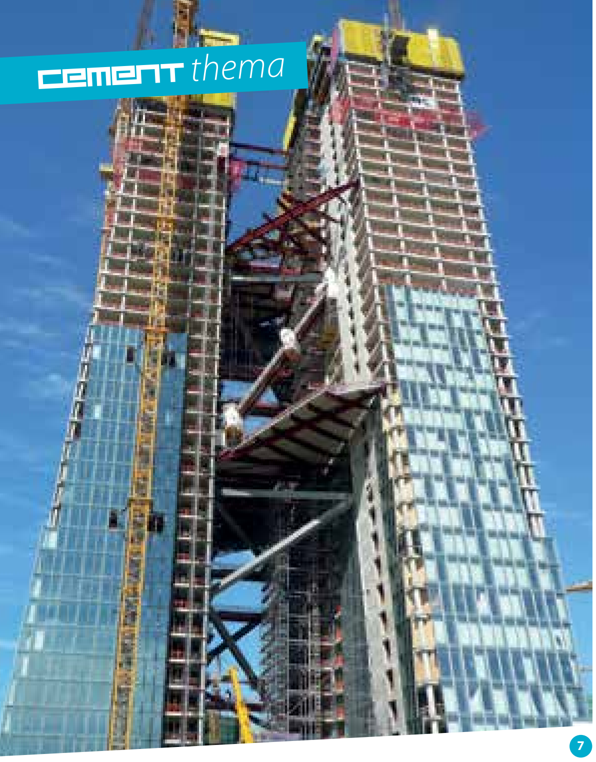
The architectural design process

The design of the new building for the European Central Bank was developed during a competition in 2002. The geometry of the striking 180 m high double towers follow the 'deconstructivistic' idea of the architects Coop Himmelb(l)au to divide a block diagonally, then turn the inner faces outside and the bottom to the top (fig. 2). The two parts are combined into one unit by a full height glass atrium.

The global structure design process

The stability system is based on the core walls of each tower. In longitudinal direction the stiffness of these cores (fig. 3) was sufficient to resist lateral loads. In cross direction the core width is only 6 m. Thus the structural systems of each tower had to be extended from two single cores to a global system consisting of the two cores and a spatial truss system which connects the cores via the atrium (fig. 4). The cores act as

[1]) *Deconstructivism is a movement of postmodern architecture which appeared in the 1980s. It gives the impression of the fragmentation of the constructed building and is characterized by an absence of harmony, continuity, or symmetry (Wikipedia).*



7

8

5 Slab geometry
6 Parametric design of the truss system
7 Construction stage: while the cores are nearly finished, the truss system follows up from approx. 60 m below
8 Installation of the trusses
9 Reinforcement of the foundation slab
10 Bracing, bridges and platform above
11 The finished structure inside the Atrium

credits: Paul Raftery

5

upper and lower chord of a global truss system. While the location of the platforms, elevators and bridges in the atrium were geometrically fixed, all diagonal elements were generated in a parametric and evolutional process. The first models show certain design steps of the process, the last one is the final one (fig. 6). The governing criteria in this process were the eigenfrequency and the stiffness.

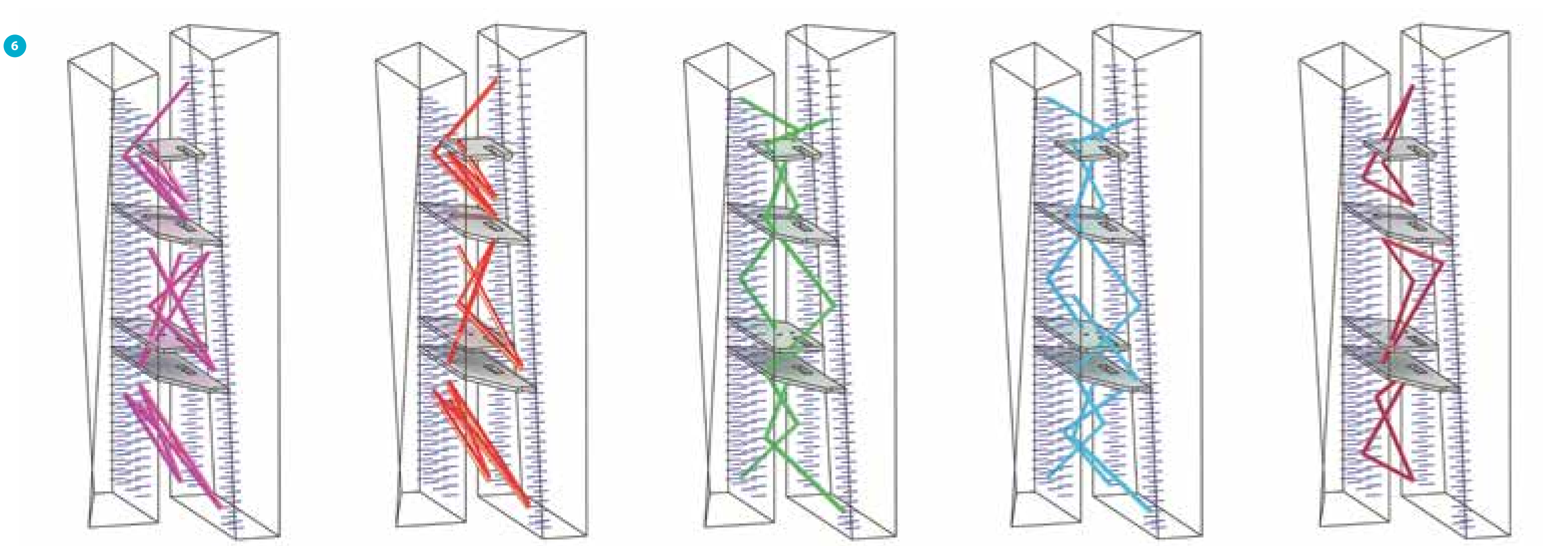
Torsional effects

Due to the inclined façade torsional effects have an impact on the towers. As both towers twist in the same direction, the global structural system, apart from wind loads, has also to resist these loads.

Slabs and columns

Due to the twist, all slabs have a different geometry and different span (fig. 5). The columns near to the façade follow the incline, while inner columns are vertical and straight.

6



9

Deformations and Pre-camber

The vertical and horizontal deformations of each column and slab were calculated considering the construction process as well as time-dependent factors like creep and shrinkage. These deformations were considered during the construction process by pre-cambering of these elements.

Construction process

During construction the structural system changed from pure core systems to partially finished truss systems (fig. 7, 8 and 10). For each construction stage the resistance of the structure had to be proved.

The foundation

All the loads of the towers are transferred into the ground by a combined pile-raft foundation (fig. 9).

Conclusion

With the construction of the new European Central Bank in Frankfurt a highly dynamic and exciting structure was realized. The design of the architects Coop Himmelb(l)au has been accompanied by the B+G structural engineers from the very beginning, i.e. already during the competition. Thereby, an optimized structure for this building has been developed. Through its expressiveness, it emphasizes and makes the seat of ECB architecturally appealing and standing out from afar (photo 1). ⊠

10

11

REFERENCES

1 Brensing, C., Bollinger, K., Grohmann, M., & Stracke, M. (2015). Architektur und Tragwerk in symbiotischer Beziehung - Der Neubau der europäischen Zentralbank in Frankfurt. *Ingenieurbaukunst* 2016, Berlin: Ernst & Sohn.

2 Santifaller, E. (2013). Symbolic metamorphoses – The European Central Bank building. *Detail Engineering 3*, (1st edition), pp. 42-51.

3 König, G., Liphardt,S. (2003). Hochhäuser aus Stahlbeton. *Betonkalender* I, Berlin: Ernst & Sohn, pp. 1-66.

4 Ruscheweyh, H. (1982). *Dynamische Windwirkung an Bauwerken*. Teil 1: Grundlagen der Anwendung. Teil 2: Praktische Anwendung. Wiesbaden: Bauverlag.

5 Krebs, A., Constantinescu, D. (1999). Der Einfluss der Axialverformung der vertikalen Tragglieder von Hochhäusern. Tagungsband Darmstädter Hochbau-Seminar, Hochhäuser.

6 Eurocode 2, Design of concrete structures - Part 1-1: General rules and rules for buildings. German version EN 1992-1-1:2004 + AC:2010.

Campus Tower in HafenCity Hamburg

On plot 80 in the Baakenhafen harbour basin in the east of HafenCity Hamburg, Garbe Immobilien-Projekte GmbH is developing the 'Campus Tower'. The construction site of approximately 3800 m² directly overlooks the Elbe and is located at the mouth of the Baakenhafen and the exit of the HafenCity Universität station of the U4 U-Bahn line. The complex of buildings mainly consists of buildings for office and residential use above a shared two-level underground car park. The gross floor area (GFA) of the planned development is approximately 22 120 m².

The 16-storey tower building with a total height of 56 m and the adjacent 7-storey office block (ground floor including gallery level + six upper floors) has been designed by Delugan Meissl Associated Architects of Austria (fig. 2). The striking triangular layout of the high-rise has a clear structure and a clear grid-like glass façade.

In the south-side building, designed by the SOP architect firm from Düsseldorf, subsidised rental apartments and freehold apartments will be built directly by the water's edge. Large windows and continuous south-facing balcony strips create high-quality living conditions.

Both buildings will meet the highest sustainability standards and will achieve the Gold standard for the HafenCity environmental certificate. Schüßler-Plan is responsible for the structural planning (design and execution phases) for the entire complex of buildings and the design of the construction pit.

Construction pit

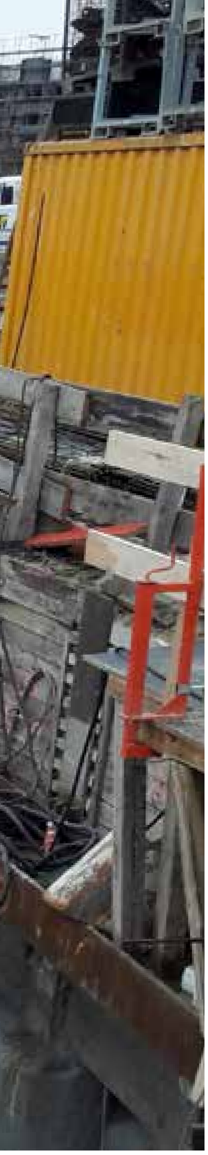
The base of the construction pit is 1.44 m above sea level. Compared to the highest point of the surrounding area, in the area of the abutment of the Baakenhafen Bridge, the maximum depth of the construction pit is approximately 8 m. In the area of lift pits, the excavation base is up to 2 m deeper.

The pit lining walls were constructed as rear anchored bored pile walls. The piles have a diameter of 750 mm an at a center to center distance of 1.55 m. The intermediate area is designed with shotconcrete. The upper 2.50 m of the pit lining will have to be removed as soon as the construction work is completed. Therefore in this area a soldier pile retaining wall consisting of I-beams inserted in the bored piles was realised.

The pit lining walls are planned as 'near rigid supported' foundation pit walls in accordance with EAB (recommendations of the working group excavation pits). According to the specifica-

Markus Krah
Schüßler-Plan Ingenieurgesellschaft mbH

1 View of the pit lining situation on the north side
2 Tower building
credits: Delugan Meissl Associated Architects
3 Cross section view of the quay wall and the building pit



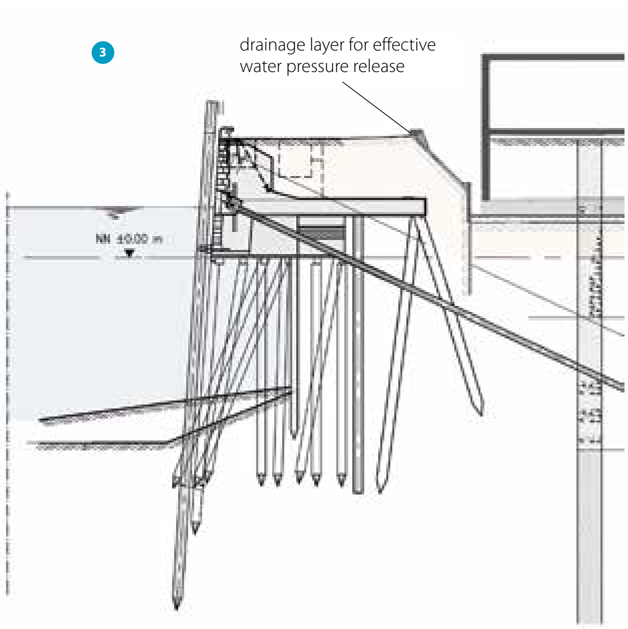
tions of the 'General conditions for licence areas' (Allgemeine Bedingungen für Gestattungsflächen) of HafenCity Hamburg GmbH, the horizontal deformations of the pit lining walls are limited to $v_h \leq 10$ mm [1].

Pit lining on the north side

On the north side of plot 80 a bored pile wall will be realised for the protection of the Versmannstraße, because a rear-anchored pit lining is not possible due to the nearby U4 line. The wall is reinforced with diagonal stays, which are supported against the partly completed base plate (photo 1). In the construction phase before the installation of the stays, the construction pit wall is supported by a berm. As a result, the required construction processes include interfaces with the building construction works, which had to be considered in advance in the planning.

Embankment and quay wall on the south side

Along the south side of the plot, the height difference between the bottom of the construction pit and the top edge of the adjacent quay wall (built in 1888) is approximately 3.50 m. The foundation pit has an incline here of between 45° and approximately 50°, and the surface is protected against erosion with shotcrete. This was necessary because the presence of existing quay makes the application of external anchorage impossible. Under the shotcrete, there is a drainage layer for effective water pressure release (fig. 3).

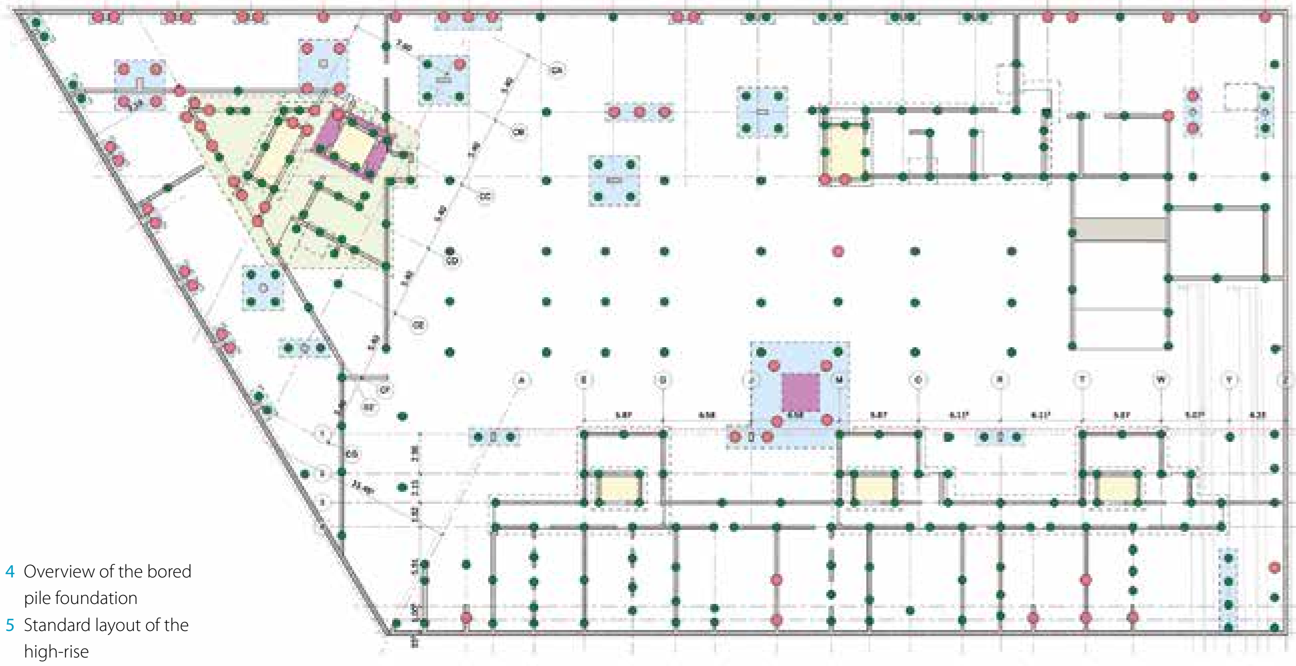


2

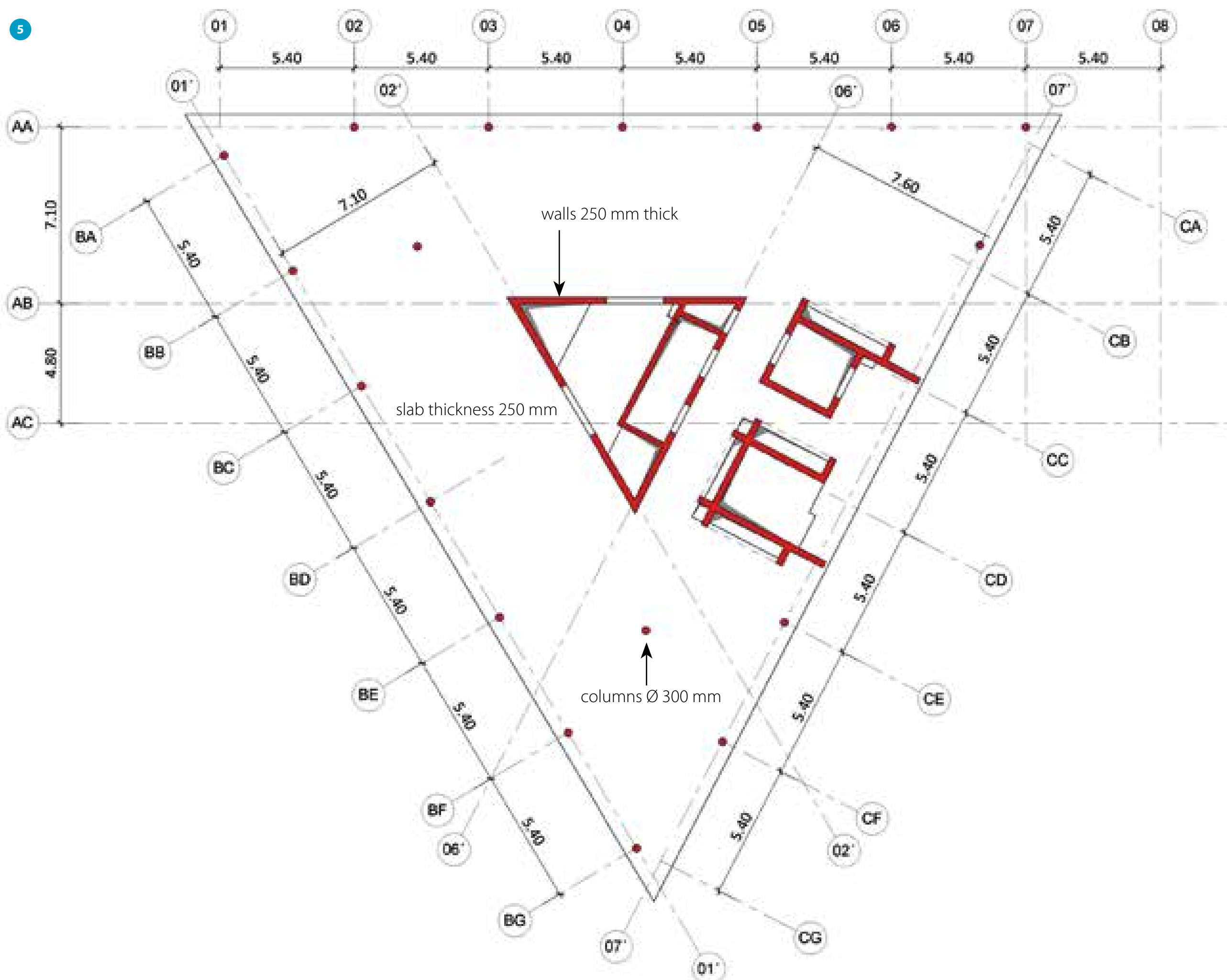
Foundation

In the investigation of underground conditions, in the first 3 m sandy fillings with soft layers (clay) were found. Next alternating layers of loosely layered sand and filled soft layers follow. Only after 8 m densely-packed sand, that is able to take a load, was found.
As the bottom edge of the structure is largely located in a filling, the load is transferred to the lower-lying sand that is able to take a load by a pile foundation. The piles are planned with diameters of Ø 600 mm and Ø 800 mm, and have a length of up to 25 m underneath the high-rise. In the calculation of the floor panel a spring stiffness of 240 MN/m (diameter 600 mm) and 340 MN/m (diameter 800) was applied. In the case of 9 m pile embedment into the load-bearing sands, a design load of $R_d = 3400$ kN was calculated for a 600 mm bored pile.

The vertical loads are diverted with point pressure and surface friction into the subsoil. Horizontal loads due to wind with short effect on the buildings and due to the sunk load cases (one-sided water pressure due to the draining-off of flood water) are transferred by areas through bedding and bending of the bored piles. For the horizontal bedding for the piles a bedding of $k_h = 1.25$ MN/m^2 was applied in the soft layer.
In addition to the usual load cases, the flood water load case with a flood level of +7,30 m above sea level and sunk load cases had to be considered: for the 0.65 m thick base plate this resulted in a maximum upward pressure of 55.50 kN/m^2. In total, there are approximately 340 bored piles spread across the



4 Overview of the bored pile foundation
5 Standard layout of the high-rise



entire construction site. Figure 4 shows the distribution of the bored piles (red: diameter 800 mm; green: diameter 600 mm).

The guideline 'Calculation models for flood protection walls, flood protection systems and waterfront structures in the area of the tidal Elbe of the Free and Hanseatic City of Hamburg' [3] and the 'Guideline for target heights and load assumptions for the HafenCity district' [4] are also taken into account in the structural calculations.

The arithmetically estimated pile resistances were reviewed with a static load test in accordance with 'EA – Pfähle' [5] (Recommendations of the Work Group 'Piles'). The load was applied centrically and axially with hydraulic presses. Steel trusses were used as abutments for the test load.

Structural design

High rise

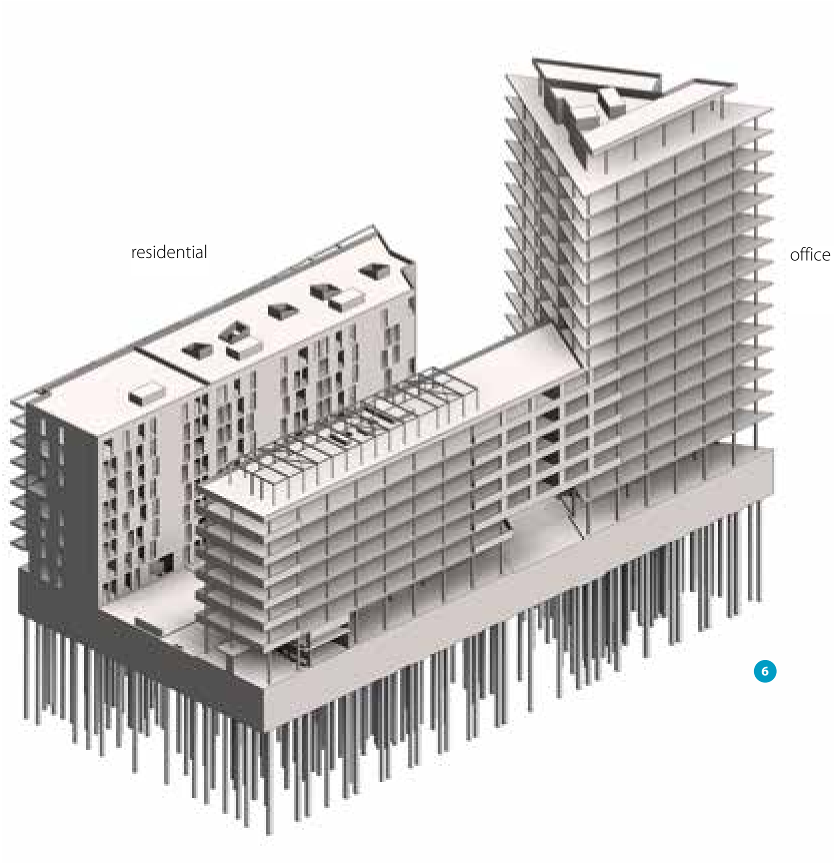
The 16-storey tower building and the adjacent office block extend along Versmannstrasse with a total length of 95 m. The standard layout of the high-rise has a clear structure with a support grid on the façade each 5.40 m with a total side length of L = 32.40 m (fig. 5). The distances to the reinforcing cores is L = 7.10 m.

The structural design is based on a reinforced concrete with 250 mm-thick flat slabs which are equipped with thermal component activation in the tower and the office block. The very slender supports in the high-rise are realised in high-strength concrete (C 80/95).

The building, which was examined holistically in a three-dimensional finite element model, is reinforced through the stairway cores and lift shafts. In the transition area between the tower and office building along Versmannstrasse, a 16.20 m wide passage to the inner courtyard is planned. This will be realised by a concrete structure in the area of the façade which will be formed over five floors as a Vierendeel truss (fig. 6).

Residential building

For the residential building, a design has been chosen which combines the efficiency and variability of the layouts: efficiency in terms of straight load transfer and flexibility in terms of the optimal arrangement of walls. In the transfer to the lower level, the loads were absorbed by roof beams and wall-like supports. The cantilever balconies in the area of the residential building are planned as prefabricated components and are fixed with Schöck Isokorb load-bearing elements in the 220 mm thick concrete slab.



6 3D model in REVIT

All plans produced by Schüßler-Plan are shown in a continuous, spatial 3D model in order to optimise the support structure, details and planning (fig. 6).
The building is currently being built and should be finished in spring 2018. ☒

REFERENCES

1 General conditions for licence areas (Version 06.05.2015), HafenCity Hamburg GmbH.

2 Geotechnical report, orienting contaminant investigation: Campus Tower, Hafencity Hamburg, Plot 80, IGB Ingenieurgesellschaft mbH, 29.07.2014.

3 Calculation models for flood protection walls, flood protection systems and waterfront structures in the area of the tidal Elbe of the Free and Hanseatic City of Hamburg; Free and Hanseatic City of Hamburg, Department for Roads, Bridges and Waterways.

4 Guideline for target heights and load assumptions for the HafenCity district; Free and Hanseatic City of Hamburg, Department for Roads, Bridges and Waterways.

5 EA – Pfähle, Recommendations of the Work Group 'Piles', German Geotechnical Society.

1

New football stadium with precast concrete structure for Belgium football club KV Ostend

Grandstands in precast concrete

When football club KV Ostend (Belgium) in June 1981 was founded after the fusion of former clubs VG Ostend and AS Ostend, it inherited the facilities dating back from the early 60's. Due to the old facilities, the club was not allowed to play after its qualification for the European competitions in 2014. Important renovations had to be done to acquire a European license. To fulfill the modern requirements and regulations in terms of safety and accessibility, new stadium grandstands with precast concrete were built.

A need to comply

For the construction works of the new stadium of KV Ostend an investment of 18 million € was needed. The new stadium structure was initially foreseen in steel. The general contractor however, proposed an alternative in precast concrete; this would result in short term cost reduction (no fire protection needed) and long term cost reduction (less maintenance). Eventually it was decided to build in precast concrete.

The work started in February 2016 with the demolition of the main grandstand and two smaller grandstands. The rebuilding

Johan Veys
Pieter van der Zee
Ergon nv Belgium

1 The new main grandstand
2 Situation sketch

of the new main grandstand and a E-grandstand behind one of the goals was finished just before the start of the new football competition, end of July 2016. The brand new grandstand can accommodate 3700 football fans. The total capacity has increased to around 8432, (3000 standing and 5125 sitting seats) and the stadium has become more comfortable. Underneath the new grandstand a space for commercial activities was created. The stadium now complies with the international UEFA standards and KV Ostend is ready to participate in European competitions.

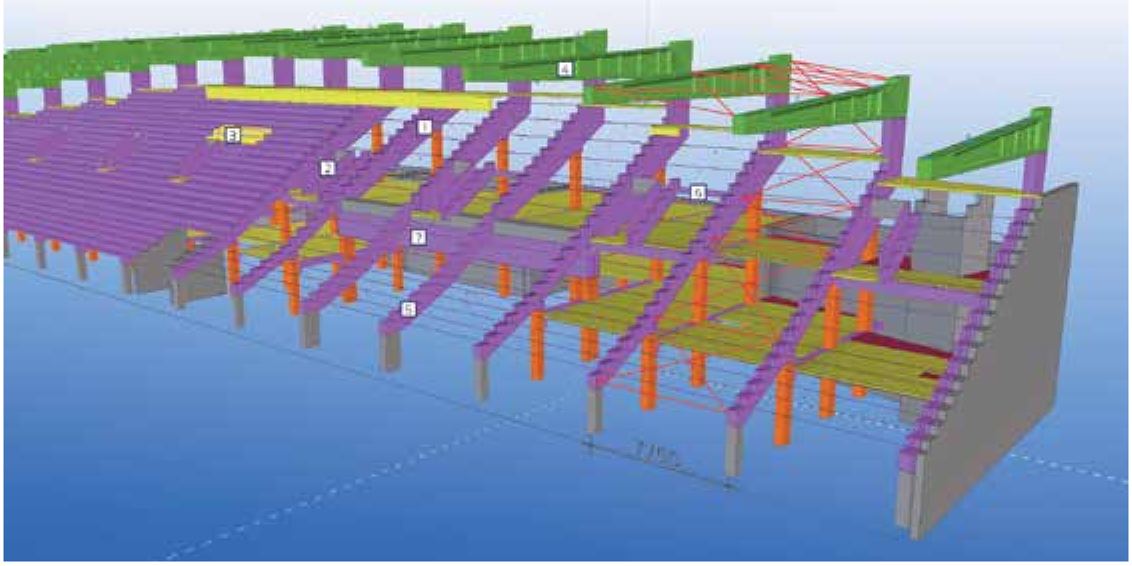
The precast structure

Ostend is a city on the North Sea, and the football stadium is located less than 300 m from the Belgian coastline. The environmental class for the exterior concrete elements is XF1, XC4, XS1. The wind class is category 0 and the wind speed taken into account is 26 m/s. The main grandstand has a length of 123.30 m, 27.90 m wide and has a height of 18.80 m. The E-grandstand is 75.60 m long, 9.00 m wide and 11.10 m high. The structure is created by using portals each 7.75 m; rafters are connected with a fixed connection to the raker beams, which are supported by several columns (fig. 3 and fig 5.).

Portal action is taken into account for the stability perpendicular to the grandstand, in the other direction there is a wind bracing in between the second and third axe. The prefabricated concrete structure with curved façade and bulb roof is made of columns that continue over several floors.

2

Legend:
1. raker beam
2. vomitory
3. terrace
4. rafter
5. buckled lower raker beam
6. trimmer beam
7. double beam above VIP lounge



3

3 3D view of the structural model
4 Connection raker-rafter beam
5 Cross section grandstand

Very short erection time

The erection of this 'not your everyday precast concrete structure' had to be completed in less than six weeks! A good organization was required because of the great number of different subcontractors present at the same time on a limited site area. The erection was done with a mobile, 400 tons crane LTM 1400.

Inside the building there are two levels of beams and - compared to standard slabs - a more performant type of hollow core slabs, with a thickness of 200 mm in order to obtain a floor as slim as possible. There is a VIP space with overhanging beams for the mezzanine floors and a party hall with double beams with a height of 1.90 m, a width of 0.54 m and a length of 23.75 m. Instead of one, two beams were used since one beam was too heavy for the erection crane. Even the weight of those two beams is already 61 tons each. These beams support the raker beams so that some columns under the raker beam in the party hall could be avoided (fig. 2).

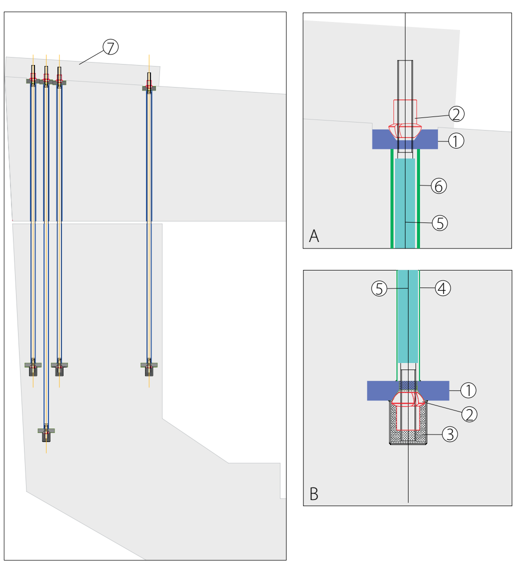
Building of the stands

The raker beams with denticulation - some of them even buckled in the longitudinal direction - weigh up to 42 tons and determine the shape of the exterior terrace. They hang over the outer facade in order to create a support for the rafter beams.

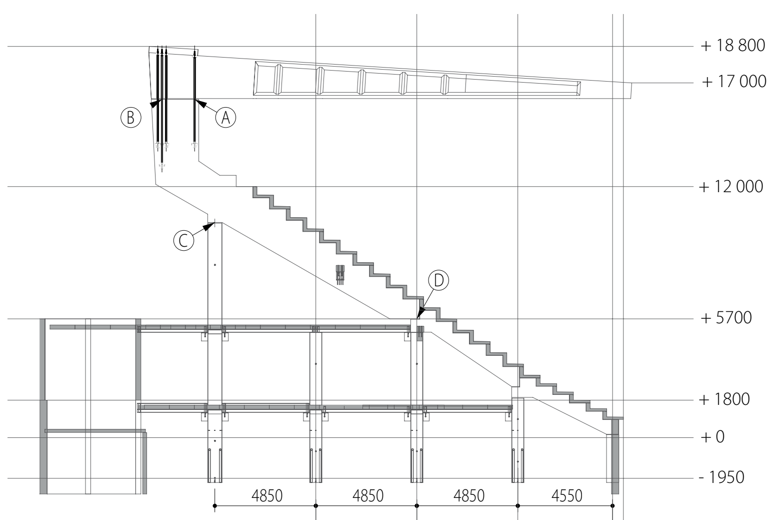
The rafter beams have to be able to absorb both positive and negative moments, caused by wind pressure and wind suction or snow. This has resulted in rafter beams with a tensile reinforcement of three layers of five bars Ø 40 on one side of the beam and eight bars Ø 40 on the other side of the beam.

The rafter beams with variable height and section (rectangular at the beginning and the end and I-section in the middle) mainly have pre-tensioning at the top (fig. 5). They were fabricated upside down because this is more safe in production and as such the prestressing steel could be horizontal. They have a length between 15.50 m and 23.00 m. These cantilevering beams are anchored in an end block through specifically designed bars of high-quality steel f_{yk} 950 N/mm² (fig. 4). The choice for this type of steel was made because the tensile force was too high to work with normal steel. These bars, protected by a thermal shrink layer, are screwed blind in the button headed nut of the cast in tube with a grease reservoir at the bottom. The nuts are anchored with distribution plates and splitting reinforcement in the concrete. To absorb the tolerances, these bars are blocked on top with a button headed nut in a hollow steel plate. The button headed nut is used to be sure that only normal forces can occur in the bar so parasitic moments are excluded. Afterwards, the bars were post-tensioned by a jack in several steps with forces over 100 tons. Thus a tensile force of 5400 kN could be absorbed with only six bars Ø 40. The self-

4



Legend:
1. hollow steel distribution plate
2. button headed nut
3. grease
4. steel straight tube
5. bar
6. ribbed steel tube
7. protecting concrete



5

weight of the beams during erection was retained with only two bars, thus facilitating a smooth execution. The post-tensioning is needed to get the crack-width in the connection within the boundaries for a sea environmental. On top of the beam a sufficient concrete cover for the entire connection was provided. The rafter has ribbed tubes so there is a better collaboration between the bar and the beam.
Because each beam is different in length and load, each one was calculated separately, e.g. the minimum and maximum reaction forces of the rafter beam on the raker beam (table 1 and fig. 5).

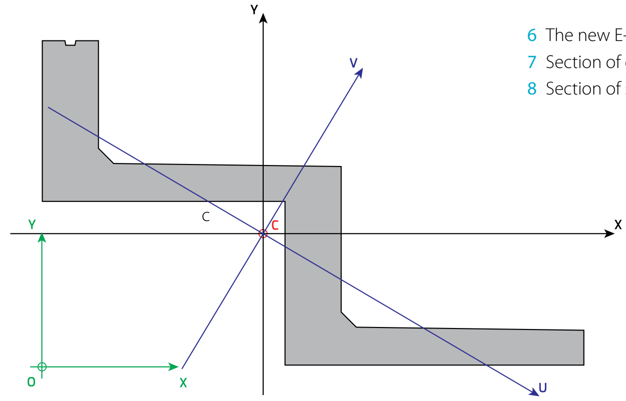
Table 1 Reaction forces raker and rafter beams in connections

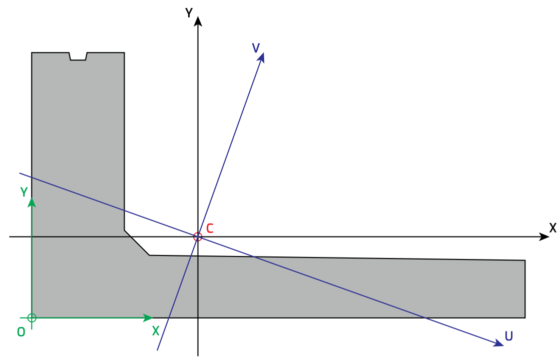
location	force	minimum	maximum
A	V_{Ed}	4790 kN (compression)	2790 kN (tension)
B	V_{Ed}	2640 kN (compression)	4085 kN (tension)
C	V_{Ed}	1480 kN (compression)	1205 kN (tension)
	M_{Ed}	4787 kNm	5041 kNm
D	V_{Ed}	5041 kN (compression)	181 kN (tension)

The 219 terraces exist of two heights in order to create a variable slope. In most cases they were executed with two stairs per element (fig. 7 and fig. 8), in order to reduce the production and erection time; this last one being the most important. In order to get the terraces visually horizontal, they were fabricated bending up 15 mm. The seats are interrupted by eight vomitories, which are hung up through sloping walls with denticulation and a trimmer beam construction between the raker beams. The first natural frequency of the terrace is 7.1 Hz, which is enough for a terrace with seats on it to avoid annoying movement. By making elements with two stairs they are a little bit more stiff because of a small rotation of the main inertia axis.

Innovation through 3D

The design was done using the Tekla 3D drawing program, thus preventing fitting mistakes in this double curved structure. This also facilitated the control of the structure by the architect and was used gratefully in a later phase by the other subcontractors. Our 3D model was awarded with the prize of the Construsoft BIM Award 2016 for the Benelux in the category 'Sports and recreation'. In order to use less materials, where possible, high-quality steel





6 The new E-grandstand
7 Section of double terrace
8 Section of single terrace

f_{pk} 1860 N/mm^2 for pre-tensioning and concrete with high strength up to C70/85 was used. With an equivalent capacity of the elements and less material, the $C0_2$ footprint is reduced. By using this superior concrete quality, it was also possible to reduce the amount of compressive reinforcement. The durability was also a consideration: an open stand with sea vista and the accompanying briny environment and chances of storms. The same goes for the strict requirements, norms and regulations of the Football Association in order to create a stand where soccer fans can safely gather to enjoy their idols play in a brand new arena. ☒

PROJECT DETAILS

architects abv+ architecten - Antwerp
engineer SBE - Sint Niklaas
general Contractor BAM Contractors - Brussels
project developer Groep Versluys - Ostend
client KV Ostend
precasters CRH structural concrete Belgium, Ergon - Lier, Prefaco - Lommel, Houthalen

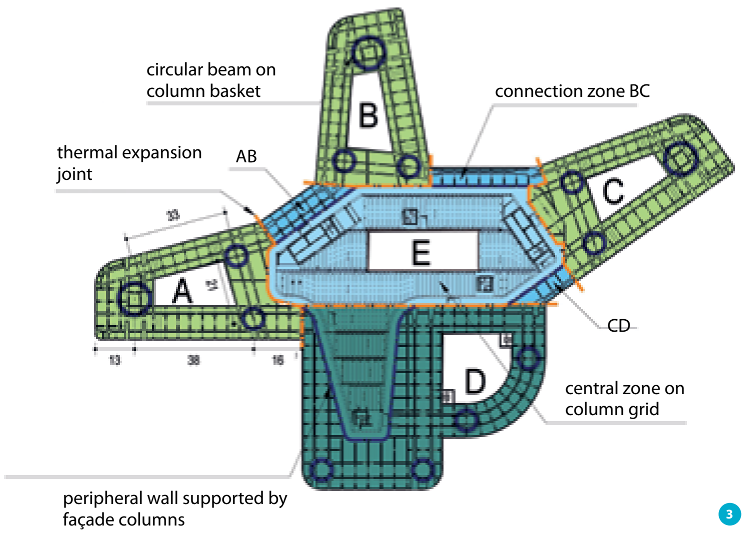
Application of post-tensioning in a concrete structure building

Hospital AZ Zeno in Knokke

In Knokke-Heist (Belgium) the new health care facility AZ Zeno is being built. Spreading over a 20 hectares ground, it houses a hospital of 360 beds, a rehabilitation center, a care hotel and public event spaces on a surface area of over 48 000 m².
The preliminary design stage began with a competition in 2007 and the inauguration is expected for the summer of 2017.

ir. Vincent Servais,
ir. Axel Rémont,
ir. Stephanie Pareit
Bureau Greisch

1 The new health care facility AZ Zeno in Knokke
2 Graphic design model: bird's-eye view on the hospital AZ Zeno
credits:: BURO II & ARCHI+I - AAPROG – BOECKX
3 General floor plan of the structure at the technical level



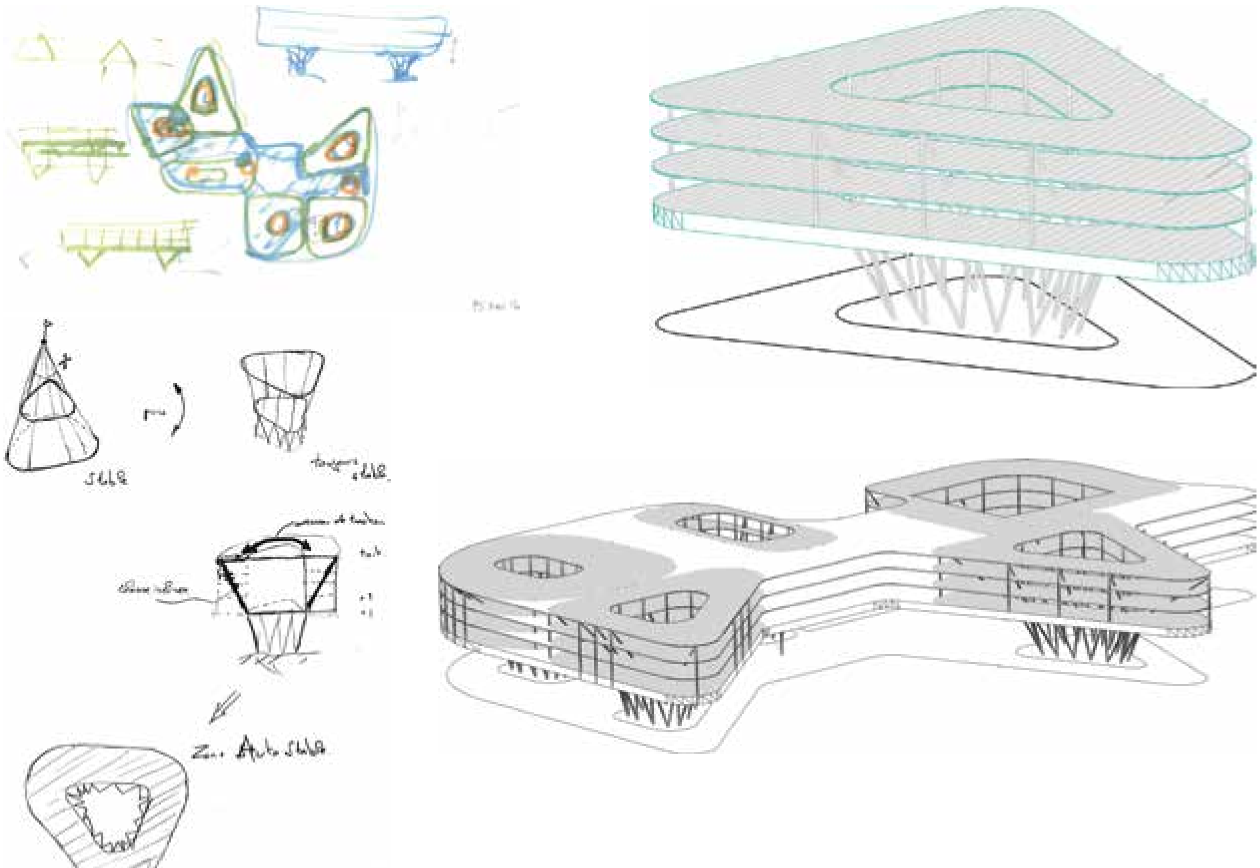
Architecture and landscape

The building seems to levitate above the landscape which gives a high level of transparency to the ground floor. Its design results from the ambition to avoid the feeling of entering a building when walking on the ground floor towards the hospital. The graphic design model of the complete building is shown in figure 2; figure 4 gives the starting structural concepts. The plan view of this low-rise building consists of a central zone, four wings and five patios (fig. 3). The building covers overall dimensions of approximately 200 m by 150 m. Above the open space at street level, the building consists of one technical level and three storeys. A basement is placed underneath the main part of the complex.
At the ground floor, the centrally located reception zone is surrounded by a 300 m glass façade that creates a seamless transition between the inside and outside, and is favorable to the integration of the building in the rural environment.
The same transparency and pleasant environment is pursued in the architectural configuration of the four hospital wings rising up from the ground and covering the outdoor space with its curve shaped undersurface. Each wing has a central patio; the corridor and rooms situated around the patios benefit from views on the landscape and abundant day light.

Structural conception

In the early stages of conception, several structural solutions were considered (fig. 4). Finally, the solution of a concrete box slab, positioned on a limited number of supports, is chosen to create the separation between the supporting deck and the upper structure, while the shape of the structural concrete directly stands out and does not require a non-structural skin. This solution is applied for the three almost identical wings A, B and C, and for wing D and the connection zones AB, BC and CD that are disposed around the central zone E (fig. 3). Each wing is composed of a double slab structure supported by three

4 Sketches and 3D simulation of the structural design in an concept stage
5 Simplified model used for shape determination and complete calculation model through wing D
6 Cross section through wing A



4

or four baskets of columns. These baskets are composed of 18 to 20 inclined steel tubes with outer diameter 298.5 mm that are arranged in two concentric circles (fig. 5).

A grid of vertical walls, varying in height from 0.70 m at the outer perimeter to maximum 3.30 m at the supports, connects the double slab structure consisting of two 250 mm thick plates and gives shear resistance to the structure. The walls are situated along the bearing directions and follow the disposition of the upper structure's columns, at its turn defined by the architectural grid of rooms and corridors with a typical span of 8 m (fig. 6).

The upper plate of the box slab is horizontal and supports the three levels of hospital rooms with a classical column and beam structure. The lower plate is curved to create a cloud-like shape that enhances the lightweight appearance of the building. It is an aesthetically attractive solution that also reflects the structural behavior: additional height and structural capacity is created at locations where bending moments are important (fig. 5). The complex geometry, the 30 to 40 m spans and the limited height of the structure have led to a cast in situ prestressed post-tensioned solution in C40/50 concrete.

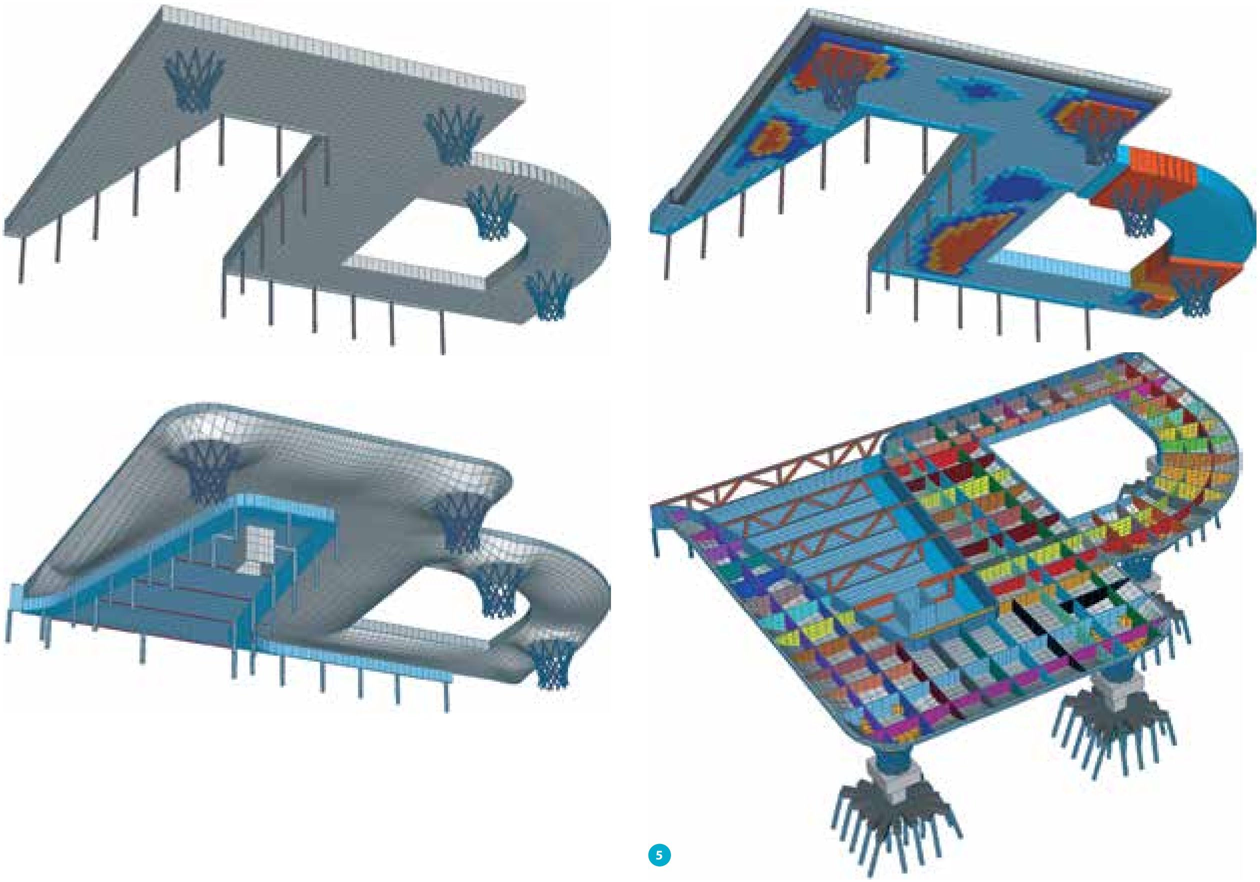
In the central zone E, it was important to provide free space in the reception area. Therefore a one-storey-high steel truss structure has been created at the first floor to support the regular column grid of the levels above. Only the vertical shafts and few columns remain on the ground floor. The truss level has a horizontal floor slab and offers the space to accommodate large technical installations. Technical ducts are connected to this equipment and branch out to the wings of the hospital passing through the hollow space between the cloud's slabs.

The building is founded on large concrete slabs, reinforced by foundation piles. Under the concentrated loads, mainly below the baskets, the foundation piles prevent excessive deformation. Under the basement, located under the central zone and wings B and D, these piles are loaded in tension when the groundwater level is high.

Shape optimization

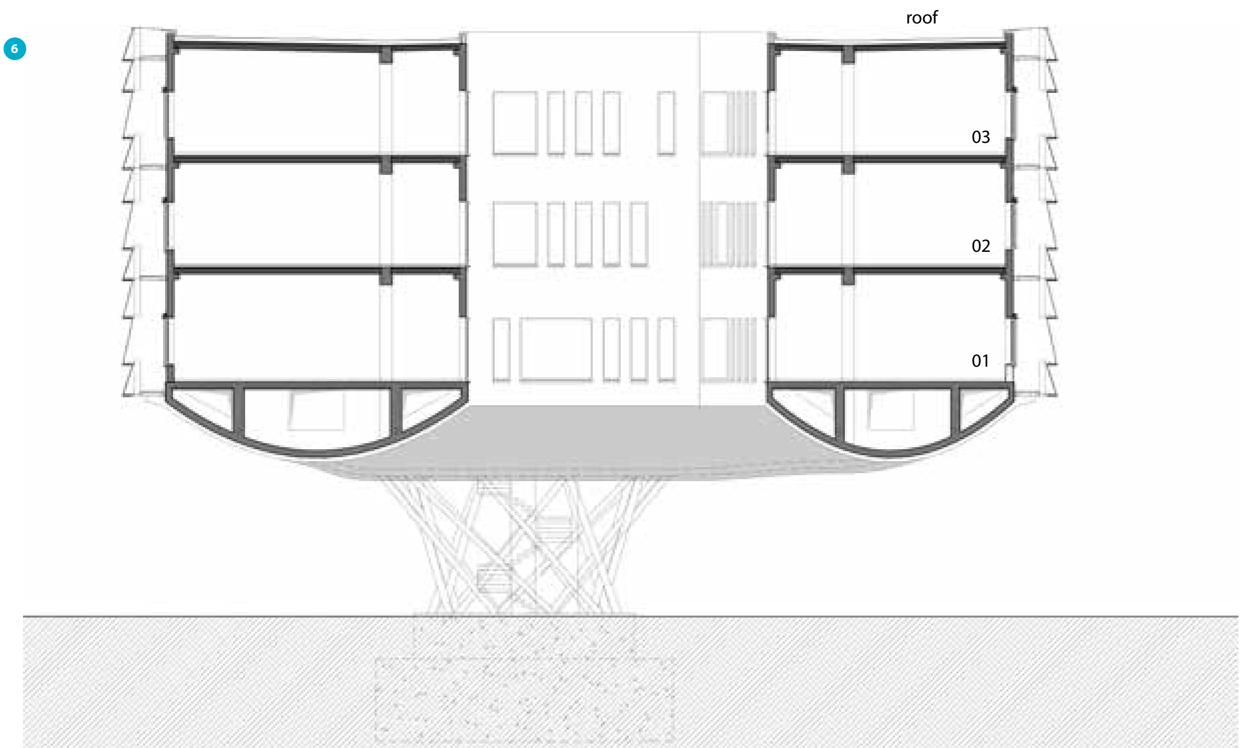
The shape of the lower plate is determined by both architectural and structural constraints. Indeed, architecture imposes geometric and aesthetic constraints such as:

- the small height of the concrete slab on its external border;
- the height of the concrete slab its transition to the central zone;



5

6



- an identical height for all supports;
- a minimum height of the structure to provide the space needed for the technical ducts between the upper and lower plate;
- a minimum height below the lower plate to allow fire rescue access.

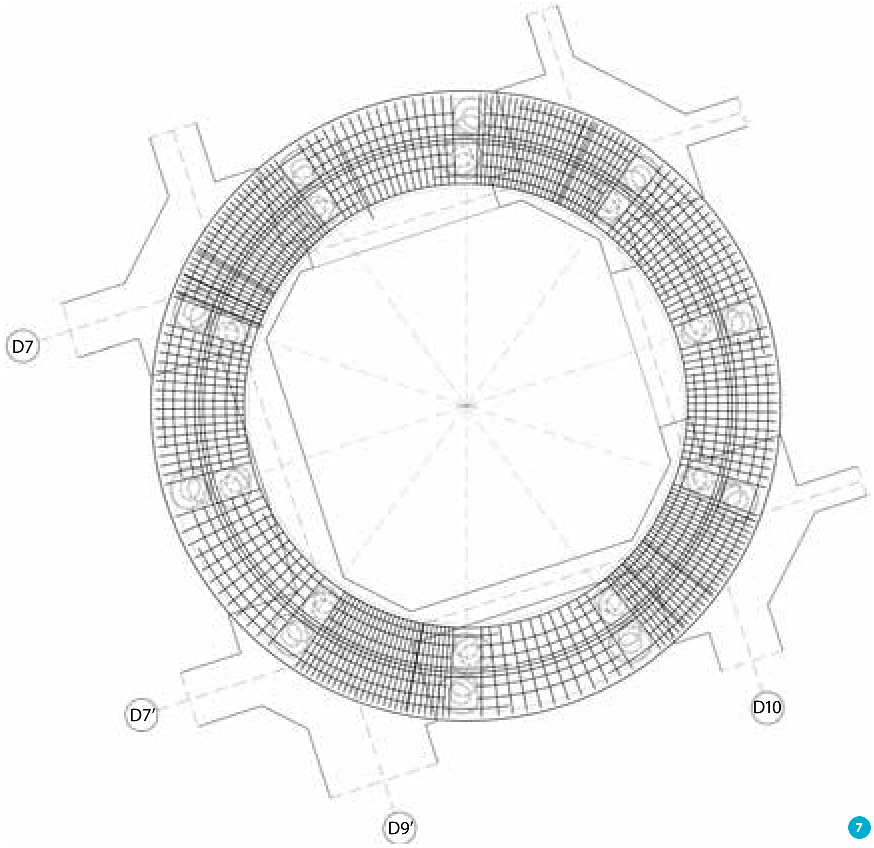
The general structural concept is to set the height of the double plate slab proportionally to the bending moments. For this purpose a first 3D finite element model was built. In this model, the double plate with vertical walls is simplified to one equivalent plate with variable thickness. Internal forces are analyzed and the thickness of the plate is defined for every

7 Drawing of the reinforcement of the ring beam
8 Transversal and longitudinal cross sections of wing A
9 Complex steel reinforcement of the bottom plate of box slab and the ring beams
credits: JL DERU
10 Posttensioning cables on the structural bearing axes of wing A

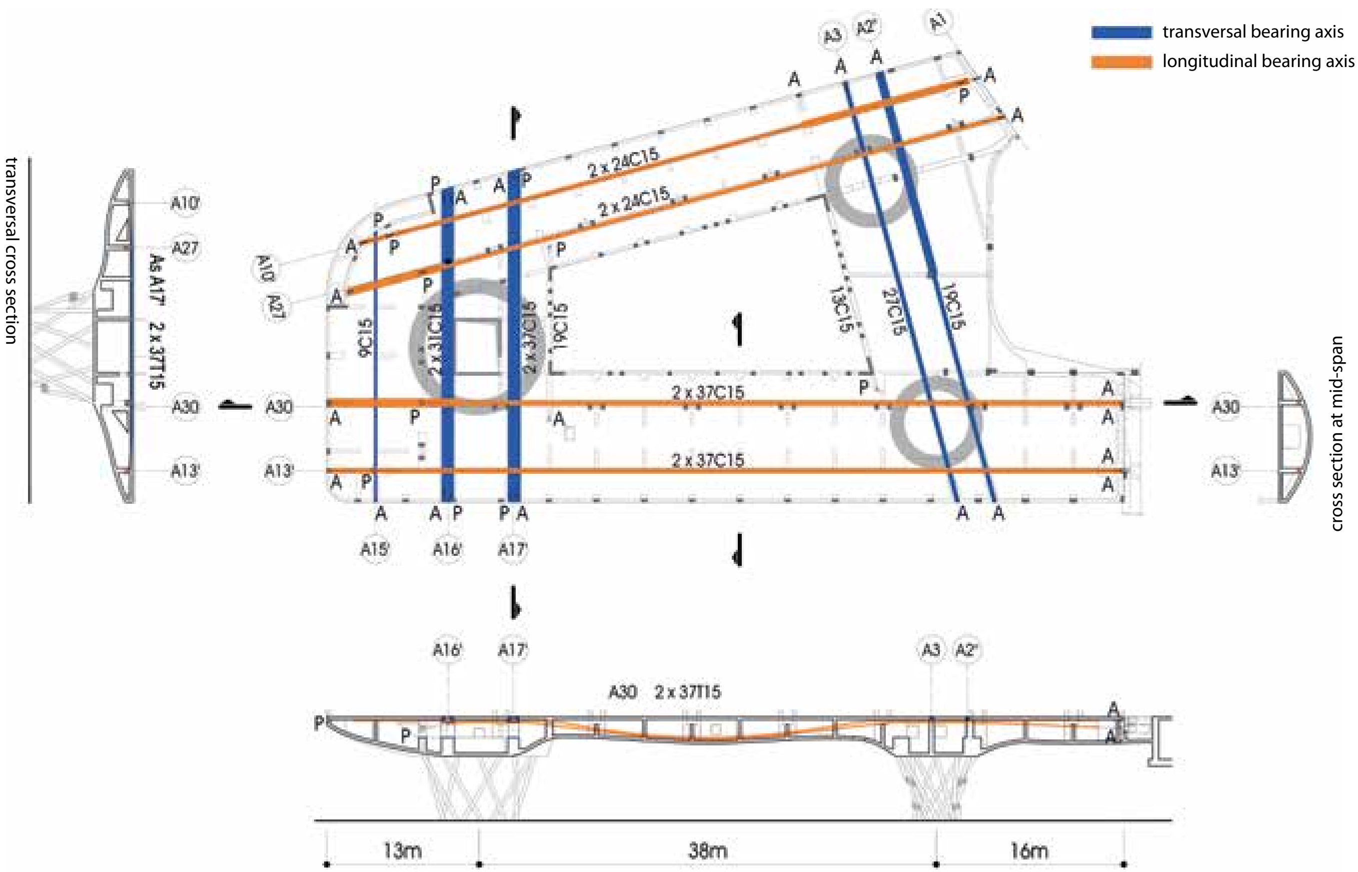
point of the surface. This is an iterative process since modification of the plate thickness causes stress redistribution. This process is carried out by an algorithm assigning, for each finite element of the slab, the optimum height regarding the bending moments in both principal directions. A manual correction is completed to avoid too many and too abrupt variations of height. Based on the modeling results, a precise 3D draft model of the structure is created. It is used to build a precise finite element model, including the representation of the upper plate, lower plate, and vertical walls. This is the final model used for further calculation.

Structural model

The 3D finite element plate model allows a realistic simulation of the overall structural behavior including effects of prestressing, creep and shrinkage, foundations and soil stiffness, and finally phasing stages of the construction. This model creates the possibility:
- to compute the stresses in all the different walls and plates;
- to determine the locations, the amount and the trajectories of prestressing cables;



7



8
longitudinal cross section

9

10

- to evaluate posttensioning losses due to restraints at supports and long term effects;
- to study the interaction between the different zones A, B, C, D and E, for example differential deformation at thermal expansion joints, and forces acting between neighboring wings;
- to evaluate displacements and their evolution over time and during the stages of the construction, taking into account creep and shrinkage effects;
- to define pre-cambers of the upper plate and the edge line of the double slab in order to respect limited tolerance for the disposition of the upper structure and the windows.

Conception and details of posttensioning

The vertical walls on the main bearing axes (fig. 8 and photo 10), are each provided with two ducts containing a number of strands varying between 19 and 37, introducing a compressive force of 170 kN per strand after losses. One of the design criteria to define the posttensioning forces in each axis is to avoid cracking of the concrete in serviceability state, for durability reasons and to limit deformations. To reach this goal, the trajectories of the cables are adjusted to compensate tensile stresses that correspond to the bending moments. Thus, the compressive posttensioning force is applied in the upper plate above the supports, and in the lower plate at mid-span.

As the global 3D finite element model does not suit to analyze local behavior, additional models are created to study details and connections. An example of this type of detail is the ring beam between the box slab and the supports. At the top of each basket structure, a concrete ring beam makes the interface between the lower plate of the concrete cloud and the steel tubes (fig. 7 and 12, photos 9 and 11). To obtain minimal visual

impact, both the ring beam and the column heads are sunk into the cloud. The ring beam's most important function is the redistribution of the vertical loads transmitted by the shear walls into the different columns. The ring beam also allows resisting the horizontal component of the compressive forces arriving on one side through the inclined lower plate, and on the other side by the tilted steel columns. Given the eccentric position of the beam compared to the lower plate, torsion had be taken into account. This results in highly steel reinforced zones.

Conclusion

The realization of the new hospital in Knokke-Heist is a one of a kind in health care construction. The structural concept is remarkable due to its high level of transparency throughout the building and its imaginative geometry. It requires a structural design that includes unusual techniques for both calculation and construction of a low-rise building. Realistic and complete 3D modeling allows to take into account all structural behaviors and interactions. Combined with advanced execution techniques like posttensioning and precise realization of the complex geometry, this project successfully integrated architectural idea with the optimized bearing structure. ☒

● PROJECT DETAILS

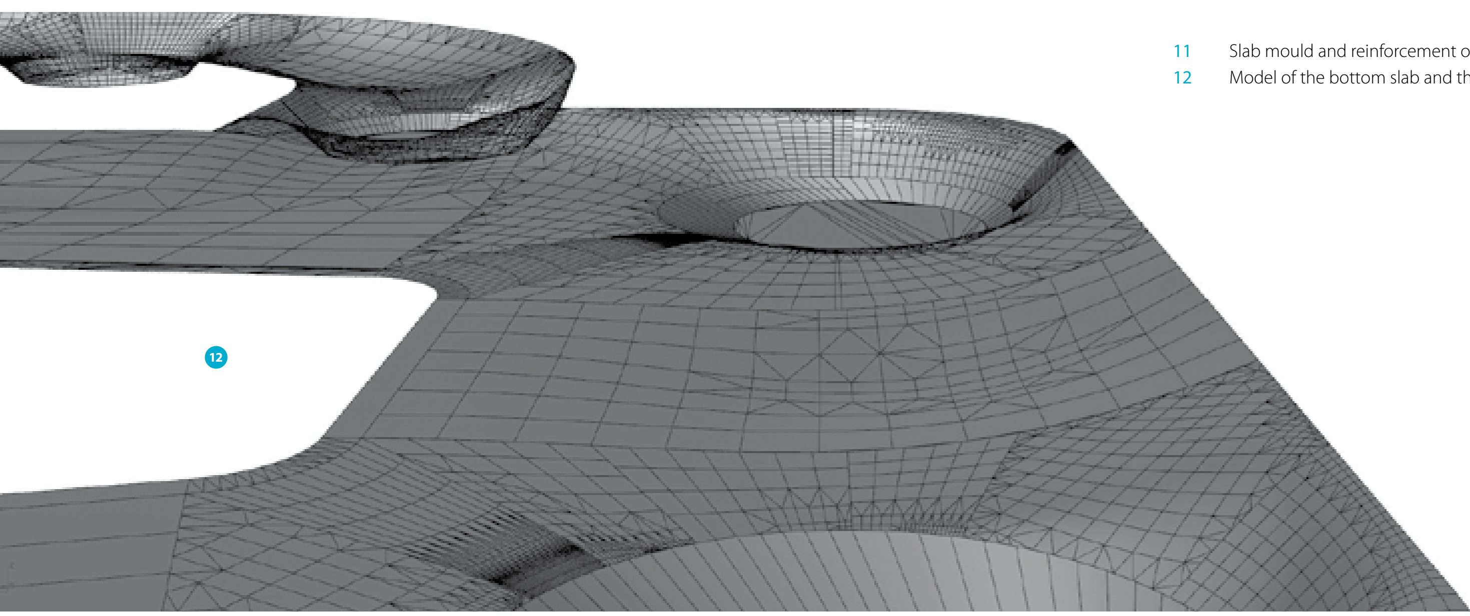
developer vzw Gezondheidszorg Oostkust

architect Temporary association BURO II & ARCHI+I - AAPROG - BOECKX.

stability Greisch – structural design of the ground level and technical level; SCES – foundations and prefabricated structure of the upper levels

technical equipment Ingenium

contractor BAM contractors



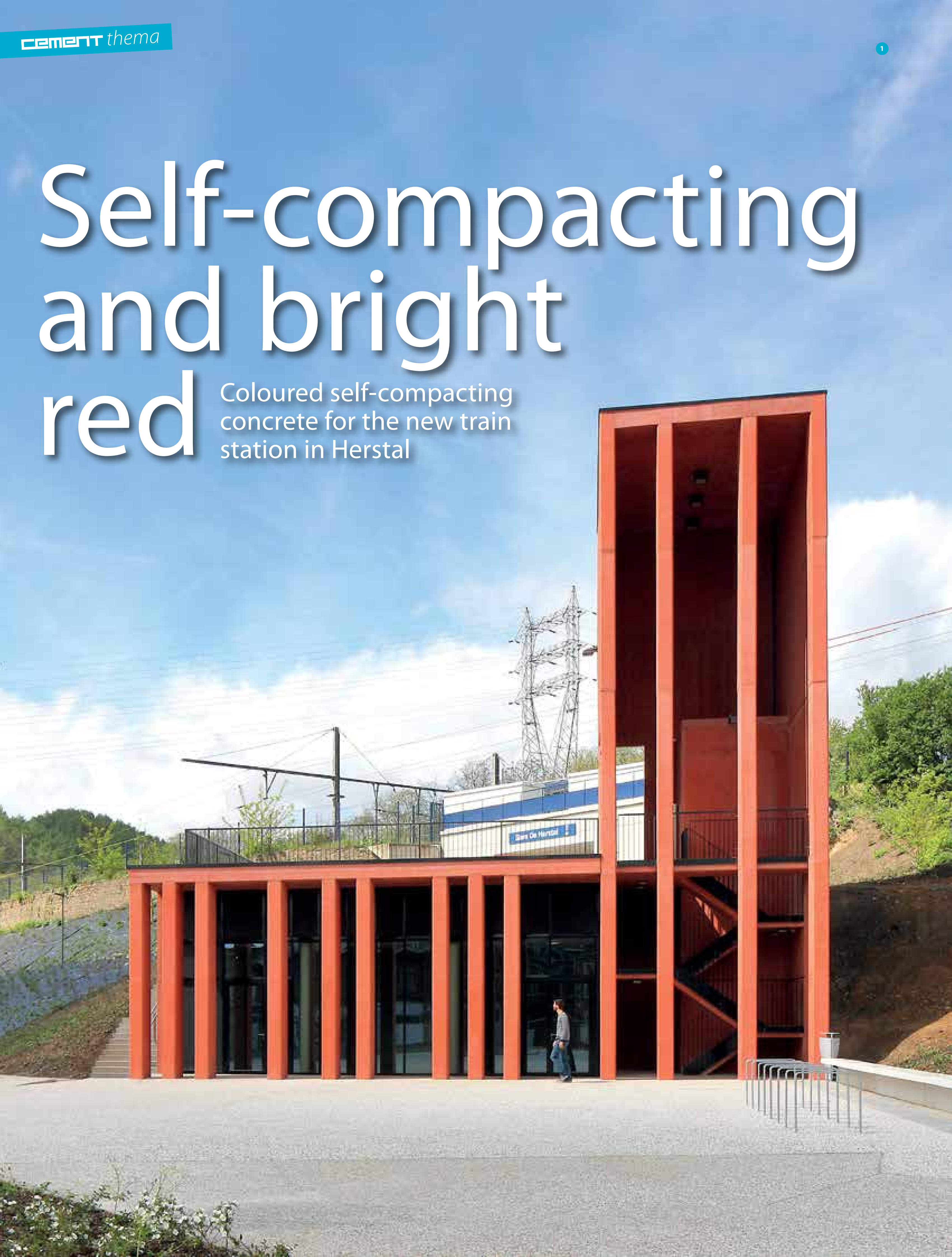
11 Slab mould and reinforcement of the ring beam

12 Model of the bottom slab and the ring beam


Beton met minder cement!?
Goed béééézig!?
Bescherm de lange levensduur van beton
Niet minder dan 25% klinker in het bindmiddel
Verduurzamen met de kaasschaaf in plaats van door echte innovaties gaat altijd ten koste van de kwaliteit en daarmee van de levensduur van beton
10% CO_2 winst = 50% kortere levensduur
Meer weten?
www.cementenbeton.nl

Self-compacting and bright red

Coloured self-compacting concrete for the new train station in Herstal



1

Jean-Philippe Jasienski,
Abdelmajid Boulaioun
MULTIPLE architecture & urbanism
Nathalie Balfroid
FEBELCEM
Steve Conard
Holcim

The new train station in Herstal, Belgium, stands as the new landmark of the city. Now connecting city parts previously disconnected, the high tower of the station ensures visibility and is becoming the symbol of the town. Beyond the strength of the architectural shape itself, the concrete building has a solid, red colour.

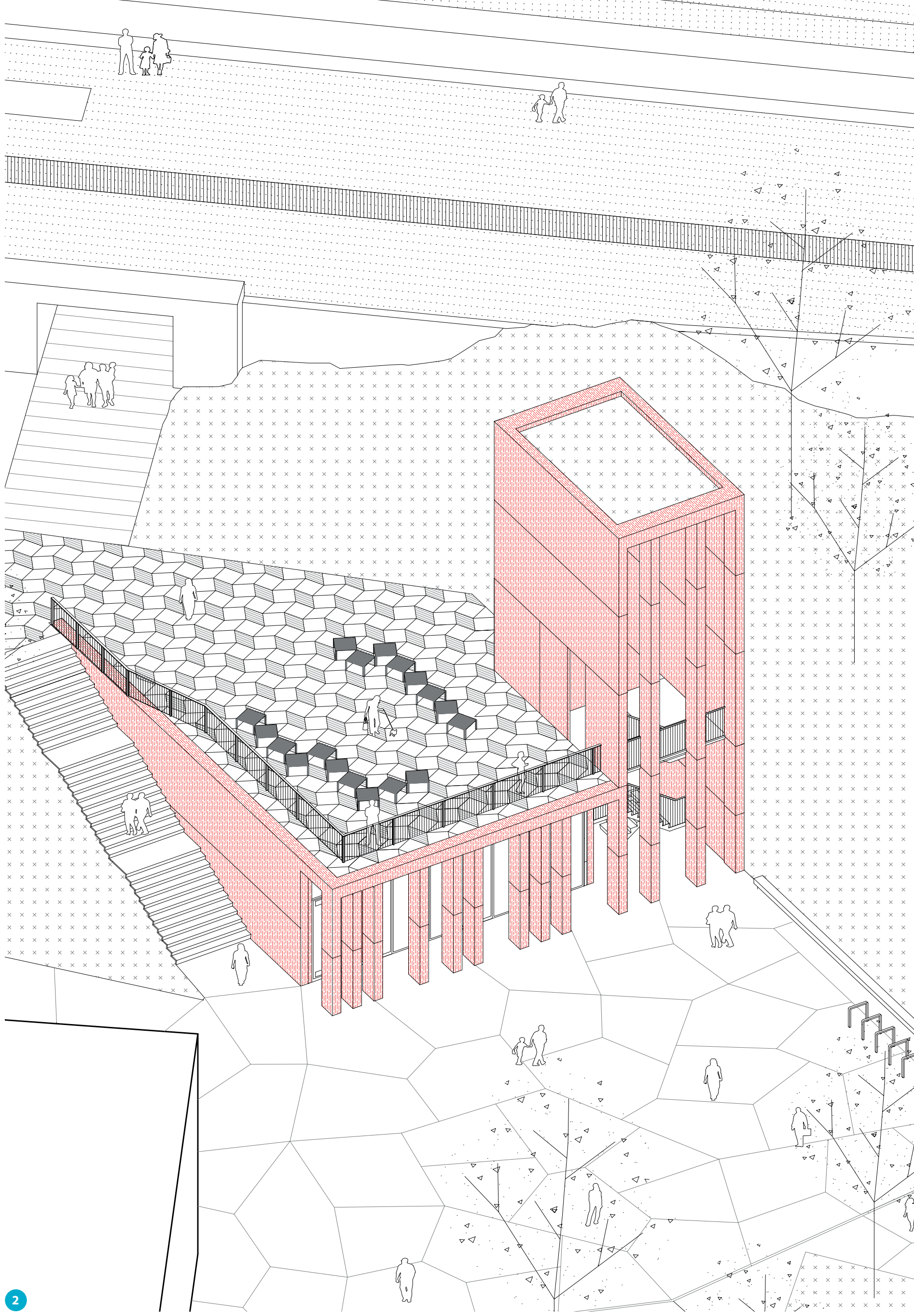
For its flexibility and in order to fulfil the structural and aesthetics requirements of the tower of the new train station in Herstal, ready-mix concrete has been chosen. In addition, the self-compacting property of the concrete facilitated meeting the aesthetic requirements for the concrete surface.
This paper redraws the main steps in the development, the production and the pouring of a self-compacting red concrete used for the construction of the building.

Architecture and landscape

The 'Pôle Marexhe', or the site of the urban renewal project, was a dense and decrepit city block, lacking public space, and hiding the green and hilly landscape in the background. This old city block was demolished and is replaced by a set of city spaces such as a new train station, four collective housing buildings including shopping facilities located around a new public square (fig. 2 and 4).

The new train station has been considered as the opportunity to create a link between two different levels of the city: the lower level valley (consisting of the square and the dense urban/industrial fabrics) and the upper, green landscape level (consisting of the hillsides and the park).
The concept of the building-square integrates two main stakes:
– generating an intermediate level, a belvedere;
– connecting the different levels of the public space.

Choosing the proper material was a key element of the project. The station is built in red self-compacting concrete (photo 1). The lateral sides of the building have a raw texture imprinted by the timber formwork (photo 3 and 5). This sober and raw expression character of the building is the inherent identity of the project. The front facade is smooth and is facing to the users of the public space.



2

First phase: study on the construction method

The tower of the station connects visually all the different levels of the project. It acts as a landmark for the city, allowing the train travellers to directly find their bearings. The tower contains the - direct - vertical circulations such as the elevator and stairs.
The height of the tower (16 m) is defined by the height of the surrounding landscape. Tower is peculiar because it does not have floors on its upper part (last 9 m out of the 16 m), making it more difficult to attain a satisfying structural rigidity.

Due to the high requirements on concrete quality and the contractor's lack of experience in making high quality exposed concrete, architectonic precast concrete first seemed as an appropriate solution. Different studies with companies specialized in precast concrete were made.
Two main limitations were met:

1 The new train station in Herstal, part of the 'Pôle Marexhe', 2016
credits: F. Dujardin

2 Axonometry of the 'Pôle Marexhe'
credits: MULTIPLE

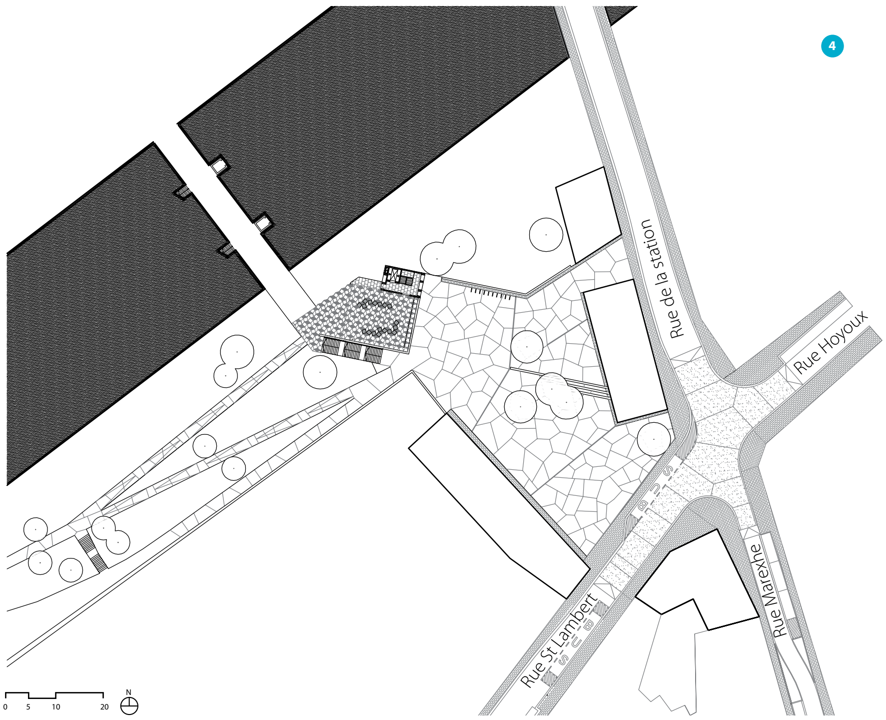
3

3, 5 The station is built in red self-compacting concrete with a raw texture imprinted by the timber formwork
credits: F. Dujardin

4 Site plan
credits: MULTIPLE

- The precast walls needed to have a high-quality finish on both sides. Due to production procedure of precast concrete element (i.e. horizontal casting), the companies could not produce massive wall elements that have architectonic finish on both faces.
- Due to the geometry of the tower (no intermediate floor on more than 9 m height), it would be very difficult to assemble massive precast wall elements and meet the structural requirements (i.e. the lateral stability).

Another solution was to produce sandwich wall panels composed of red architectonic concrete on both sides. These elements would then be assembled and connected by casting concrete between the two panels. This solution would easily meet the structural requirements. Unfortunately, the precast concrete industry is divided in two types of manufacturing companies. On one hand, the companies that make architectonic concrete do not build these specific structural elements. On the other hand, the companies that build sandwich wall panels could not meet the aesthetic requirements of the projects (architectonic finish and colour). In addition, the details (i.e. some angles and connections) would not attain the targeted level of quality.
Finally it was decided to use ready-mix concrete.



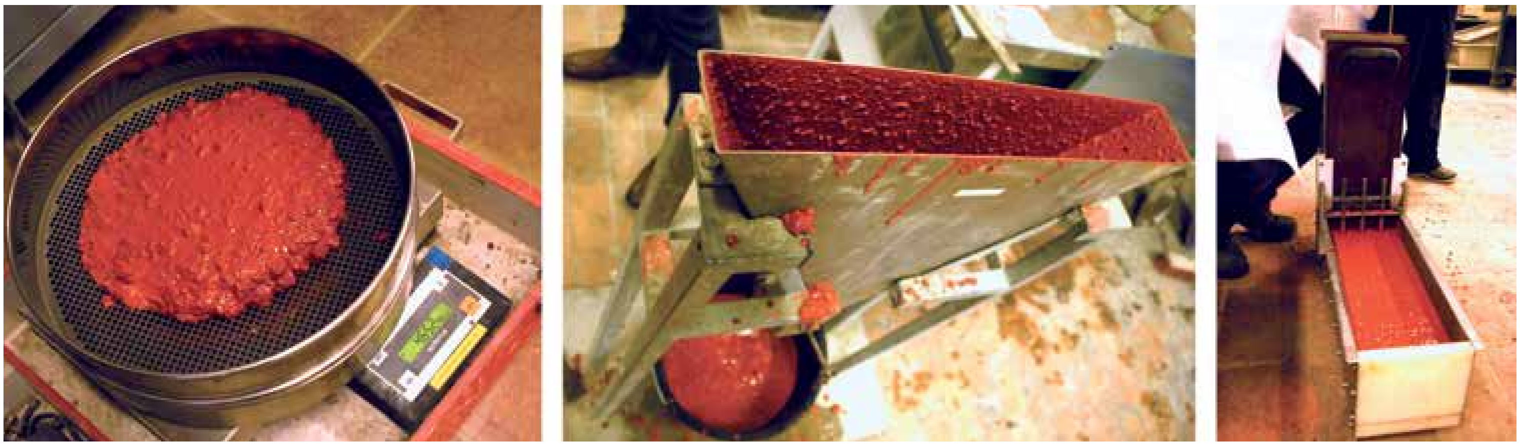
4

5

6 Tests at the CRIC lab; determination of the colour and the texture
credits: J.P. Jasienski

7 Tests on self-compacting concrete:
(a) resistance to segregation,
(b) viscosity and flow velocity, and
(c) mobility and obstacles penetration
credits: J.P. Jasienski

6a 6b 6c 6d



7a 7b 7c

Second phase: definition of the composition of the concrete

The desired colour was intense and bright red. In order to reach this colour, different tests (both in the laboratory and on-site) were necessary. The first series of laboratory tests were mainly focused on obtaining a satisfying colour (photo 6). The different type of pigments, their percentage in the mix and the type of cement (white or light grey) were tested. Based on experience and literature (Lanxess, 2014), the percentage of pigment recommended before saturation is around 6%. In this case, it was fixed to 5.5%, which corresponds to one bag of pigment
(20 kg) per m^3 of concrete – a simple measure that avoids differences in production. This first test series helped to decide the exact pigment to pick (some were either too orange or too scarlet), but also the kind of cement. It appeared that the brightness of CEM III/B (light grey) and white cement were very similar and both satisfying. Therefore, the CEM III/B was further used.

The lateral faces of the station were textured. Different tests were carried out with polyurethane-elastomers form liners and wooden strips. The wooden strips were chosen due to their more 'organic' aspect and their ability to visually unify and 'correct' small imperfections in concrete itself (photo 6c and 6 d).

Results were still unsatisfactory due to bubbles at the surface. It seemed that the concrete was sticking to the formwork. Although the concrete was successfully fulfilling the usual tests (density, segregation and air content), the problem was not solved. Different types of formwork oils were tested but the results were still unacceptable.

In order to solve the problem of the bubbles it was decided to continue with self-compacting concrete. Moreover, considering the contractor's lack of experience, a self-compacting concrete appeared as a good way to minimize the impact of the concrete pouring on the surface quality. A simpler pouring method and the absence of a vibration process (the concrete self-places and self-compacts) were key points to opt for this kind of concrete. Tests with a modified composition including limestone filler (instead of fly ashes that would damage the red colour) were carried out to improve the self-compacting ability and the concrete surface finish.

8 The two 1/1 scale on-site mock-ups (a and b), and the resulting concrete, references for the reception on the further works (c and d)
credits: J.P. Jasienski

9 Inside view on the new Herstal train station
credits: F. Dujardin

Table 1 Main characteristics of the coloured Self-compacting concrete

Materials	quantity
- CEM III/B 42,5N LA HSR	380 kg/m³
- Limestone Filler L	220 kg/m³
- Water/cement ratio	0,50
- Red pigments	20 kg/m³

Table 2 Tests executed in the labs and fixed values

Tests	values
- MVh – Density (NBN EN 12350-6)	2320kg/m³*
- Concrete slump flow test – Abrams cone (NBN EN 12350-8)	660 mm T500 = 3,5 s No segregation*
- Air content (NBN EN 12350-7)	2,6%*
- Mobility and fill rate – L-box with 3 bars (NBN EN 12350-10) - Viscosity and flow velocity - V Funnel test (NBN EN 12350-9)	0,9 15,0 s
- Resistance to segregation - Stability at sieve (NBN EN 12350-11) % of laitance Bleeding after 15 min	 13 % no

* these measures were tested both at the concrete mixer plant and on the work site.

Due to lack of standards on self-compacting concrete, several lab tests were performed (photo 7) and used subsequently as base for the on-site casting (table 2).

Third phase: building site realization

Two 1/1 scale mock-ups were realized for three different purposes (photo 8). First to verify that the same specifications and quality of the approved samples made in the lab are still valid in real conditions (outdoor, bigger scale, ready-mix concrete from the concrete mixer plant, type of oil used for a formwork, resistance of formwork panels, setting the speed of the concrete pump…) and to approve the texture (soft and wood-textured) and the final colour. Secondly, to validate a reference that would stay on-site to serve as base for the further works. Eventually this was used to train the workers how to pour the concrete properly.

A strict building procedure was applied to avoid loss of quality:

- A clear process for the formworks was established. The formwork oil had to be applied within maximum 24 hours before casting, and the wood strips had to be humidified. It was assured that the formwork could resist very high pressure caused by the height of pouring (more than 5 m).
- The concrete had to comply to established tests and reference values (see Second phase - at the concrete mixer plant and on-site). The details of the delivery note also needed to be verified. The contractor had to work with a nearby concrete mixer plant to ensure the workability of the fresh concrete (less than 15 minutes in this case).
- During the mock-up tests, it appeared that it was better to set the speed of the concrete pump to the minimum and pour the concrete directly without any fall. Thus the extremity of the pump had to be always dipped inside the concrete. Since the handling of the head of the pump is not such an easy task, the same worker did the pouring for all the concrete of the building.
- A strict formwork stripping time of 48 to 72 h was defined through the mock-ups. This value had to be respected to secure the homogeneity of the hue.
- Due to different reasons (such as blemishes) some elements have been demolished and re-built on the building site. Their non-satisfactory aspect was easy to determine thanks to the mock-ups used as references and to the CIB scale of the standard PTV 21-601. For the main columns, an underesti-mated value of the pressure of the concrete (almost 6 m

8a

8b

8c

8d


9

height poured at one time) led to deformations of the formwork that resulted in slightly bowed columns. The contractor eventually removed them and re-casted them with more reinforced formwork.

Conclusions

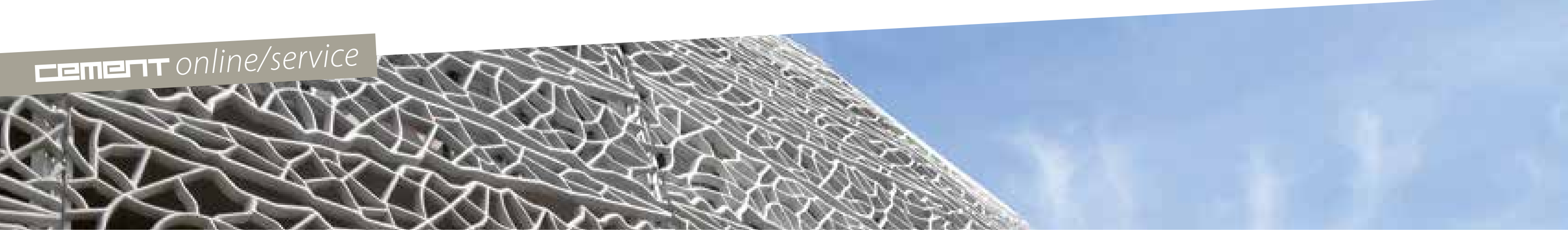
Through the case study of the realization of Herstal train station, this paper explained the methodology that was developed from the design process to the realization of the building to achieve a high quality and uniformly coloured self-compacting concrete structure.

Grand-Prix d'architecture de Wallonie 2015

The project was awarded Grand-Prix d'architecture de Wallonie 2015. The jury highlighted that the project aims to be both modest and very ambitious and sets a new frame for public spaces to support urban life.

REFERENCES

1 Basic rules for achieving optimal results when colouring building materials (2014). *Concrete Plant International* 5, Lanxess Deutschland GmbH, pp 80-90.
2 NBN EN 12350-6:2009. Testing fresh concrete - Density.
3 NBN EN 12350-7:2009. Testing fresh concrete - Air content: Pressure methods.
4 NBN EN 12350-8:2010. Testing fresh concrete - Self-compacting concrete: Slump-flow test.
5 NBN EN 12350-9:2010. Testing fresh concrete - Self-compacting concrete: V-funnel test.
6 NBN EN 12350-10:2010. Testing fresh concrete - Self-compacting concrete: L box test.
7 NBN EN 12350-11:2010. Testing fresh concrete - Self-compacting concrete: Sieve segregation test.
8 PTV 21-601:2016. Precast elements of architectural concrete, PROBETON Technical Prescriptions.

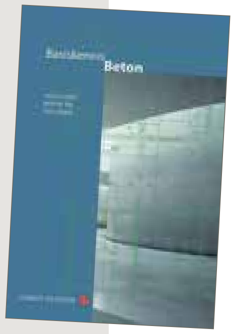


Betonkalender

Op www. betonkalender.nl ziet u in één oogopslag welke evenementen er voor de beton- en constructie wereld worden georganiseerd. Ook maakt de website een goede afstemming tussen activiteiten mogelijk. De betonkalender is voor iedereen toegankelijk en deelname is gratis. Wilt u uw evenement onder de aandacht brengen? Vraag een account aan via cement@aeneas.nl.

Nog geen lid?

Nader kennis maken met *Cement*? Wordt dan lid op de nieuwsbrief of bestel een proefnummer in de webshop.



Wilt u niets missen van *Cement*? Kies dan voor een compleet jaarlidmaatschap en ontvang acht keer per jaar *Cement* in uw brievenbus én 24 uur per dag toegang tot 67 jaar waardevolle vakinformatie. Word nú lid en ontvang het boek Basiskennis Beton (CB1) cadeau.

Ga naar de webshop op www.cementonline.nl.

Volgend themanummer: Tunnels

Er worden momenteel veel belangrijke tunnels gerealiseerd, zowel in Nederland als in het buitenland. In het volgende themanummer van *Cement* gaan we in op de ontwikkelingen op het gebied van tunnelbouw en worden de meest interessante tunnelprojecten belicht.

Overige geplande thema's

- Materiaalontwikkeling
- Sluizen

Digitaal woordenboek voor de betonbranche

In de bouwwereld worden veel verschillende begripppen gebruikt, met soms meerdere betekenissen. Het Beton-Lexicon biedt uitkomst. Op www.betonlexicon.nl vindt u ruim 375 begrippen uit de cement- en betonwereld, die steeds worden geactualiseerd en aangevuld. De begrippen zijn voorzien van uitleg, illustraties en foto's en bevatten verwijzingen naar websites, vakliteratuur en regelgeving.

Meest geraadpleegde artikelen mei 2017

1. Wanneer is beton waterdicht?, *Cement* 2016/1
2. UHSB als alternatief voor hout, *Cement* 2016/2
3. De uitdagingen van een integraalconstructie, *Cement* 2016/6
4. Bijzondere vloersystemen voor stadskantoor, *Cement* 2016/2
5. 165 m prefab beton, *Cement* 2017/2

Meer actuele informatie vindt u op www.cementonline.nl

Volg *Cement* ook op Twitter @CementNL en LinkedIn

COLOFON
Cement, vakblad over betonconstructies, is hét vakblad van en voor constructeurs en verschijnt 8 keer per jaar. Het vakblad is een onderdeel van het kennisplatform *Cement*, een uitgave van Aeneas Media bv in opdracht van het Cement&BetonCentrum. *Cement* brengt tevens informatie van de Betonvereniging en van SBRCURnet op betongebied.

Uitgave
Aeneas Media bv
Veemarktkade 8, Ruimte 4121
5222 AE 's-Hertogenbosch
T 073 205 10 10, www.aeneas.nl

Redactie
prof.dr.ir. Dick Hordijk (hoofdredacteur), dr.ir. drs. René Braam, ir. Paul Lagendijk, ir. Jacques Linssen, ir. René Sterken, ing. Henk Wapperom

Redactieraad
ir. André Burger (voorzitter), prof. dr.ir. Jos Brouwers, Stijn Joosten MSc, ir. Guus Keusters, ing. Joop van Leeuwen, dr.ir. Mantijn van Leeuwen, ing. Michael van Nielen PMSE, ir. Ton Pielkenrood, ir. Kees Quartel, ir. Hans Ramler, ir. Luc Rens, ir. Paul Rijpstra, ing. Jos Rooijakkers, prof.dr.ir. Theo Salet, dr.ir. Raphaël Steenbergen, prof.dr.ir. Luc Taerwe, prof.dr.ir. Lucie Vandewalle, ir. Cindy Vissering, ing. Henk ter Welle, ing. Jan van der Windt

Uitgever / vakredacteur
ir. Jacques Linssen
j.linssen@aeneas.nl, T 073 205 10 22

Productiecoördinatie
Hanneke Schaap
h.schaap@aeneas.nl, T 073 205 10 19

Eindredactie
Redactie & Zo, ir. Caroline van der Meulen

Ontwerp
Twin Media bv, Rimke Bartels

Vormgeving
Twin Media bv

Website
www.cementonline.nl

Advertentieverkoop
Sanne Verdonk
s.verdonk@aeneas.nl, T 073 205 10 23

Lezersservice
lezersservice@aeneas.nl
T 073 205 10 10

Abonnementen 2017

Jaarabonnement *Cement* vakblad	€ 180,-
Jaarabonnement *Cement* online	€ 155,-
Jaarabonnement *Cement* compleet (vakblad + online)	€ 270,-

Voor meer abonnementsvormen kijk op www.cementonline.nl of mail naar lezersservice@aeneas.nl.

Alle prijzen zijn exclusief btw. Buiten Nederland geldt een toeslag voor extra porto. Abonnementen lopen per jaar en kunnen elk gewenst moment ingaan. Opzeggen kan telefonisch, uiterlijk twee maanden voor vervaldatum.

Overname artikelen
Overname van artikelen en illustraties is alleen toegestaan na schriftelijke toestemming.

Voorwaarden
Cement wordt tevens elektronisch opgeslagen en geëxploiteerd. Alle auteurs van tekstbijdragen in de vorm van artikelen of ingezonden brieven en/of makers van beeldmateriaal worden geacht daarvan op de hoogte te zijn en daarmee in te stemmen, e.e.a. overeenkomstig de publicatie- en/of inkoopvoorwaarden. Deze liggen bij de redactie ter inzage en zijn op te vragen.

Hoewel de grootst mogelijke zorg wordt besteed aan de inhoud van het blad, zijn redactie en uitgever van *Cement* niet aansprakelijk voor de gevolgen, van welke aard ook, van handelingen en/of beslissingen gebaseerd op de informatie in deze uitgave.

Niet altijd kunnen rechthebbenden van gebruikt beeldmateriaal worden achterhaald. Belanghebbenden kunnen contact opnemen met de uitgever.

ISSN 0008-8811

MIX
Papier van verantwoorde herkomst
FSC® C015620

Waalbrug Ewijk (A 50)
Dyckerhoff Basal levert hoge sterkte beton C 90/105; Dyckerhoff levert speciale cement VARIODUR 30

Fotos: Bart van Hoek

Dyckerhoff …maakt er meer van.

Dyckerhoff,
Verkoopkantoor Nederland/Belgie
nl@dyckerhoff.com
www.dyckerhoff.nl

Dyckerhoff