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

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

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.
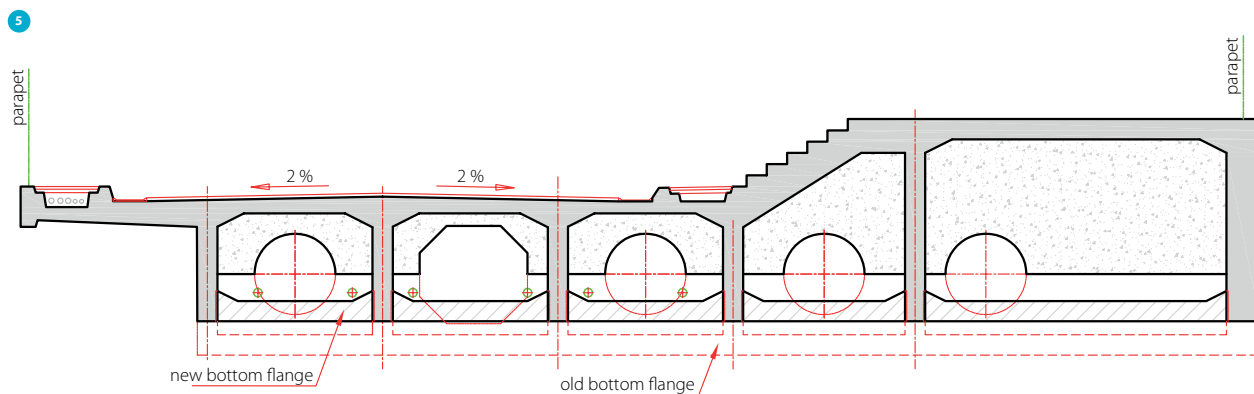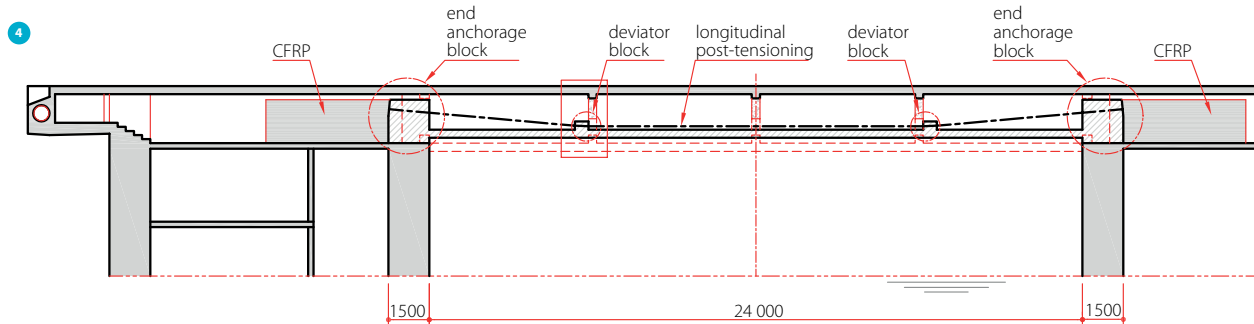
Study phase
With regard to the construction phases, the following major issues had to be addressed:
- the capacity of the existing structure;
- the design of the new bottom flange;
- the post-tensioning forces, their anchoring and the potential additional strengthening measures can be without this.
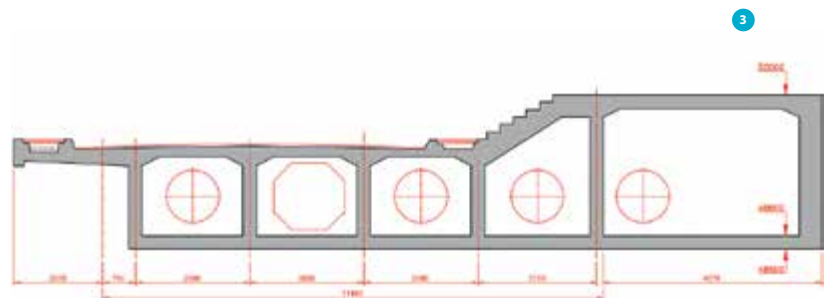
Existing structure

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.
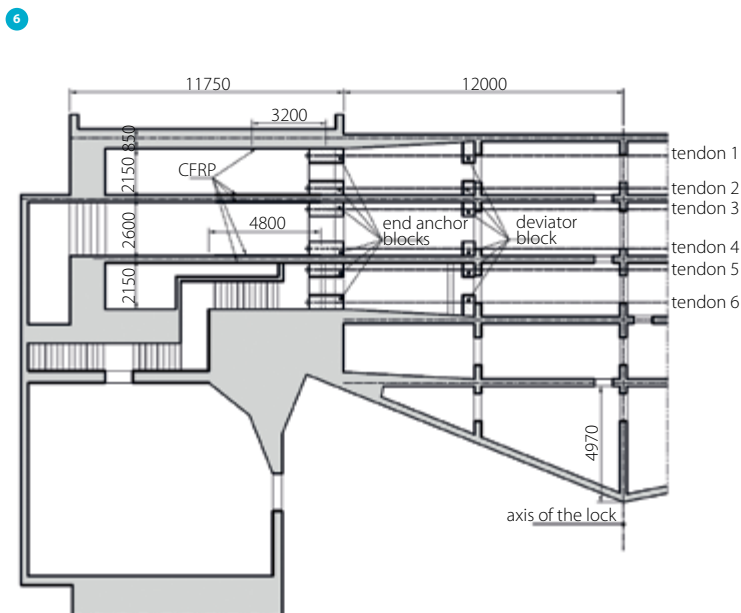
Acting loads on the integrated structure of lock and bridge

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.

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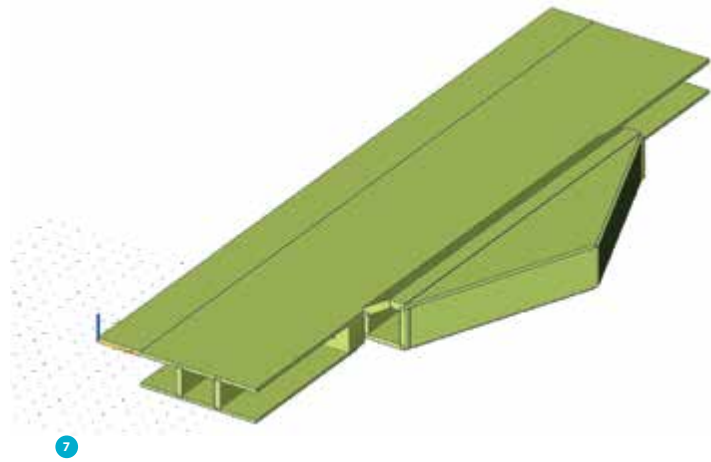
Finite element modelling

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
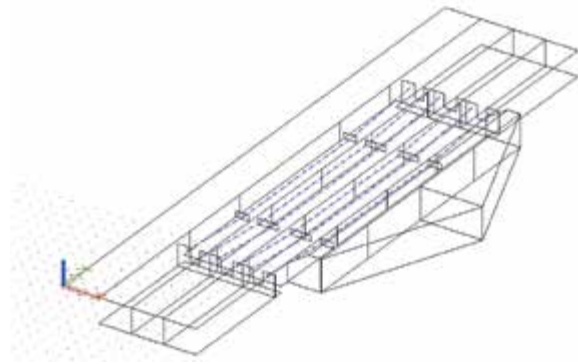
anchored to the existing girder webs (photo 9). Beside resisting loads during transient situation (prior to application of post-tensioning), this longitudinal reinforcement will also be active in persistent situation, together with the longitudinal post-tensioning.

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 mm², f_{yk} = 1860 N/mm²). 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-

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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.
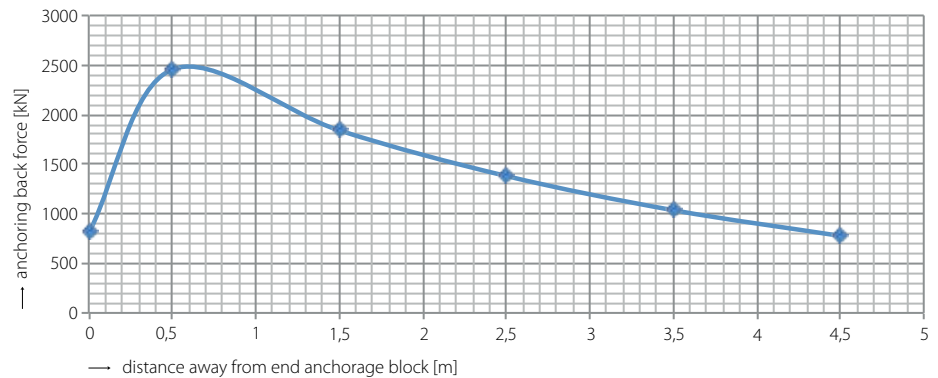
Located at the extremities of the tendons the end anchorage blocks 'nail' the longitudinal post-tensioning forces to the structure (photo 11). Because of the magnitude of these forces and the limited capacity of the existing structure, both vertical as well as horizontal anchoring of the blocks was necessary. 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)

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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.

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

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

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