First application of the balanced lowering method to build two bridges in Austria

Johann Kollegger | Dominik Suza | Clemens Proksch-Weilguni | Wolfgang Träger

TU Wien, Institute for Structural Engineering, Vienna, Austria

Abstract
This new bridge construction method consists of building the bridge girders in a vertical position and of rotating the bridge girders into the final horizontal position. In order to rotate the bridge girders, additional structural elements, in this case compression struts, are required. In some topographical situations, it will be more efficient to produce the bridge girders with this innovative method than with the traditional incremental launching method or the balanced cantilever method. The span of the bridge girders is reduced by the compression struts, which enables considerable savings in construction materials. This article explains the design and the first application of the balanced lowering method to build two bridges across the Lafnitz and Lahnbach rivers as part of the Fürstenfeld Motorway in the southeast of Austria. The good cooperation of the project participants allowed for the successful first-time application of the method for the two bridges described in this article. The unproblematic application of the balanced lowering method for the prestressed concrete bridge with 2.0 m high bridge girders proved that considerable savings in resources and costs were possible compared to the originally planned steel-concrete-composite bridges with 4.2 and 4.6 m high bridge girders.

KEYWORDS
balanced lift method, balanced lowering method, post-tensioned bridge, precast concrete elements

1 | FROM LOWERING OF ARCH HALVES TO THE BALANCED LOWERING METHOD

Lowering of arch halves was first implemented for the erection of reinforced concrete arches in 1955 by...
Riccardo Morandi during the construction of the Lussia Bridge (arch span of 70 m) in Italy, as described by Troyano. The most prominent and also largest application of this construction method was the erection of the Argentobel Bridge in Germany in 1984. The main advantage of this building technique compared to other arch construction methods using falsework or cantilever construction is the fact, that the arch halves are cast in an almost vertical position. This allows for a rapid construction progress using climbing or sliding formwork, as only low bending moments occur during the construction of the arch halves.

Lowering of arch halves is an established construction method for the erection of arch bridges and is applied regularly in Japan, Spain and other countries. Figure 1 shows a picture of the lowering process of an arch half during the construction of the Arnoia Bridge in Spain in 2012. Each of the two arch halves was approximately 70 m long and weighed 1100 tons. A tension force of 1100 kN was required to move each arch half from the vertical position to an inclined starting position for the lowering process. The cable elongation during the lowering of each arch half was 33 m and the maximum force in each of the two tendons was 2400 kN.

When construction methods for arches and bridge girders are compared, it is obvious that arches as well as bridge girders can be built using falsework or by cantilever construction methods. However, while there was a methodology for the lowering of arch halves, no counterpart existed for bridge girders.

The idea for the balanced lowering method arose when a construction method for bridges was sought which, based on the lowering of arch halves, begins with the installation of the bridge girders in a vertical position followed by a subsequent rotation into the final horizontal position. If the bridge girders are erected in a vertical position next to the pier, additional supporting elements (compression struts) are required in order to allow for a rotation of the bridge girders. To carry out the lowering procedure the underlying load-bearing system should be statically determined. In the final state of the bridge structure, the compression struts form an integral part of the load-bearing system, reducing the spans of the bridge girders significantly.

The patent application for the balanced lowering method was submitted to the German Patent and Trademark Office in 2006. The German patent was granted within 1 year. Patents for the novel construction method were subsequently granted in 18 further countries, including the USA, Japan, China, Russia, and India.

2 | DESIGN OF THE BRIDGES

The new S7 motorway “Fürstenfelder Schnellstraße” in the southeast of Austria requires crossings over the Lafnitz and Lahnbach rivers. The lengths of Lafnitz Bridge and Lahnbach Bridge are roughly 120 and 105 m, respectively. The bridge structures are very similar and each consists of two parallel crossings 14.5 m wide. The areas where the two bridges for the S7 motorway were built are ecologically sensitive and part of the “Natura 2000” nature reserve. The bridges were needed to cross the rivers and to provide options for a passage for deer.

The original design was based on steel-concrete-composite bridges with two spans. This choice was made due to the location of the bridges in the “Natura 2000” nature reserve prohibiting the use of any invasive building measures as for example the erection on formwork. During the construction, only limited areas next to the abutments and the central piers were accessible. The construction method of the steel-concrete-composite bridges with incremental launching would have required construction heights of 4.2 and 4.6 m for the Lahnbach Bridge and the Lafnitz Bridge, respectively. The alternative design was based on the construction of the bridges across the Lafnitz and the Lahnbach as two-span post-tensioned concrete bridges. The cross-section of each structure was designed as a double-T girder and a construction height of 2.0 m. The large difference in the height of the cross-section, 4.6 versus 2.0 m, is due to the compression struts, which reduce the spans of the post-tensioned concrete bridges, as can be seen in Figure 2.

The two spans of 57.85 m are each divided into spans of 37.65 and 20.20 m. The central part with a length of 72 m was erected with the balanced lowering method.
The gaps between the central part and the abutments were spanned using suspended girders.

Two cross-sections of the alternative design are shown in Figure 3. The thickness of the deck slab is equal to 0.4 m in the central part and equal to 0.25 m at the cantilevers. The width of the girders varies between 1.0 and 2.0 m. The height and the width of the compression struts are equal to 1.25 and 2.0 m, respectively.

Four separate bridge lowering operations were required for the construction of each bridge. The bridge girders and the compression struts were designed using thin-walled precast elements in order to keep the weight of the elements, which were to be rotated during the lowering process, as low as possible. The implementation of prefabricated elements did not only guarantee a fast construction process, but also contributed to the reduction of the construction costs as the prefabricated elements served as lost formwork.

The alternative design was developed with a high level of detail, taking all construction phases into account and calculating the exact layout of the tendons. Thereafter, the exact amount of building material (concrete, reinforcing steel, post-tensioning steel) required was determined and a construction description prepared. The accumulated documents for the construction of the bridge girders, the deck slab, the compression struts, and the central piers made it possible to calculate the construction costs for the Lafnitz and Lahnbach Bridges erected by the balanced lowering method and to compare them with the construction costs of the planned steel-concrete-composite bridges. This comparison showed that the construction of the bridge decks of the two bridges using the balanced lowering method would save approximately 30% of the costs. The positive result in favor of the alternative design can be ascribed to the savings in building materials and the use of concrete, which is a cheaper building material than structural steel. The reduction of the construction height of the bridge deck from 4.2 or 4.6 m (steel-concrete-composite bridges) to 2.0 m (post-tensioned concrete bridges) was achieved by reducing the spans in the alternative design. As already explained, the compression struts, which are necessary for the lowering process, form an integral part of the bridge structure in the final state and reduce the spans of the bridge deck. After seeing the alternative design and the calculations, the project management of ASFINAG Baumanagement GmbH decided to change the original design to one implementing the balanced lowering method and immediately placed a corresponding planning order.

3 | CONSTRUCTION STAGES

This section describes the construction stages of one girder of the bridges. Since there are eight girders in the two bridges, the individual construction stages shown in Figure 4 had to be carried out eight times.

Figure 4 shows a section in the longitudinal axes of the bridge girders and compression struts. Therefore, only one column of the two columns of the auxiliary pylon is shown in Figure 4. In order to facilitate the
understanding of the individual construction stages, drawings of the joints A, B, and C before and after the lowering process are shown in Figure 5. Joints A and B were standard steel constructions with radial spherical bearings and bolts. Joints C were equipped with concrete saddles, which acted as deviators for the tendons B. The lowering cables were attached to hinges in the joints C.

After the abutments and the central piers were constructed, the substructure for the auxiliary pylon was installed in construction stage 1. The substructure (height 7.8 m, steel weight 6 tons) of the auxiliary pylon was

FIGURE 4 Construction stages for the erection of one girder of Lafnitz Bridge (taken from Reference 7 and adapted; © Wiley & Sons)

FIGURE 5 Joints a, b, and c before and after the lowering process in construction stage 5 (taken from Reference 6 and adapted; © Wiley & Sons)
placed on the foundation of the central pier and connected to it using threaded rods and steel wedges. The auxiliary pylon, consisting of two columns built with Liebherr tower crane sections (185HC) with a height of 29 m, was then placed on the substructure. Two guiding rails were attached to the tower crane sections. In the next steps, the compression struts (length = 18 m, weight = 37 tons) were mounted and connected with bolts (diameter 100 mm, material quality Ck45) to the steel joints in the joints A, and subsequently fixated to the auxiliary pylon with tension belts. The compression struts consisted of wall elements, which were cast in a horizontal position, a bottom slab, and a deck slab, which were produced after tilting the wall elements. The thickness of the wall and slab elements was equal to 110 mm. A platform was then mounted on top of the auxiliary pylon, connecting the two tower crane columns and thus creating a frame effect in the transverse direction.

Due to local conditions, it was not possible to deliver the 36 m long bridge girders in one piece. The bridge girders therefore consisted of two parts—part 1 with a length of 19.5 m and a weight of 30 tons and part 2 with a length of 16.5 m and a weight of 22 tons. The first construction step of construction stage 2 consisted of lifting the first sections of the bridge girders and connecting them with bolts (diameter 90 mm, material quality Ck45) to the steel joints of the compression struts in the joints B. In the next step, the remaining parts of the bridge girders were lifted, set down on elastomeric bearings, aligned, and then connected with four threaded rebars (diameter 26 mm, B550B). The joint, with a planned thickness of 20 mm, between the two parts of the bridge girders was then filled with grouting mortar.

Once a minimum compressive strength of 5.0 N/mm² in the grouting mortar was achieved, construction stage 3 began with the tilting of the bridge girders. The tilting of the bridge girders caused only minor additional stresses in the grouting mortar, because the major part of the dead weight of the bridge girders 2 was still carried by the elastomeric bearings. In the next step tendons type A were coupled and tendons type B, (four tendons with four monostrands St 1860 each) were installed. The tendons type B, shown in green color in Figure 4, were anchored in the joints B and guided over a saddle (radius 0.75 m) in joints C. After installation, the individual monostrands of the tendons were tensioned with a force of 2 kN each, connecting the two bridge girders to each other. In this construction phase, the entire structure tended toward a system closing behavior, where not even high wind forces were able to trigger an uncontrolled lowering process. In the next construction step, two strand-lowering units and two lowering cables with four strands each, shown in blue color in Figure 4, were installed and anchored in joints C for the following lowering process.

In construction stage 4, the compression struts were pushed aside apart by two hydraulic jacks positioned at the top of the central pier until the system closing behavior of the structure was overcome. While the compression struts were being pressed apart, the two lowering cables, were adjusted and kept under tension. Horizontal compression forces of approximately 60 kN were required to trigger the lowering process. These horizontal compressive forces became smaller as the inclination of the compression struts progressed and were equal to zero at a tilting angle of 2° (measured at node C with respect to the vertical axis) corresponding to a stroke of 320 mm of each horizontal jack. From this point on, the static equilibrium of the structure was ensured by the tensile forces in the lowering cables (construction stage 5).

In construction stage 5, the static system consisted of two single span beams with cantilevers and two three-hinge frames—one on the left side and one on the right side of the auxiliary pylon, as is shown in Figure 6. Each bridge girder had the static system of a single span beam with a cantilever. Each bridge girder was supported at the hinge of joint B and at the hinge of the joint C, where the lowering cable was attached (see Figure 5). The larger part of the weight of the bridge girders was transferred to the three-hinge frames in the joints B and the remaining part of the weight was carried by the lowering cables.

Half of the weight of each compression strut was carried by the three-hinge frame at joint B and the other half was taken by the hinge in the joint A. Each three-hinge frame consisted of a compression strut and the tendons B, which carried only tensile forces. Tendons B were in contact with the bridge girders only at the anchorages of the joints B and at the concrete saddles of
the joints C. The deviation forces of tendons B at the concrete saddles were transferred within the joints C to the hinges, which were attached to the lowering cables. The vertical reaction forces at the joints C were carried by the lowering cables. The horizontal forces at the joints C were carried by the tendons B.

During the lowering process, horizontal forces due to wind and weight differences of the structural elements were transferred at the joints C to the guiding rails mounted in the auxiliary pylon. After a lowering distance of 26.5 m, the bridge girders were placed onto roller bearings (diameter 50 mm) on the central pier and the lowering cables were released. The planned longitudinal inclination of the bridge of 0.6° was achieved by a horizontal shift of the bridge girders in joints C. For this purpose, the guiding rails were equipped with removable lining plates. The actual movement was carried out by screwing two M24 screws. The exact alignment of the bridge girders was achieved by tightening the tendons type B.

In construction stage 6, the type I temporary tendons (four monostrands each) and type II temporary tendons (8 monostrands each) were installed, anchored and tensioned to the required forces using hydraulic jacks positioned on the platform at the top of the auxiliary pylon.

The compression struts were then symmetrically filled with self-compacting concrete in two phases. The concreting process was carried out from the bottom up, through filling openings at the joints A and approximately in the middle of the compression struts, in order to avoid voids in the in-situ concrete. After that, the volumes of the bridge girders around the joints B were filled up to a height of 1.0 m.

The installation of the 21.95 m long prefabricated suspended girders connecting the part of the girder which had been erected by the balanced lowering method with the abutments, was carried out with mobile cranes positioned behind the abutments. Prior to the installation of the suspended girders, the forces in the temporary tendons were adjusted so that tensile stresses would not occur in the bridge girders. After installation of the suspended girders on the cantilever ends of the bridge girders and the abutments, the 20 mm wide grouting joints were filled with mortar. Two type C1 tendons, each with 19 strands, were then installed throughout the entire length of the structure. In construction stage 7, the type C1 tendons were considered as external tendons, which were arranged at the deflection points in plastic ducts. Once the mortar had reached a compressive strength of 15 N/mm², the tendons were stressed to 250 kN each, creating a rigid connection in the grouted joints. Following installation and stressing of the type III temporary tendons (7 strands), additional supports were available to ensure a stable system for placing the in-situ concrete (construction stage 8). The U-shaped prefabricated elements forming the lost formwork of the girders were filled with two layers of in-situ concrete. The first 0.5 m thick layer consisted of two pours of 0.25 m each. The second 1.18 m thick layer consisted of three pours (Lahnbach Bridge) and four pours (Lafnitz Bridge), which were installed when the first layer had reached a compressive strength of 15 N/mm². In construction stage 9, the temporary tendons were released and the platform at the top of the auxiliary pylon removed. The auxiliary pylon, was then dismantled and reinstalled at the next central pier. Construction stages 1–9 were then repeated to build the next girder of the bridge.

### 4 PRODUCTION OF THE THIN-WALLED PRECAST ELEMENTS

The production of the bridge girders, the compression struts and the suspended girders was carried out in a prefabrication plant located at a distance of around 150 km from the construction site. For the production of one girder of the Lafnitz Bridge and the Lahnbach Bridge (see Figure 4), two compression struts, two bridge girders each consisting of two parts, as well as two additional suspended girders—resulting in a total of eight precast elements—were required. For the four girders of the bridge across the Lahnbach and the four girders of the bridge across the Lafnitz, which was subsequently built using the same construction method, 64 precast elements were produced. The 16 compression struts had identical dimensions and reinforcement arrangements. The dimensions of the bridge girders were also the same for both bridges. The only difference in the girder production was the tendon positioning and the amount of reinforcement needed.

Thin-walled slab elements produced using industrial construction methods have been successfully used in building construction and industrial buildings for decades. These prefabricated elements with a thickness of 50 to 70 mm serve as lost formwork for the in-situ concrete that is subsequently cast. For the bridge girders and the suspended girders, with their U-shaped cross-sections, 70 mm thick thin-walled slab elements were used as the side walls.

In the left part of Figure 7, the production of the side walls can be seen. The side walls were cast in a horizontal position with a height of 1.8 m and lengths between 5.5 and 8.0 m. A detailed view of the reinforcement of the side walls is shown in Figure 8. The longitudinal reinforcement consisted of 10 mm diameter rebars spaced at
100 mm. For the vertical reinforcement (in the final position) rebars with a diameter of 12 mm spaced at 150 mm were used.

Lattice girders spaced at intervals of 450 mm were placed on top of the horizontal reinforcement in order to ensure a good connection between the side walls and the in-situ concrete. Rebars with a triangular shape (diameter 12 mm, spaced at 150 mm) were positioned at the bottom of the side walls in order to enhance the stiffness of the connection of the side walls and the bottom slab with a thickness of 120 mm.

At several locations on the bridge girders and the suspended girders, cross-beams—spanning from one side wall to the second side wall—were required, in order to provide deflection saddles for the C1 longitudinal tendons and anchorage points for the temporary tendons used in construction stages 6–8. These cross beams were fixed to the side walls with ladder reinforcements which consisted of three 8 mm diameter rebars which were welded to two transverse rebars (8 mm) to provide a good anchorage in the concrete of the side walls. Only 40 mm was available for the anchorage, since the concrete cover, which was determined according to Eurocode 2, in the side walls was equal to 30 mm. The ladder reinforcements, placed orthogonally to the side wall, can be seen in Figure 8.

The right part of Figure 7 shows the production of the bridge girders. Usually four bridge girders or suspended girders were produced simultaneously. The side walls were placed on a steel shuttering, where they were fixed using steel profiles welded to the surface of the shuttering. Figure 7 shows a photo of a type 1 bridge girder after placing the stirrups and the longitudinal reinforcement in the bottom slab. In the foreground of the photo, ladder reinforcements can be seen protruding out
of the side walls. This ladder reinforcement was used to connect the cross beam—located at the tip of the cantilever of bridge girder type 1 (see construction stages 2–6 in Figure 4)—to the side walls. In the cross beam, which can also be seen in Figure 7, the type II temporary tendons were anchored and stressed in construction stage 6. The ducts of the two external C1 tendons were placed below the cross beam (construction stage 8 in Figure 4). The ducts for the other 19-strand tendons were installed in the precast girders at the precast plant (Figure 9).

The cross beam shown in Figure 7 had the following two additional functions: (1) as the anchorage of two monostrands (the anchorages can be seen in Figure 7), which were tensioned during construction stage 2, connecting the two sections of the bridge girders to each other and (2) to act as a support for the subsequently installed suspended girder connecting the balanced lowering part of the bridge to the abutments (construction stage 7).

At the upper end of the side walls, transverse reinforcement bars (diameter 20 mm) were welded to the top chord of the lattice girders. A truss bracing was formed using 12 mm diameter reinforcement bars installed on top of the 20 mm diameter reinforcement bar. This construction approach, which increased the stiffness of the girders during transport and assembly, can be seen in Figure 11.

5 ASSEMBLY OF THE PRECAST GIRDERS AND LOWERING PROCESS

The assembly of the precast girders and the subsequent lowering process were carried out as shown in construction stages 1 through 5 of Figure 4. Figure 10 shows a photograph of the Lahnbach Bridge in construction stage 2. The tender documents specified that all work steps from mounting of the type 2 bridge girder (construction stage 2) to the completion of the lowering process (construction stage 5) could only be carried out when wind speeds were lower than or equal to 12 m/s. Construction stages 2–5 could be completed within 48 h. With the help
of site-specific weather forecasts, the feasibility of these wind-dependent work steps was assessed in advance. During the construction of the bridges across the Lahnbach and the Lafnitz rivers, the wind conditions were favorable, allowing for timely completion of all eight lowering processes.

After the tilting of the bridge girders, the type A tendons were coupled and the type B tendons were installed in construction stage 3. A detailed view of a bridge girder in the tilted position after the installation of the B tendons can be seen in Figure 11. The joint between bridge girder 1 and bridge girder 2 is located in the lower part of the photograph. The couplers of the A tendons, which consisted in total of four monostrands, are marked in Figure 11 with red circles. The type A tendons were required to ensure compressive stresses in the joints between the bridge girders 1 and 2 during the lowering process. The tendons were stressed after tilting of the bridge girders in construction stage 3. Figure 12 shows the upper end of the bridge girders (joints C) after installation of the four type B tendons. Two tendons, with four monostrands each, were attached to the steel cross beam (cross section 100 mm by 140 mm) using compression fittings. The other two tendons, with a total of eight strands, were installed through recesses in the steel cross beam. Concrete saddles in joints C allowed for the attainment of the required curvature of the monostrands in this construction stage. The steel cross beam was equipped with lining plates at both ends so that it could be installed in the guiding rail with a lateral distance of 20 mm. Radial spherical plain bearings were installed at the end areas of the cross beam, allowing for a lowering without much friction in the guiding rail. Figure 12 also shows the four anchorages of the monostrands of the type A tendons in each bridge girder and the ducts for C1 type tendons. The bridge girders had lifting points in joints C, which were used during transport and the vertical installation in construction stage 2. The U-shaped steel structures, which were fixed to the lifting points with a 60 mm diameter bolt, were used to hold the anchor heads for the lowering cables (4 strands each) are also visible in Figure 12.

During the lowering process the tips of the bridge girders moved away from the steel cross beam, because of the increasing strain in the type B tendons and the separation of the monostrands from the saddles in joint C. This scenario can be seen in Figure 13.

The actual lowering process was carried out within 3–4 h. During the entire time, special care was taken to ensure that the horizontal distances of the cantilever tips
from the central axis only varied within 200 mm on each side.

Figure 14 shows the last lowering process of Lafnitz Bridge after a lowering of joints C by approximately 5 m. During one of the lowering processes of Lafnitz Bridge, the forces in the two lowering cables were measured with load cells. The measured forces in the lowering cables are shown in Figure 15. As described above, the forces in the lowering cables are equal to zero until the compression struts were pushed apart by 320 mm with the aid of two hydraulic jacks mounted on top of the central pier (construction stage 4 in Figure 4). At this point the statical system changed from a closing behavior to an opening behavior and the forces in the lowering cables increased rapidly to a maximum value of 290 kN. The forces in the lowering cables remained approximately constant after lowering joints C by 5 m. The forces in the lowering cables became gradually smaller at the end of the lowering process. When the joints C had been lowered by 26.5 m, the forces in the lowering cables reached the calculated value of 50 kN for the horizontal position of the bridge girders.

A close-up view of joint B during the lowering process is shown in Figure 16. The steel plates of joint B were fixed to the girder by welding them to steel plates with welded-on reinforcing bars which were installed in the compression struts and the bridge girders. The transition of the bridge girder from the vertical starting position to the horizontal end position, required a relative rotation of 155° between the compression struts and the bridge girders in joint B. The maximum compression force of 1600 kN in the compression struts did not occur until the final state was reached.
In the structural design phase, distances of 20 mm between the supporting elements (bridge girders, compression struts, central pier, and temporary tendons) during assembly and 50 mm between the structural elements that rotated relative to each other during the lowering process (e.g., in joint B) had been planned. These inaccuracy precautions were actually too conservative due to the high degree of accuracy in production and assembly of the precast elements, resulting in deviations from the theoretical target positions of less than 10 mm and in most situations of less than 5 mm.

As previously mentioned, almost all the required reinforcement had already been placed in the bridge girder at this point, since its placement on the construction site after lowering—instead of in the precast plant—would have been far more intricate, time-consuming and expensive. The only installation of reinforcement required on the construction site took place in the connection areas in the joints between the individual elements and at the connection between the bridge girders and the subsequently mounted suspended girders forming the connection to the abutments. This decision resulted in a very high reinforcement content in the thin-walled bridge girders of up to 500 kg reinforcing steel per cubic meter of concrete. The high reinforcement ratio was explained in the tender documents and did not cause any problems during the construction of the bridges. In all the calculations for transport and assembly, as well as for the lowering process, the specific weight of the reinforced concrete was therefore set to 29 kN/m³.

6 | CASTING OF THE IN-SITU CONCRETE AND CONSTRUCTION OF THE DECK SLAB

After aligning the bridge girders in their final position, which was carried out with a deviation of ±3 mm from the nominal position, joints A and C were filled with concrete, thus fixing the geometry of the bridge. Thereafter, the type I and II temporary tendons were installed and the compression struts and joint B were filled with concrete. In construction stage 7 (see Figure 4), the additional suspended girders needed to span to the abutments were mounted, the grouting joint between the newly added girders and the bridge girders was filled, and the temporary type III tendons, along with type C1 tendons were installed. The in-situ concrete for the girders was cast in two consecutive days with concreting heights of 0.5 m on the first day and 1.18 m on the second. Figure 17 shows the placing of the in-situ concrete, which was carried out symmetrically to the central pier using two concrete pumps. The entire on-site concreting work was carried out for all eight girders, with the width varying from 1.0 to 2.0 m and a height of 1.80 m, without any problems and within a few hours for each casting operation. Some parts of the thin-walled girders were subjected to the maximum allowable compressive stresses, which is equal to 60% of the characteristic concrete stresses according to Eurocode 2 (24 N/mm² for concrete C40/50), during the placing of the in-situ concrete in the construction stage 8. The redistribution of these high compressive stresses to the hardened in-situ concrete due to creep effects was considered in the design.

The deck slab was constructed in a similar way to a steel-concrete-composite bridge using a formwork carriage (Figure 18). The formwork carriage was fixed to the construction with threaded rods, which had been installed using templates in the second layer of the filler concrete of the girders. The length of one casting section was specified as 15 m.

The incremental construction of the girders, the compression struts, and the deck slab was considered in the
design of the bridges. Calculations considering creep effects were carried out with the aid of a finite element program.

7 | CONCLUDING REMARKS

With the construction of the bridge across the Lafnitz and the Lahnbach rivers, it was proven that the balanced lowering method allows for considerable cost savings under suitable topographical boundary conditions, compared to a conventionally constructed bridge using for example, incremental launching. The costs for the construction of the bridge decks (using the balanced lowering method), the compression struts, and the central piers were determined by the first author. Based on the quantities of the construction materials, a detailed method statement, and offers obtained from construction companies a price of € 4,700,000,- was calculated and communicated to the owner ASFINAG. The costs for the originally planned steel-concrete-composite bridges (bridge deck and central piers) were obtained from the quantities of the required construction materials and a comparison with tender prices of other similar bridges built for ASFINAG in the recent past. A price of € 6,800,000,- for the erection of steel-concrete-composite bridges over Lafnitz and Lahnbach was determined. The best offer for the construction of the bridge decks, the compression struts and the central piers received during the bidding process was equal to € 4,840,000,- and thus only 3% higher than the estimated amount. Savings of € 1,960,000,—which amounts to 29% of the € 6,800,000,—for the originally planned steel-concrete-composite bridges, could be achieved. As was already mentioned before, the main reason for the savings of almost 2 million Euros was the reduction of the quantities of the construction materials due to the action of the compression struts. In case of the Lafnitz Bridge the spans of 57.85 m were divided into spans of 37.65 and 20.2 m (see Figure 2).

The balanced lowering method also offers advantages for the construction of replacement structures, due to the fast construction and vertical assembly possible—even in confined spaces. An example with suitable boundary conditions for the application of the balanced lowering method is shown in Figure 19. For the construction of this three-span bridge with piers of modest height, the compression struts and the bridge girders are assembled on both sides of the center span. Once the two lowering processes have been carried out, the valley is spanned by the connection of the two lowered bridge girders resulting in a large center span. The technically reasonably range of application of such bridges is 50 to 100 m for the main span.

The balanced lift method is a very similar bridge construction method, which is particularly advantageous for bridges with high piers. In the balanced lift method, the top points C of the bridge girders are fixed to the top of the pier and the lower points A of the compression struts are lifted in order to rotate the bridge girders (see construction stage 4 in Figure 4). This mechanism is similar to the opening of an umbrella. The balanced lift method is applicable for bridges with spans from 50 to 200 m. As in the balanced lowering method, the compression struts reduce the span and therefore allow considerable resource savings.

The research work, which was required to develop the balanced lowering and balanced lift methods, is described in References 7–9.

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DATA AVAILABILITY STATEMENT

Data sharing not applicable to this article as no datasets were generated or analysed during the current study.
ORCID
Clemens Proksch-Weilguni https://orcid.org/0000-0002-7465-1346

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AUTHOR BIOGRAPHIES

Johann Kollegger, TU Wien, Institute for Structural Engineering, Karlsplatz 13, E212-2, Vienna 1040, Austria. Email: johann.kollegger@tuwien.ac.at

Dominik Suza, TU Wien, Institute for Structural Engineering, Karlsplatz 13, E212-2, Vienna 1040, Austria. Email: dominik.suza@tuwien.ac.at

Clemens Proksch-Weilguni, TU Wien, Institute for Structural Engineering, Karlsplatz 13, E212-2, Vienna 1040, Austria. Email: clemens.proksch-weilguni@tuwien.ac.at

Wolfgang Träger, TU Wien, Institute for Structural Engineering, Karlsplatz 13, E212-2, Vienna 1040, Austria. Email: wolfgang.traeger@tuwien.ac.at

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