



# LEAP OF FAITH

The end piers of the arch mimic the slopes of the Tamina Valley, highlighting the structure's harmonious integration with its natural surroundings

The biggest arch bridge in Switzerland is due to open to traffic this year. **Holger Haug** reports on its design and construction

**T**ravelling between two communities in St Gallen will become easier and safer this year, when a new bridge over the Tamina Gorge opens to traffic. Work on the dramatic concrete arch bridge, which crosses the valley at a height of 200m, has taken just four years to complete - one year less than originally anticipated.

The Tamina Valley forms the southern tip of the canton of St Gallen, ending at the spa town of Bad Ragaz in the alpine Rhine valley. Both sides of the Tamina Valley are accessible by roads from the centre of Bad Ragaz, rising in a series of bends on the steep, forested slopes. The road which leads to Valens is in a bad condition; it passes through an active landslide area and due to the related geological risk no longer meets the required standards. The communities of Bad Ragaz and Pfäfers carried out investigations which demonstrated that a bridge over the valley would be a safer and cheaper solution than the periodic rehabilitation and maintenance of the road to Valens. The chief civil engineer of the canton of St Gallen launched a design competition in 2007 with the intention of finding the best solution for a bridge over the valley.

The competition jury said that Leonhardt, Andrä & Partner's winning design, which proposed an asymmetrical arc across the valley, took into account both the local conditions and the sensitive landscape. In 2009 and 2010 the preliminary design was developed by Leonhardt, Andrä & Partner working with geotechnical specialist Smoltczyk & Partner. The project was approved by the local authority in 2012 and LAP continued the design in collaboration with Swiss consulting engineer DSP Ingenieure & Planer. The contract was advertised for tender the same year.

The joint venture Taminabrücke - formed of companies Strabag, Glattbrugg, J Erni, Flims Dorf & Meisterbau and Balzers - was awarded the contract in December 2012 by the canton of St Gallen. The contractor's alternative construction method reduced the programme from five to four years while still costing the same as the tendered concept. Detailed design was also carried out by Leonhardt, Andrä & Partner. In the first step the implementation of the existing design document of the construction concept proposed

by the joint venture was carried out, and Leonhardt, Andrä & Partner checked the designs of all auxiliary works and supervised the erection of the falsework on behalf of the canton of St Gallen.

The structural system of the Tamina bridge is formed mainly by the arch and superstructure which create a continuous prestressed girder. The superstructure is connected monolithically to the arch by inclined columns and is 417m long; including the abutments, the total length of the structure is 473m and road width between the concrete barriers is 9.5m. The arrangement of the columns on the arch and the higher piers leads to spans of 45m to 60m. In the area of the arch crown, the superstructure and arch combine into a single element over a length of approximately 57m.

On both sides the valley falls steeply from different levels so with the asymmetrical arch the foundations could be placed above the steeper slopes over the valley sides. The sound rock formations in these places are suitable for arch loadings. At an early stage in the design it was recognised that construction of the arch foundation on the Valens side would be very difficult, so engineers decided to adjust the gradient on this side slightly to allow the foundation to be placed in a flatter and more accessible location.

The arch is designed as a reinforced concrete structure using concrete class C45/55 and spans 260m across the Tamina gorge. It has fixed connections at the foundations where the depth is 4m on the Pfäfer side and 3.2m on the Valens side and this depth decreases continuously to just 2m at the arch crown. The width of the arch member varies from 9m and 7m at the foundations to 5m at the crown. Most of the arch cross-section, roughly from the foundations to the short central columns, is a hollow box in order to save weight. In the crown area where the depth is smaller, a solid cross-section was chosen. The thickness of the flanges of the box is a maximum of 1m and it decreases continuously towards the crown down to 500mm, depending on the structural requirements. The webs are a constant width of 900mm, and this was due to the dimensions of the cable anchorages needed for the temporary stays.

To reduce the length of the end spans, the piers between the arch foundations and

deck are not vertical, but rise approximately perpendicular to the arch. Because of this, their inclination mimics the slope of the valley walls, and this was one of the aspects of the design that was highlighted for its harmonious integration with the natural surroundings. Along with the superstructure, the inclined piers form a rigid frame structure with the deck as a frame member and piers as the frame stem - to allow the deck to span the valley sides without any piers, for a more aesthetically-pleasing and economical result. In cross-section the width of these piers increases towards the foundation on Pfäfers side from 5m to 7.1m and on Valens side from 5m to nearly 6m. In elevation they widen significantly towards the top where they connect to the deck, clearly emphasising their function as part of the frame structure, especially in distinction to the pinned arch columns. The cross-section of these two piers is mostly a hollow box with walls of 400mm and 500mm thickness. Only the lower parts are designed with a solid cross-section.

In the axes of the three intermediate piers the superstructure is supported on slender pinned columns made of self-compacting concrete grade C45/55. In section view with variable widths these match both the arch and superstructure dimensions. These columns are provided with concrete hinges at both ends to reduce constraint forces in the arch and superstructure due to stresses acting in plan of the structure. The quality of the self-compacting concrete in the area of the concrete hinges was tested with the help of a full-size test specimen which was cut open after the concrete had cured.

The superstructure is designed as a continuous prestressed concrete girder which is 5m wide and has a typical depth of 2.75m. This increases towards the two inclined piers on the foundations which also emphasises its function as frame member. At mid-span the bottom slab is 300mm thick; this increases to just over 2m at the piers on the foundations, to meet structural requirements. At both ends the superstructure is supported by two spherical bearings, of which one at each side is also fixed in the transverse direction. In the end spans, close to the abutments, the superstructure is curved in plan and the resulting additional torsional moments are carried by the box to the bearings and transferred to the bridge piers by a coupling force. To avoid uplift forces at the side of the bridge seat, cables are arranged which are prestressing with a total force of up to 8.7MN to compensate the tensile forces in the bearings.

The width of the webs is 550mm, which is sufficiently wide to accommodate two ducts side by side in one layer. At the intersection with the webs the top slab is 500mm thick and this reduces at the centre to 300mm, and 350mm respectively to allow the installation of tendons. Webs and top slab are widened for the arrangement of the tendon anchorages, especially at the ends of one segment.

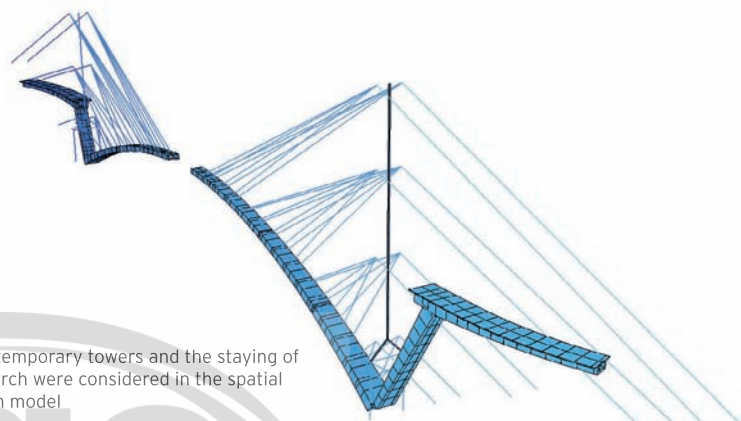
The structural analysis was based on a spatial beam model; for this process the temporary towers and the staying of the arch also had to be considered in the model. For the erection of the structure more than 50 different construction stages had to be investigated, starting with the free-cantilever method, followed by the dismantling of the temporary stays and finally the span-by span construction of the deck by the use of falsework supported on the arch. The stresses due to dead load and prestressing are the result of the stage-by-stage analysis and these results were considered in the design at final stage.

The design was based on Swiss standards with additional local measurements for wind load taken into account. Despite the exposed position of the bridge structure in the valley, these measurements confirmed that it was sufficient to use the normative reference value for wind pressure given with  $q_0 = 1.3\text{kN/m}^2$  for Bad Ragaz. Further aerodynamic investigations proved this approach and confirmed that it was not necessary to increase the wind loads in the design.

In the preliminary design stage, during analysis of the global system, special attention was paid to the design of some local areas, such as the foundation springings, the frame corner between the inclined piers and the deck, and the intersection in the crown area between the arch and the deck. In the later design phases, the main dimensions from the competition design could be confirmed and structural details such as the design of the concrete hinges or the bearing layout could be further optimised.



The temporary cables typically supported the arch every second segment



The temporary towers and the staying of the arch were considered in the spatial beam model

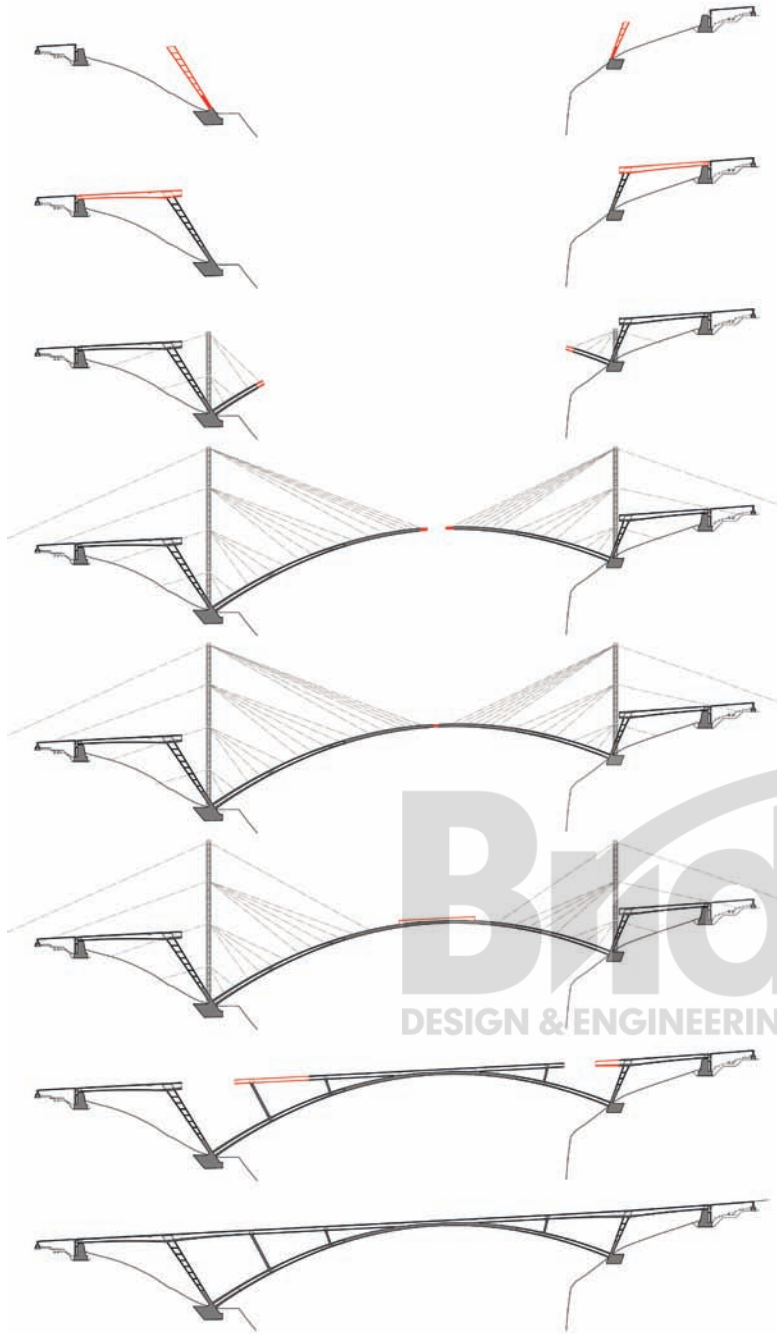
In addition to the analysis of construction and final stages, the analysis of special topics was based on discrete structural models. For the analysis of the superstructure in the transverse direction and its behaviour under torsional stresses, for example, a 40m-long section of the superstructure was modelled using finite elements. Also extraordinary load cases such as seismic events, either during construction or in the permanent state, or the loss of a temporary stay, were verified in the design.

The tender documents included details of the designer's construction concept, which proposed erection of the side spans using ground-based falsework. Subsequently the arch would be erected from both sides by the free cantilever method with temporary stays and steel towers on the frame corner of the side spans. By using the existing auxiliary devices, this procedure was planned in greater detail.

During tender the contractor was given the option to propose an alternative construction concept. The joint venture also proposed to erect the arch by the free-cantilever method with temporary stays, but with the temporary towers placed directly on the foundations, with tower legs on both sides of the inclined piers instead of on the approach bridges. The steel towers were approximately 105m tall on Pfäfers side and 80m on Valens side; significantly taller than those in the original construction plan in the tender. Some 820t of steel on the Pfäfers side and 520t of steel on the Valens side was used for the temporary towers, including the cross-beams for the cable anchorages and the working platforms. The contractor proposed to erect the superstructure in segments on falsework supported by the arch.

The temporary cables supported the arch at short intervals, typically every second segment. On the back span, the cables were split into groups and anchored directly into the ground, rather than using a single anchorage to take the entire vertical cable forces. These temporary stays were coated mono-strands of steel grade St1680/1860, each strand having a cross-sectional area of 150mm<sup>2</sup>. Standard tendon anchors were used to connect them into the webs; the main span cables were coupled to the anchor by short strand pieces with single-strand couplers. At the cross-beams the cables were connected using specially produced anchor elements; these were also used to connect the back-span cables at the anchor blocks.

## REPORTS



More than 50 different construction stages were investigated in the model

► Stressing of the cables was force-controlled but simultaneously their elongation was monitored. The concrete arch was erected in parallel from both sides: 32 segments from Pfäfers and 23 from Valens. Once the contractor had become familiar with the construction procedure, up to 5m of arch could be formed, reinforced and concreted per week on both sides.

Erection of the superstructure started with the first segment at the crown of the arch, followed by four segments on each side up to the connection with the side spans. To minimise stresses in the arch during the erection process, corresponding segments were erected by stages: the trough first on both sides and then the top slabs one after the other. Erecting the superstructure on falsework using segments of 30m to 40m length proved cheaper and quicker than the free-cantilever method. Moreover, there were fewer construction joints, which will improve the structure's durability.

The organisation of flexible construction procedures and the use of standardised falsework elements offer a strong argument for the contractor's proposed method.



Temporary supports enabled the side spans to be built as the towers were erected

Instead of having to build the bridge elements in series, with this method the work was able to be advanced in parallel, where possible. With the steel towers placed directly on the foundations, the side spans could be built at the same time as the towers were erected. And the temporary towers were extended in height as the arch construction proceeded across the valley.

With the temporary towers placed on the springing foundations they became significantly higher than they would have been if placed on the frame corner of the side spans but in doing so the horizontal compression struts beneath the superstructure could be omitted. The valley-side oriented force component in the inclined piers became smaller and the transfer to the ground required fewer rock anchors.

The scaffolding designed by the contractor used standard elements as much as possible, with only a few elements being custom made; in the end only the cross-beams at the towers were made of welded steel plates. These cross-beams were the anchor point for all the temporary cables, which all had different inclinations and had to be stressed up to 2.2MN. The main advantage of the contractor's alternative was the elimination of the horizontal compression strut, an expensive individual construction that was intended to carry a compressive force of up to 135MN. Instead the excellent foundation conditions were used. The back-span cables had smaller forces and were anchored directly into the rock using standard structural elements.

Although the cost of the falsework in the contractor's alternative was the same as the cost of the procedure in the tender, the alternative enabled the construction programme to be reduced by a year. With the easier execution of the elements for the scaffolding and fewer individual parts, the higher costs of installation due to difficult site access roads for the heavy lifting equipment was more than compensated.

The structural system of the Tamina Bridge was a very flexible one during the construction of the arch. In order to achieve the designed gradient for the carriageway, the deformation of the system had to be compensated by a pre-cambered erection, with values analysed from the structural system. In the calculation process, the exact construction sequence was considered and in order to reliably analyse the time-dependant behaviour of the concrete, the actual construction time schedule was also represented.

These camber values made it possible to correctly install the temporary falsework and carry out necessary elevation controls. The designer provided target values for the arrangement of the form travellers and geometry controls and the actual position of the arch was surveyed after completion of each segment. Any misalignment was corrected by adjusting the position of form travellers for the following segment. To eliminate any temperature effects on the surveying, relative coordinates were used.

In principle the semi-integral construction method does not allow for adjustments of deviations from the pre-camber, but the construction method and structural system of the Tamina Bridge did have this option to a certain extent. For example, after completion of the arch for the intermediate columns, the lengths were fixed and so discrepancies in the arch's geometry influenced the elevation of the carriageway.

However the extreme flexibility of the concrete arch was one of the biggest



Construction of the superstructure began with the first segment at the crown of the arch

challenges for the site. During erection of the last segments on each side, after concrete was poured the tip of the arch displaced downwards by approximately 700mm and when the form traveller moved to the next position another 300mm had to be added. Only at the end of the construction process for one segment, when stressing the front and back-stay cables the tip of the arch was lifted by 1m.

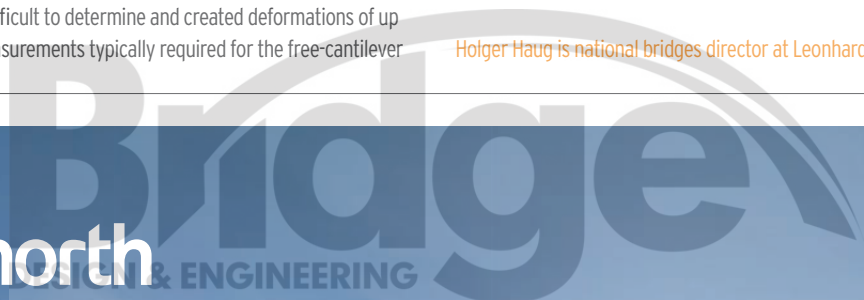
Between two stressing operations the top of the temporary towers deviated from the vertical by up to 350mm. But these deformations resulted from permanent loads which could be handled, in contrast to variable loads from wind or temperature effects such as solar radiation. These were more difficult to determine and created deformations of up to 100mm. Hence the reference measurements typically required for the free-cantilever

method were done early in the morning before sunrise. Stressing of the cables was done in 20-30 carefully detailed steps and was permanently monitored by surveying the deformation of the arch and the tower during the operation. The tip of the arch was then adjusted according to the relative coordinate for that stage. Consequently when the closure pour took place, both sides of the arch were at the designed elevation: the Pfäfers side was exactly in place and the Valens side was just 50mm above the calculated level so it could be adjusted by reducing the cable force and using the form travellers and jacks

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Engineering and Project Management



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