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特別講演 I

SOME INNOVATIVE PRESTRESSED CONCRETE STRUCTURES IN SWITZERLAND

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1. INTRODUCTION

This contribution presents a number of selected examples showing the potential of prestressed concrete construction. These examples allow understanding how prestressing can play a significant role in defining the shape of a structure and in configuring its structural behaviour.

The works presented herein have been part of the consultant activity of the author in Switzerland since 1987 (conceptual design). Although they are not necessarily connected between them, they all express a search towards efficient, light and slender structures. They also show that developing innovative or unusual shapes should also take advantage of previous experiences, both personal and of other designers, which have to be critically revised with creativity and rigour to adapt them to the actual requirements of each project.

2. TWO BRIDGES WITH HIGHLY ECCENTRIC EXTERNAL TENDONS

2.1 The evolution from the underspanned beam to the bridges with highly eccentric external tendons

In the beginning of the 19th century, it was found that the length that could be spanned by simply supported beams could be increased by means of an eccentric external tension chord (leading to the so-called underspanned beams, Fig. 1a [1]). These systems were quickly adopted in bridge design by increasing the number of deviation struts, leading to the underspanned suspension bridges [2]. This evolution from the classical suspension bridges allowed removing the need for pylons supporting the tension cable (Fig. 1b) and showed an outstanding efficiency for carrying the permanent loads of the structure.

Nevertheless, as for conventional suspension bridges, the new system was particularly sensitive to concentrated or variable live loads, experiencing potentially large deflections. A solution to this problem was developed by providing the deck slab with sufficient bending stiffness to distribute the influence of concentrated loads on the tension member. Although this approach was very reasonable for long spans (where dead load is more important), it reduces the efficiency of light bridges with small span lengths. More suitable solutions were thus sought for these cases. One approach consisted on superimposing various underspanned beams, leading to the so-called Fink's beam (Fig. 1c).

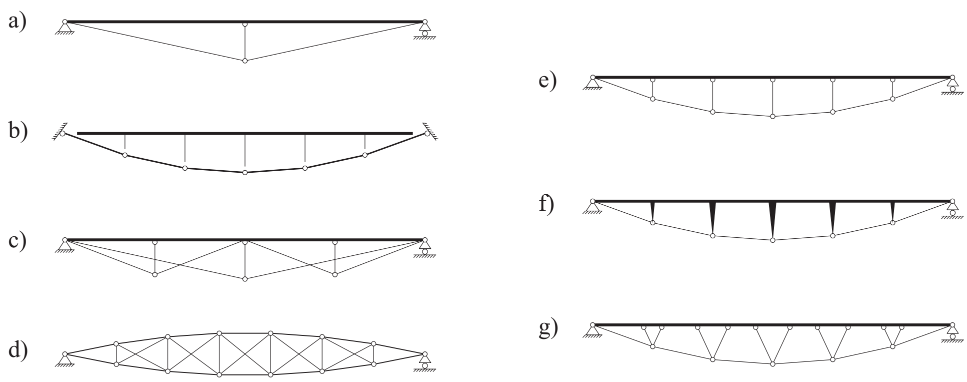

Fig. 1: Underspanned simply supported systems for bridges: (a) simple underspanned beam; (b) tension member anchored in the soil; (c) Fink's beam; (d) fish-bellied girder; (e) underspanned beam with hinged deviation members; (f) underspanned beam with deviation members clamped at the deck; and (g) underspanned beam with V-shaped deviation members (adapted from [2])

Other developments oriented more to truss members, as the fish-bellied girders (Fig. 1d). Amongst these systems, where the compression forces were carried by a steel compression chord, trusses showed the highest efficiency and gradually dominated the structural solutions for bridges, leading to the abandon of underspanned beams in the second half of the 19th century [2].

Nevertheless, underspanning was used as stiffening of falsework or for strengthening of existing bridge girders. Another interesting application of underspanned beams was also found in reinforced concrete. In 1934 Franz Dischinger patented the external prestressing in Germany [3]. In Fig. 2 it is clear that this system behaves as a concrete beam with a prestressed external tendon. This system was used by Dischinger already in 1937 in the three-span bridge at Aue (Germany) with a clear span length of 69 meters. The prestressed element consisted of steel bars with a relatively low prestresing stress. Unfortunately, the shrinkage and creep strains originated high prestressing losses, leading to large crack widths and deflections in the structure. To solve this problem, higher strength steels were later used for the prestressing steel, allowing pre-strains of about 5-6 mm/m. This solution was already suggested and patented in 1928 by the French engineer E. Freyssinet and allows keeping the prestressing losses due to creep and shrinkage below acceptable thresholds.

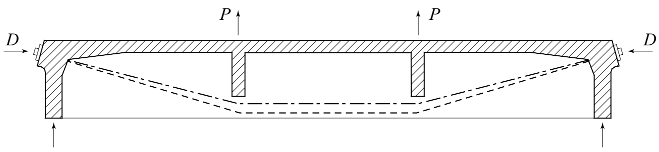

Fig. 2: External prestressing according to the patent of F. Dischinger, 1934 (adapted from [3])

In the 50's and 60's of past century, the concept of Freysinnet for bonded prestressing was dominantly used. At the beginning of the 70's, the technology of external prestressing started to be developed again. This development allowed also interesting solutions for precast segmental bridges [4] and for bridge girders with very thin or truss-like webs [5]. The underspanned solutions took advantage of the new developments in the field of prestressing tendons for external prestressing and cable-stayed bridges. This led to a new rise of underspanned solutions in the last decades of past century. Some relevant examples are the underspanned Neckartal bridge Weitingen [6] and the bridge over the Obere Argen [7], both in Germany, as well as the Osmort bridge (Spain) with a prestressed concrete deck [8]. In fact, the undersapnned solutions can be seen as an evolution of the external prestressing, where the prestressing tendon develops outside of the concrete cross-section. This leads to an increased effective depth but keeping a slender and simple cross-section [9]. Research works performed in Japan [10] have also shown that the actual structural behaviour of these structures is very satisfactory both at serviceability and ultimate limit states.

2.2 Number and shape of deviation elements

Bridges with highly eccentric external tendons require, in addition to the deck loaded in compression and to the external tension cable, a number of deviation elements linking the compression and tension chords and carrying the deviation forces of the cable to the deck. The simplest solution for the deviation elements consists of doubly-hinged struts (Figs. 1a and 1e). The aforementioned examples built in Germany and Spain belong to this type of solutions.
The number of deviation elements is governing for the design of the deck and for the complexity of the construction of the structure. With respect to permanent loads, the deviation elements reduce the effective span of the deck, limiting the bending moments acting on it. Assuming plastic design, the bending moments are proportional to the term $1/(n+1)^2$, where n refers to the number of deviation elements. This function is plotted in Fig. 3a. For variable live loads and for concentrated loads, the reduction of the inner forces in the deck due to the deviation elements is however more limited, refer to Fig. 3b (curve a).

A decrease of the bending moments in the deck can be achieved when the deviation elements are clamped on it (Fig. 1f), leading to a more suitable distribution of the tension forces in the tension cable. Such enhanced efficiency can be observed in Fig. 5b (curve b). Due to the fact that the deck needs to clamp relatively significant bending moments resulting from the deviation elements, a more suitable solution can be obtained by using V-shaped deviation elements, as shown in Fig. 1g and 4. In comparison to the previous solution, the clear spans of the deck are also reduced, limiting even more its acting bending moments. As shown in Fig. 3b, the use of four V-shaped deviation elements allows reducing the bending moments to approximately one half of those corresponding to doubly-hinged struts. First applications of this approach were performed in 1987 for a footbridge in Faido, Switzerland (figure 4, [11]) and the Inachus bridge in Beppu, Japan, designed by Mamoru Kawaguchi some years later [12].

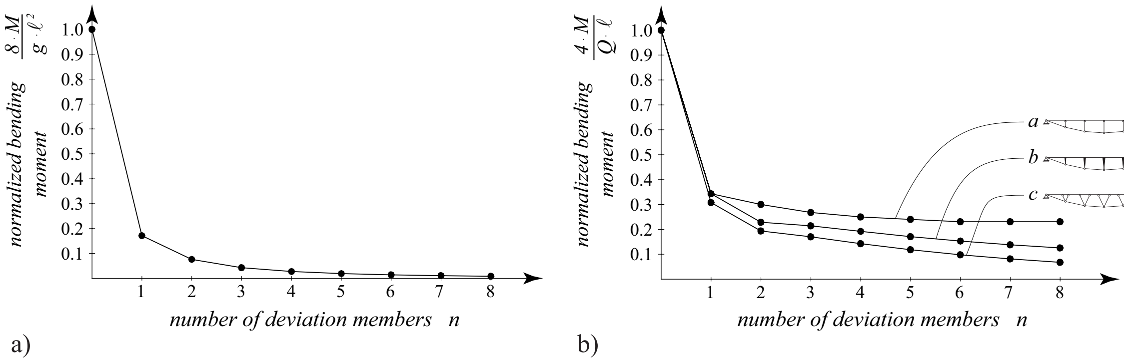

Fig. 3: Normalised bending demand of the deck assuming a plastic design as a function of the number of deviation elements (n = 0 referring to a non-underspanned member): (a) for a constant distributed load; and (b) for a concentrated load Q (curve c calculated with ℓ/f = 15, angle of deviation elements equal to 60°) (adapted from [2])

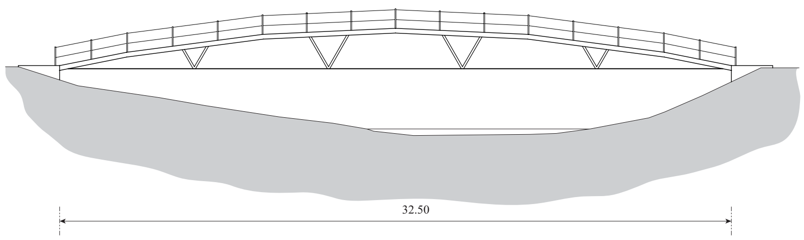

Fig. 4: Pedestrian Bridge in Faido, Switzerland, design by A. Muttoni, 1987 [11]

2.3 Design approaches for the underspanned tie

Two possibilities are available for the underspanned tie. One refers to the classical use of steel profiles or concrete ties whereas the other to the use of prestressing cables of high-strength steel. The former was firstly introduced in these systems by means of steel profiles. The main advantages of this solution are the possibility of using the steel elements as falsework of the deck (as for composite construction), simple detailing at the deviation points and an enhanced robustness and stiffness of the construction. The latter (prestressed underspanned cable) can be interpreted as an evolution of external prestressing. The cable can be composed of bars, wires or prestressing strands. The main advantages of this solution are the following: simplest control of counter-deflections, possibility of adjusting after concreting of the deck, lighter weight (transport, erection), no in-situ welding, simpler anchorage detailing at the abutments and the possibility of using the tension cable for a continuous or framed system [10,11]. In the following, two bridges will be presented where, by means of a combination of both principles for the underspanned tie, the advantages of both solutions are met.

2.4 Two examples of bridges with highly eccentric external tendons

The two bridges were built in Switzerland, one over the river Capriasca, (1995-1996) and the other over the river Ticino, (finished in 1997). Both were designed by the author and built following the system with V-shaped deviation elements and lenticular prestressed tendon. The bridges are innovative solutions proposed by the contractor, which were less expensive than those originally planned by the authorities. The first bridge was built to replace part of an old arch bridge built in the 50's, and the second one was built to replace a reinforced concrete bridge built in the 30's with an insufficient load-carrying capacity.

In the central part of both bridges, the structural system is a lenticular beam, where the top compression chord is a deck made of a slab reinforced with two longitudinal stiffeners, and the bottom tension chord has a polygonal shape (Fig. 5). The particular aspect of this system is the V-shaped deviation elements. As previously described, this system can be considered as a truss without diagonals, which offers two advantages: reduction of the number of nodes and easiness of construction. The shear forces on the parts without diagonals are carried by the vertical component of the tensile force in the chord and by the bridge deck.

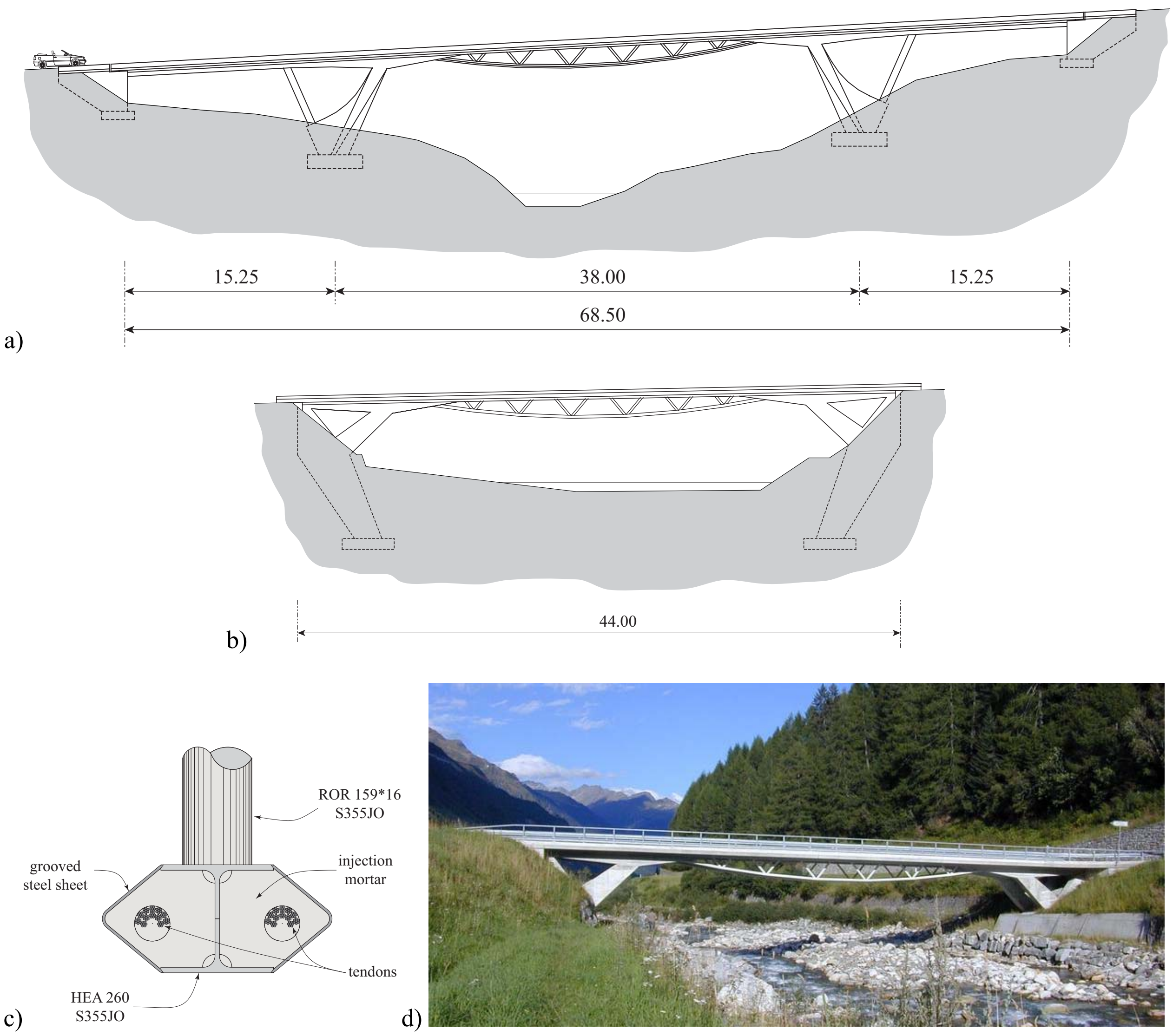

Fig. 5: Two examples of bridges with highly eccentric external tendons: (a) bridge over river Capriasca; (b) bridge over river Ticino; (c) tension member; and (d) view of bridge over river Ticino after completition of works (adapted from [2,11])

From the point of view of construction, the most relevant aspect is that the prestressing tendons develop outside the reinforced concrete section in the region subjected to positive bending moments. In this region, they are encased within a steel box section. At both extremities of the bridge, the deck can however be considered as a conventional prestressed structure.

The steel tension chord is made up of a W-shape, with two bent steel plates, welded to create a box section. Two prestressing tendons (VSL system with 12 Ø 0.6" strand each) are located within the box. After casting of the deck, the steel tension chord is injected with cement grout to create a composite section and to ensure a good protection of the tendons. Bond between the injection grout and the steel profile is improved by the use of grooved steel sheets on the inside surface of the tension chord. In comparison to more conventional solutions that use a single prestressing tendon, there are a number of advantages:

- tendons are doubly protected (both by the box steel section and by the grout injection) increasing durability and

protecting them against impacts or acts of vandalism;
- the chord section is stiffer ;
- the steel tension chord and the deviation elements can be constructed as a girder by adding an additional top steel member, creating a lenticular member before concreting of the deck. This girder can be used in addition as a self-standing formwork. The top steel profile is finally embedded within the deck.

The first bridge (Fig. 5a) is part of the main road Tesserete – Gola di Lago and crosses the Capriasca river near the Odogno village. Its structural system corresponds to a frame with two short lateral spans. The prestressing cables are anchored at each end and are continuous over the full bridge length. In the central part, these cables develop outside the prestressed concrete section and are located inside the steel tension chord as described.

The construction sequence was as follows:
1. construction of the foundation , the inclined piers and the lateral parts of the deck;
2. erection of the lenticular steel span (made of steel chords and V-shaped connecting members);
3. placing of the formwork on the steel span and pouring of the deck in the central part;
4. placing of the prestressing tendons. In the central part, this operation is easier due to the presence of conventional steel ducts fixed inside the steel box section;
5. injection between steel box section and the steel ducts with cement grout;
6. stressing of the tendons and injection of the ducts;
7. waterproofing, asphalting and finishing.

The second bridge (Fig. 5b), near the village of Villa Bedretto, crosses the Ticino River a few kilometers downstream from its spring in the Saint Gottardo mountain range. It consists of a prestressed concrete frame with integral abutments and without expansion joints. The piers are inclined toward the span to reduce the horizontal thrust of the frame and to facilitate the excavation of the foundation. With respect to the first bridge over river Capriasca, the second project optimises some constructive details and the deck was lighter. At the location of the deviation regions of the underspanned tie, steel saddles were welded. This allowed performing a partial prestressing of the structure before pouring of the deck's central part and without the need of executing before the grout injection (the deviation forces of the cable were directly transferred by the saddles to the V-shaped deviation members). As a consequence, the box steel section could be optimized since the construction sequence was no longer governing for this element (during pouring of the deck, the tensile chord was already prestressed) and the injection of the tendons was performed after the full prestressing of the tendons.

3. SOME CONCRETE BRIDGES WITH UNCONVENTIONAL CROSS SECTIONS

3.1 Two bridges with open cross-section and inclined webs

The Hexentobel bridge (approximately 340 m long) and the Marchtobel bridge (86 m long, Fig. 6a) have been built between 2004 and 2006 in the eastern part of Switzerland. They were built during construction of a new road situated between an existing one and a mountain railway line (Fig. 6c). The terrain had a steep slope of 38˚. Although this slope approximately corresponds to the internal angle of friction of the soil, there is no evidence of ground movements in the soil in the vicinity of the bridges. The bridges are very close to the ground surface. The alignment is characterized by strongly variable radius of curvature ($R_{\min}$ = 110 m), transverse slope (-7% to +7%) and longitudinal slope (0.5% to 4%).

The project described in the present paper was selected as the winner of the design and construction competition (Fig. 6). In the design phase, the construction procedure was developed in close collaboration between the contractors and the designers and had a significant influence on the development of the project. The most important factors were the complex topographical conditions (steep slope and small clearance between the bridge and the slope, limited space between the existing road and the railway line) as well as the geological conditions (requiring deep foundations).

The standard span of the two bridges is 32.1 m. It results from an optimization of the total costs. Typical side spans have a length of 27 m. The superstructure of both bridges consists of an open concrete cross section constructed in situ. It is prestressed in the longitudinal direction with two tendons (22 strands ø 0.6" each). Over the supports, thanks to an overlapping technique, three tendons are available in each web. The bridge girder and all supports are monolithically connected, so that mechanical bearings are only used at the abutments. The structural system of both bridges is thus a floating prestressed concrete frame, which is stabilized by the piers in the longitudinal direction.

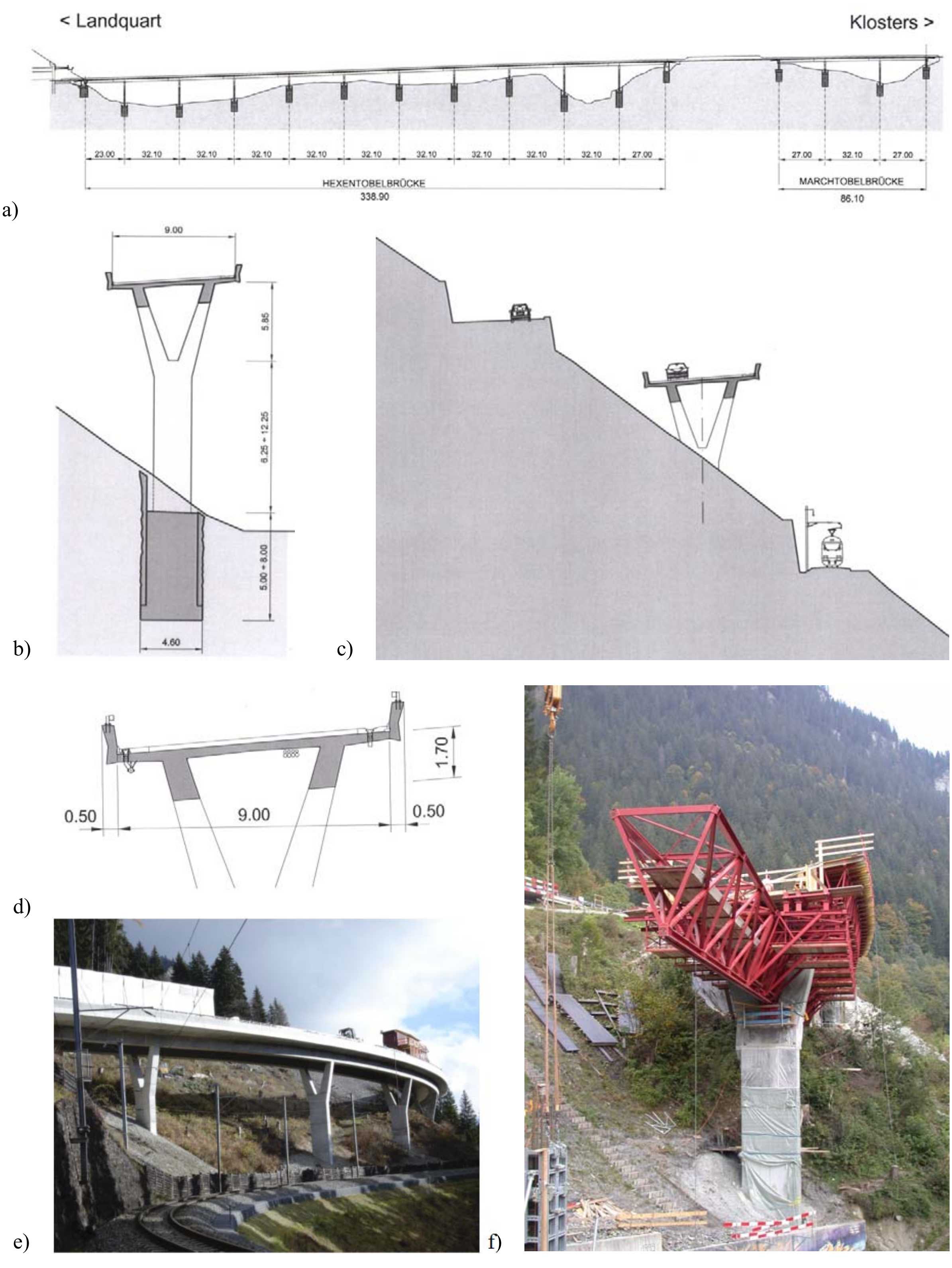

Fig. 6: The Hexentobel and Marchentobel bridges: (a) view of the bridges; (b-d) typical cross section; (d) bridge after construction; and bridge under construction (adapted from [13])

The bottom part of the pier has a rectangular cross section 2.80 m×0.80 m. Depending on the topographical conditions, its height varies between 6 m and 12 m. The slenderness of the piers both in the transverse and the longitudinal direction, results from an optimization of the piers and of the foundation shaft geometry. In its top part, the pier is composed of two inclined struts monolithically connected to the bridge girder (Fig. 6b). This shape, which is identical for all piers, derives from the construction procedure. Due to the difficult topographical conditions, the bridge superstructure was constructed with travelling scaffolding. Thus, accounting for the close presence of the slope, it was decided to place the main girder of the scaffolding in the centre of the cross section (fig. 6f). This led to the Y-shaped piers, which in turn required a triangular cross section for the main travelling girder.

The piers are founded in oval shafts. The shafts have external dimensions of 3.60 m×4.60 m and are supported by a 40 cm thick shaft ring. Their dimensions are optimal for small excavating machines that performed the required excavation in consecutive batches of 1.50 m depth. The depth of the shafts is variable (up to 9 m). After the excavation, each shaft was filled with concrete and the 1.10 m thick pier foundation was placed on it.

The depth of the bridge girder is equal to 1.70 m and remains constant over the whole length of the bridge. The width of the roadway is 9 m. An open bridge cross section without diaphragms over the piers was selected to simplify as much as possible the removal of the formworks for the concrete construction.

To avoid deviation forces dues to the inclination of the upper part of the Y-shaped piers (that could be usually carried by the diaphragms), it was decided to incline also the webs (Fig. 6b). This solution, which is usually for box girders, is quite uncommon for open cross sections. In this case, it has the advantage that the deviation force in the transverse direction is distributed over the entire length of the bridge and thus, diaphragms can be avoided.

In addition, to resist the compressive forces induced by the longitudinal bending, the webs are thicker at the bottom than at the top (Fig. 6d).

The abutments have a conventional layout, with shaft foundations similar to the piers. For the 86 m long Marchtobel bridge, which has the same foundations, piers and deck, it was possible to avoid the use of an expansion joint by means of suitable shape of the transition slabs.

The development of the construction procedure and the design of the bridges influenced each other. The triangular travelling girder consists of a 41 m launching girder and a 20 m launching nose. The girder is supported on rollers placed in the V-notch on top of the piers and is moved using hydraulic jacks. During the construction of a span, it is fixed to the already constructed bridge girder and supported on the next pier.

Because of the horizontal curvature, the travelling girder needs to be rotated horizontally before it is moved to the next pier. This is achieved by rotating the girder around the V-notch on the pier using a special crossbeam suspended on the previously constructed span. This girder allows the rear section of the travelling girder to be moved outwards thus allowing the rotational movement.

3.2 An integral bridge with a single web T-shaped cross-section

The new bridge over the Verzasca River at Frasco (Switzerland) was designed in 2002 and built in 2009. It is an innovative structural concept coming from the need of respecting maximum transparency for the structure, minimizing the impact on the flow of the River, enhancing the durability and robustness of the structure and reducing construction and maintenance costs.

The road through the Verzasca valley crosses over the Verzasca River by means of a 19th century masonry bridge. The existing bridge has two spans, with a pile founded on the rocks at the middle of the course. The old bridge presented some deficiencies such as a rather limited width (4.80 m), damages in the vaults and stability problems on the vertical walls. A rehabilitation project of the bridge was however not performed due to its significant cost. As an alternative, the project for a new bridge was proposed by the author of this paper.

The design of the new bridge was marked by the presence of the existing one, as it needed to be calm and temperate but with sufficient personality. As a synthesis of such considerations, it was selected the design of a slender prestressed concrete girder, ensuring maximum transparency in order not to hide the existing stone bridge. These exigencies as well as economic considerations explain the choice of a T-shaped cross-section with a single web.

The flow of the Verzasca River has notable floods during spring and summer. Typically, the flow carries no more than 10 m^3/s. However, during floods, the river may carry up to 660 m^3/s. In addition, during floods, the first meters of the soil can be eroded. Due to these considerations, foundations needed to be deep enough to avoid scour danger. In order to comply with such exigencies and to clamp the T-shaped girder to increase the slenderness of the bridge, an efficient structural concept was developed. It consists in creating two clamping cells (one by side) founded on footings sufficiently deep. Each clamping cell is composed of a prestressed inclined wall and an inclined strut, the girder being clamped on both elements. The shape and dimensions of the various elements were optimized in order to provide maximum transparency.

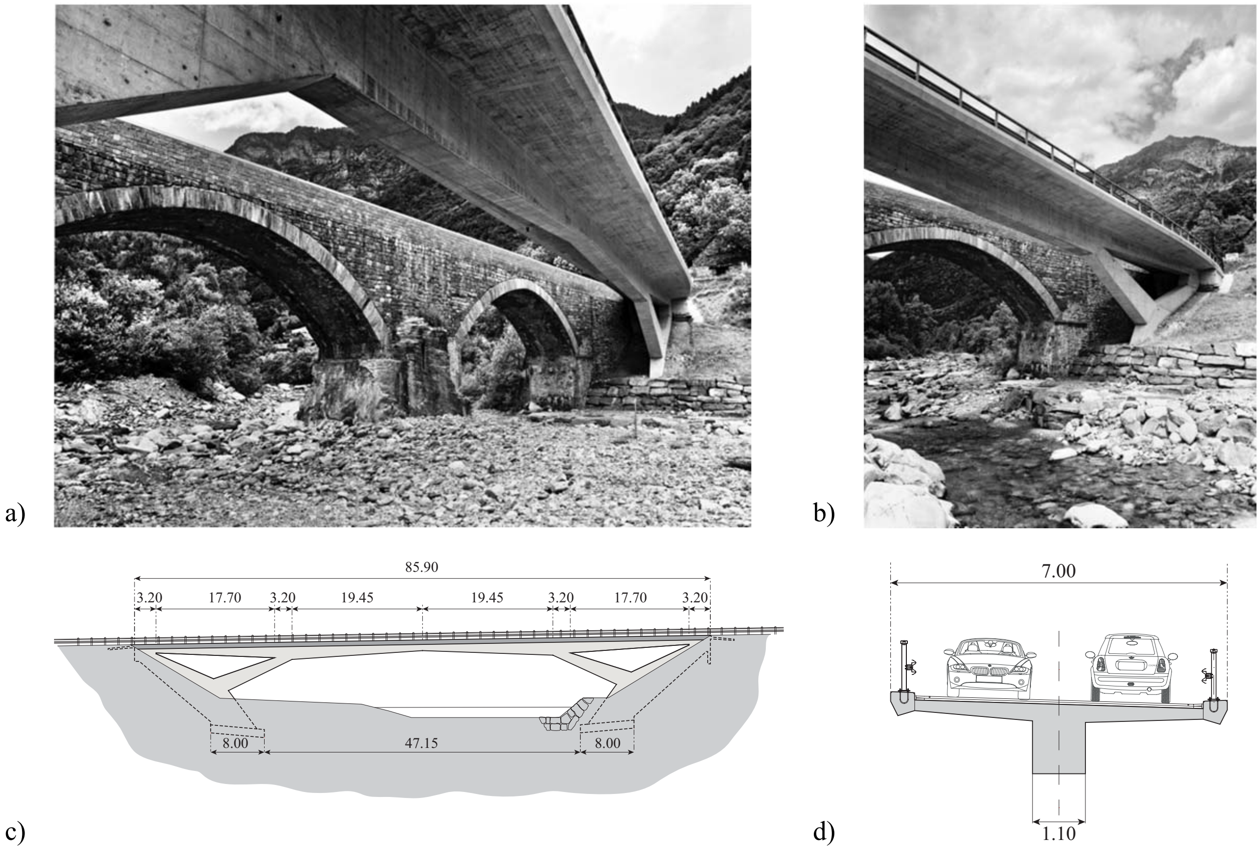

Fig. 7: Bridge over the Verzasca river: (a,b) view of the new and old bridges; (c) horizontal view; and (d) cross-section of the bridge (adapted from [14])

The bridge is 85.20 m long, with a central clear span of 41.60 m. The deck slab is 7.00 m wide, supported on a central web 1.10 m thick (T-shaped cross-section). Such cross section was selected in order to ensure maximum transparency and to increase durability due to the protection offered by the deck slab. Torsion due to eccentric loading is carried by the central web to the clamping cells. The girder has a slenderness equal to 1.50/43.10 = 1/29 at mid-span and equal to 2.50/43.10 = 1/17 at the clamped sections. Longitudinal prestressing of the girder consists of three tendons with 19 strands 15.7 mm diameter.

In order to minimize interference with the flow of the Verzasca River, the struts of the clamping cells do not carry directly the loads to the foundations. As a consequence, significant efforts develop in the wall. Two tendons with 19 strands 15.7 mm diameter are thus placed to ensure sufficient strength at ultimate and correct behaviour under service loads. The shape of the clamping cells was optimized during design by development of stress fields and strut-and-tie models [14].

The framed structure is fully monolithic to the abutments and transition slabs. This avoided placing expansion joints or bearings with the aim of reducing maintenance costs.

4. A PRESTRESSED CONCRETE SHELL

4.1 Why a concrete shell?

First concrete shells were built as thin members in reinforced concrete [15]. In Europe (F. Dischinger, E. Torroja, R. Maillart) and in America (A. Tedesko), designers experienced the advantages of double-curvature constructions (dominated by membrane forces and with limited bending moments and shear forces) allowing to build structures with thicknesses as low as 30-40 millimetres. These works, built mainly between 1920 and 1940, typically presented shapes defined by analytical expressions (such as sections of spheres, cylinders or hyperbolic paraboloids). They presented stringers in the edges to ensure as perfect membrane behaviour as possible. The difficulties found for the analytical treatment of thin shells explains the rather limited number of shapes used during this period and the reduced number of designers using them. Following this period, interesting new developments took place between 1940 and 1970 in America by the Spanish architect F. Candela and the Uruguayan engineer E. Dieste (the latter experiencing also with the development of masonry shells). Their approach consisted on performing as simpler as possible analyses (particularly for Candela) and combining different sections of previous shapes, privileging mostly hyperbolic paraboloids due to their plastic qualities and easiness of construction. Their approach led to a larger and richer variety of forms. In Switzerland, H. Isler [16,17] built between the 1960's and 1980's also an impressive number of unusual shells whose shape was obtained and optimized by different mechanical analogies (pneumatic, gravity-shaped membranes…). It can be noted that, actually, the different experiences on concrete shells were mostly linked to the genius of their designers rather than a continuous evolution in concrete shell design.

In the 1980's and 1990's, concrete shells were seldom used as a consequence of the large amount of man work required for formworking and placing of reinforcement with respect to the material costs, which typically privileged other structural solutions. In the last years, the situation has somewhat changed. The possibilities offered by new types of concrete (as fibre-reinforced concrete), the introduction of prestressed reinforcement in these structures, the numerical cut of formworks and their positioning at construction site as well as the new possibilities available for the analysis of these structures (related mostly to computer software) have allowed the development of a new approach to shells, with more freedom in the choice of the shape. Excellent examples of these new possibilities have been developed in Japan by the architect Toyo Ito and the engineer Mutsuro Sasaki, as the Meiso no Muri Crematorium in Gifu (2006).

Another example of an innovative concrete shell taking advantage of these developments has been recently constructed in Switzerland [18] in order to cover a new mall to be at Chiasso with an ellipsoid-shaped roof. This satisfied the requirements of the client in terms of usability, architectural needs and image. The thickness of the ellipsoid was decisive since it directly influenced the amount of surfaces that could be rented. Solutions were investigated using timber and steel linear elements. Local buckling was however governing for these solutions, requiring significant thicknesses in the most critical parts. This led to uneconomical solutions for the client, with significant reductions on the rentable surfaces. A concrete shell showed on the contrary to be a suitable solution. Its thickness was of only 100 mm in the critical regions influencing the rentable surfaces. This allowed the client to have sufficient space at disposal and optimized the cost of the mall.

4.2 Geometry and main properties of the shell

The ellipsoid shell has axis dimensions of 92.8 m (long axis) × 51.8 m (small axis) × 22.5 m (height). The ellipsoid is cut by a horizontal plane and is supported on a concrete basement composed of transverse walls, leading to a total height for the shell of 18.24 m, see Fig. 8. The thickness of the shell was variable. A value of 100 mm was selected as the default thickness. This was justified by constructive reasons (minimum thickness respecting necessary reinforcement cover) and also allow ensuring sufficient safety against buckling. Four reinforcement layers were provided, two at the intrados and two at the extrados of the shell. The reinforcement layers were oriented following the radial (meridian) and tangential (parallel) directions. This was selected as the most effective layout for statical reasons. Arrangement of four layers of reinforcement was justified to control bending moments and shear forces developing at the basement connection, near the prestressed zone and for connecting to the steel piece placed at the zenith opening (Fig. 8). Bending moments and shear forces in other regions were very limited. Four reinforcement layers were nevertheless arranged in all regions for constructive reasons, to ensure suitable crack control (which may potentially appear depending on the load cases) and to ensure sufficient safety against buckling of the structure.

In addition to the ordinary reinforcement, 35 post-tensioning tendons (0.6" monostrand tendon) were arranged near the equator of the shell (from level +5.50 m to level +12.60 m, refer to Fig. 8) to carry quite significant tension membrane forces along the horizontal direction. They presented in addition a limited dimension for the plastic duct thus minimizing the disturbance in the compression field developing through the shell. The thickness of the shell was increased in this region to 120 mm (between levels +4.24 and +13.35).

At the level of the connection to the concrete basement (between levels +4.24 and +5.14) shear studs were also arranged to provide sufficient shear strength and deformation capacity in this region (subjected to parasitic shear forces and bending moments).The thickness of the shell was also 120 mm from level +21.60 to the zenith opening. On the top part, the increased thickness allowed to link the concrete shell to a steel structure placed at the zenith opening (10.21 m × 5.70 m), allowing day light to reach the inside of the mall. In addition, between levels +4.81 m and +18.78 m, a number of circular openings (diameter 0.40 m) were also arranged, see Fig. 8.

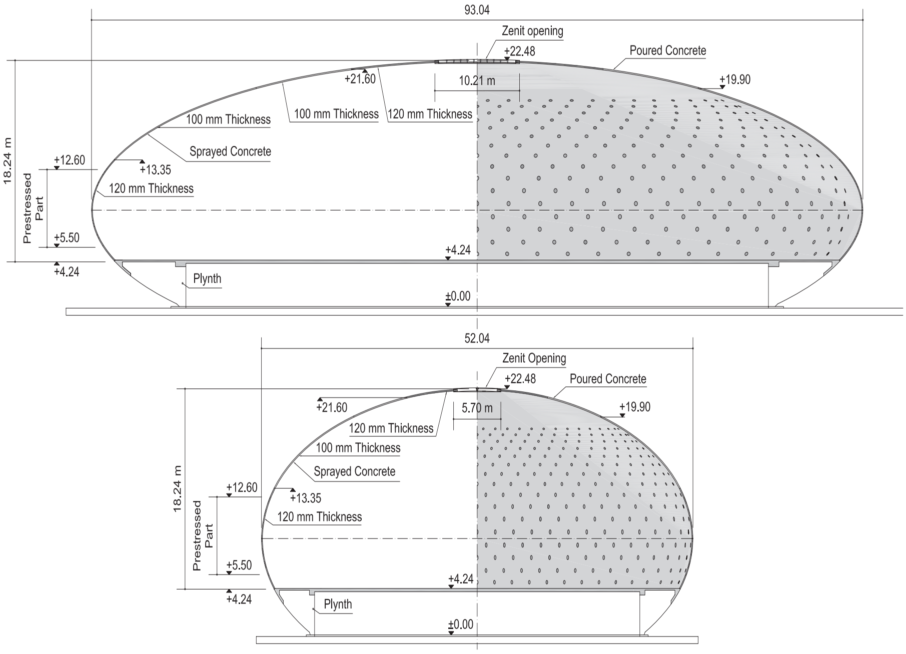

Fig. 8: Main geometrical dimensions of the concrete shell: (a) section along long axis; (b) section along small axis (adapted from [18])

4.3 Concrete properties

The structure was cast using sprayed concrete from level +4.24 m to level +19.90 m. This allows using conventional (one-side) formwork for the entire shell. Where the slope was sufficiently limited (lower than 20°, from level +19.90 m to level +22.48 m), concrete was poured conventionally. For both concrete types, a characteristic compressive strength (f_{ck}) at 28 days equal to 30 MPa was specified. In the sprayed concrete region, between levels +4.24 m to level +13.36 m, hooked metallic steel fibres (30 kg/m^3) were used. The fibres had a length of 30 mm and a length-to-diameter ratio of 80. The fibres were arranged to enhance crack control (in the post-tensioned region) and to improve the ductility of concrete under high shear forces (at the link to the basement). The sprayed concrete had 300 kg/m^3 of cement and 25 kg/m^3 of lean lime. The latter was placed to enhance workability of the concrete. The aggregate sizes between 0 and 4 mm were 70 % of the total, the rest ranging between 4 and 8 mm. The addition of water was performed at the spraying gun.

4.4 Construction

Formwork was placed over wood scaffolding, Fig. 9a. The formwork was composed of panels bent in situ and screwed at their corresponding location (Fig. 9b). Reinforcement was then placed and concrete was sprayed or poured in situ (Fig. 9c,d). Time for placing of the reinforcement and concreting of the shell took about 3 months in total.

After concreting, decentering of the shell was performed. For the present shell, it was performed in a number of phases, in order to avoid decentering to be the governing design situation. First, half of the post-tensioning force was applied (one out of two tendons post-tensioned). Then, the wood scaffolding in contact with the post-tensioned zone was removed, followed by the post-tensioning of all tendons. This operation ensured correct post-tensioning transfer to the concrete. Finally, the vertical struts of the scaffolding supporting the top region of the shell were gradually released, leading to the complete decentring of the structure. Measured deflections were recorded during the process in good agreement to predicted values. Some pictures of the completed work can be seen in Fig. 9.

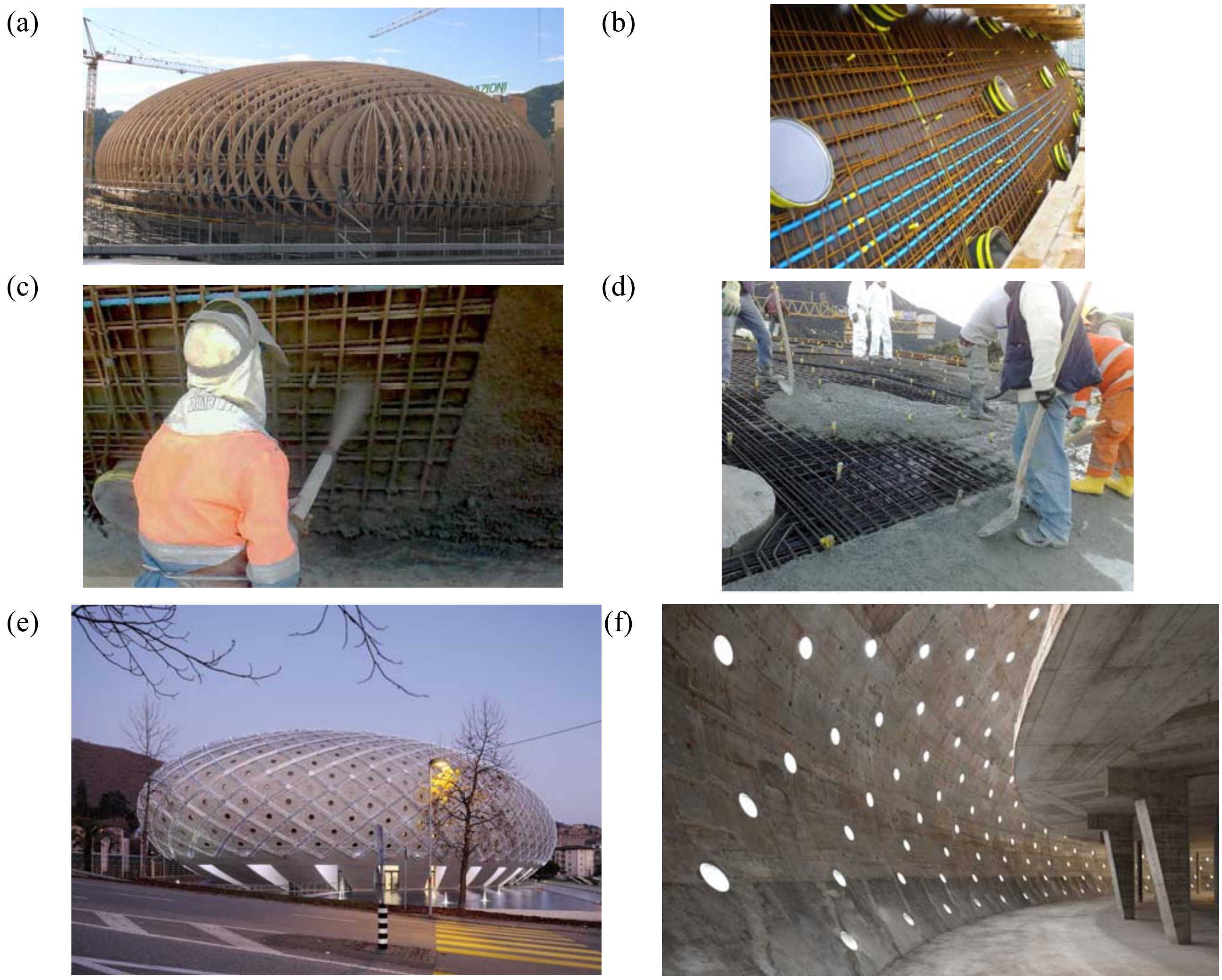

Fig. 9: Construction of the shell: (a) temporary scaffolding; (b) placing of prestressing tendons; (c) spraying of concrete; (d) pouring of concrete; and (e-f) completed work

5. CONCLUSION

Prestressed concrete was invented, developed and started to be used almost one century ago. Despite the fact that it has been intensively used, its possibilities for creating innovative structures are still open. Some promising fields for its development have been presented in this paper with reference to a number of selected examples:

- Highly eccentric external tendons
- Girders with inclined webs or with only one web
- Thin concrete shells

These cases show that the use of prestressing allows designing and building structures in an efficient, light and slender manner. This allows a rational use of natural resources and leads to a more sustainable manner of building. It should also be noted that these structures can be considered, to a large extent, evolutions of previous shapes that have become attractive and competitive again by means of new technologies. Knowledge of previous experiences, combined with creativity and sense of construction, is thus usually required to find suitable solutions for new structural challenges.

REFERENCES

1) Dietrich, R. J.: Faszination Brücke, Baukunst – Technik – Geschichte. München: Callwey 1998
2) Muttoni A., Bridges with highly eccentric external tendons, Stahlbau, Vol. 71 (2002), Issue 8, (in German: Brücken mit vorgespannter Stahlunterpannung), pp. 592-597
3) Günschel, G.: Grosse Konstrukteure I, Berlin: Verlag Ullstein 1966
4) Ganz, H. R., Meyer, M. E.: Segmentbauweise, Schweizer Ingenieur und Architekt (1997), pp. 528-533
5) Mathivat, J.: L'évolution récente des ponts en béton précontraint, Ingénieurs et architectes suisses (1987), pp. 86-91
6) Wössner, K., Andrä, W., Kahman, R., Schumann, H. und Hommel, D.: Die Neckartalbrücke Weitingen. Stahlbau 52 (1893), pp. 65-77 and 113-124
7) Schlaich, J., Seidel, J. und Sander, D.: Teilweise unterspannte Schrägseilbrücke über die Obere Argen. IVBH 13. Kongress (1988), Helsinki, Bericht, pp. 863-868
8) Manterola Amisen, J., Fernandez Troyano, L., Lopez Padilla, A. und Martinez Cutillas, A. :The Osormort Viaduct near Barcelona, L'industria italiana del cemento (2000), pp. 384-397
9) Menn, C.: Unterspannung von Brückenträgern mit gedrungenem Querschnitt, Eine sinnvolle Anwendung der externen Vorspannung, Bauingenieur 65 (1990), pp. 209-219
10) Aravinthan T., Witchukreangkrai E., and Mutsuyoshi H, Flexural Behavior of Two-Span Continuous Prestressed Concrete Girders with Highly Eccentric External Tendons, ACI Structural Journal, V. 102, No. 3, May-June 2005, pp. 402-411.
11) Muttoni, A.: Brücken mit einem innovativen statischen System, Schweizer Ingenieur und Architekt (1997), pp. 548-551
12) Kawaguchi M., Granite Pedestrian Bridge, Beppu, Japan, Structural Engineering International, Volume 6, Number 3, 1 August 1996, pp. 148-149
13) Figi H., Lurati F., Muttoni A., Somaini D., The Hexentobel and Marchtobel bridges, Structural Concrete in Switzerland, Report for the fib-Congrees in Naples, Italy, 2006, pp. 92-97.
14) Muttoni A., Muttoni L., Lurati F., The New Bridge at Frasco over the Verzasca River, Structural Concrete in Switzerland, Report for the fib-Congrees in Washingtgton DC, USA, 2010, pp. 77-80.
15) Cassinello, P., Schlaich, M., Torroja, J. A., Félix Candela. In memorian (1910-1997). From thin concrete shells to the 21st century's lightweight structures (in Spanish: Félix Candela. En memoria (1910-1997). Del cascarón de hormigón a las estructuras ligeras del s. XXI), Informes de la Construcción, Vol 62, No 519, 2010, pp. 5-26
16) Chilton, J., Isler, H.. Heinz Isler : The engineer's contribution to contemporary architecture, RIBA Publications/Thomas Telford, 2000, pp. 20-29
17) Kotnik, T., and Schwartz, J., The Architecture of Heinz Isler, Journal of the International Association for Shell and Spatial Structures, Vol.52, Nr.3, 2011, pp. 185-190
18) Muttoni, A., Lurati, F., Fernández Ruiz, M., Concrete shells - Towards efficient structures: Construction of an ellipsoidal concrete shell in Switzerland, Structural Concrete, Ernst & Sohn, Germany, Vol. 14, No. 1, 2013, pp. 43-50

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