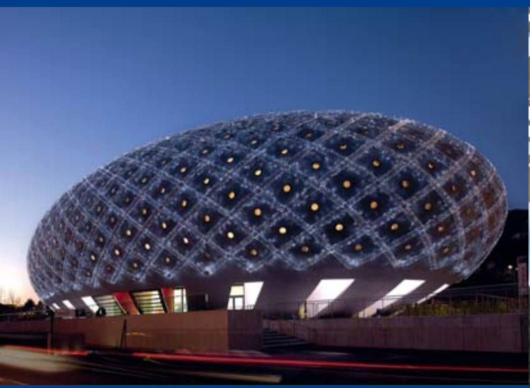
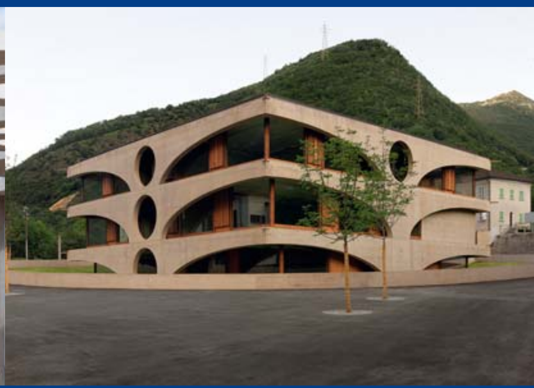
Hochbau

Bâtiment

Buildings

Die Sporthallen Mülimatt in Brugg
Sports hall Mülimatt, Brugg

Massimo Laffranchi, Armand Fürst

Fig. 1
Ansicht der Faltwerkstruktur mit verglasten Flächen an den Stirn- und Längsseiten.
View of the folded-plated concrete structure with glazed facades at the front and longitudinal sides.
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Einleitung
Das neue Sportausbildungszentrum mit zwei Dreifachturnhallen, diversen kleineren Turnhallen und Sporträumen, Unterrichtsräumen sowie Sportaussenplätzen wurde an einem einzigen Standort beim Naherholungsgebiet an der Aare in Brugg erstellt.
Das Gebäude ist aufgrund des exponierten Standorts, seiner beachtlichen Abmessungen und der Nähe zum Bahndamm ein Blickfänger für die Bahnreisenden. Diese Feststellung sowie das Bestreben, ein effizientes und leichtes Tragwerk zu realisieren, um die freie Spannweite der ungeteilten Dreifachhallen von 50 m zu überbrücken, führte zum Entwurf einer dünnwandigen, gefalteten Betonschale, die alle Sport- und Unterrichtsräume umhüllt (Fig. 1 und 3). Das auf die Geländeneigung abgestimmte, asymmetrische Faltwerk in Sichtbeton verleiht der Dachfläche und den auf Aare- und Bahnseite unterschiedlich hohen Längsfassaden ein einheitliches Erscheinungsbild. Die Dachträger stabilisieren die verglasten Stirnflächen. Das monolithische Betontragwerk wirkt ausserdem als wetterfeste, gefaltete Haut. Die Wärmedämmung im Dachbereich ist in der heruntergehängten Decke integriert, die an den Dachträgern befestigt ist. Die Stiele schützen und beschatten ihrerseits die verglasten Flächen der Längsfassaden. Das Regenwasser wird in den Rinnen der Dachfalten gesammelt und entlang der Stielflächen abgeleitet (Fig. 2).

Introduction
The new sport facility with two triple gyms together with several minor gyms and classrooms as well as an outdoor sports field was built in a single location giving appropriate consideration to the recreational area along the river Aare.
Due to the distinctive location and its striking external dimensions, the building is an eye-catcher for rail travellers. This consideration, as well as the need for an efficient and light structure resulted in a thin-walled, folded-plate concrete shell structure that encloses all the sports and teaching facilities (Fig. 1 and 3). It rises from two different ground levels and extends over the roof surface and the longitudinal facades. The glazed faces at the front of the building are stabilised by the front roof beams. Further, the monolithic concrete structure distinguishes the sports hall and acts as a

Konzepte für das vorfabrizierte Faltwerk
Während die Fundationen, das Tragwerk des erdberührten Sockelgeschosses sowie sämtliche Decken und Einbauten der Sporthalle vor Ort betoniert wurden, besteht die darüber gespannte

weatherproof folded membrane. The ceiling within the thermal insulation is fixed to the bottom of the roof beams, whilst the column beams protect and shade the glazing on the long side of the building. Rainwater is collected in the roof folds and runs along the surface of the column beams, which function as drain gutters as a result of their shape (Fig. 2).

Description and design of the prefabricated structure

As opposed to the foundations, the underground structure as well as the floor slabs and walls of the sports hall, which are made of in-situ concrete, the folded-plate frame structure is realised in pre-cast concrete. Thus, the most cost-effective solution and high execution quality can be guaranteed by taking advantage of self-compacting concrete technology and of the possibilities given by the post-tensioning method. The thickness of the structural members is reduced to a minimum, allowing placement of small tendons in the section and their anchorages in the frame corners, without altering the appearance of the fair-faced concrete structure. The size of the elements is chosen to minimise the total number of joints. Element weight and length is limited by the handling in the production facility and by the conditions for road transportation. The 27 shorter column beam elements on the railway side have a length of 11.1 m and a weight of 35 t. Those on the river side have a length of 14.3 m and a weight of 43 t each. The 81 roof beam elements have a constant length of 16.3 m and a weight of about 49 t.

The roof- and column-beam elements are connected by in-situ concrete joints and internal post-tensioning tendons to form 27 monolithic 'frame units' (FU) with a span of 52.6 m. The 30 mm wide gaps between the FUs are grouted by a special cement grout. Additionally, welded steel plates along the roof ridge provide a rigid connection and enable the

Faltwerkstruktur aus Betonfertigteilen. Im Werk liessen sich wirtschaftlich optimierte Lösungen und eine hohe Ausführungsqualität durch den Einsatz von selbstverdichtendem Beton und der Vorspanntechnologie erzielen. Kleine Spannglieder mit besonderen, schmalen Endverankerungen in den Rahmenecken ermöglichten dünnwandige, vorgespannte Faltwerkscheiben, ohne das angestrebte Erscheinungsbild der Sichtbetonflächen zu beeinflussen. Die Fertigteile wurden so gross wie möglich gewählt, um die Anzahl der Ortbetonfugen zu minimieren. Die Grenzen betreffend Grösse und Gewicht setzten die Einrichtungen zur Handhabung der Elemente im Werk und die Bedingungen für den Strassentransport. Die 27 kürzeren Stiele der südlichen Fassade auf der Bahnseite sind 11,1 m lang und 35 t schwer. Jene der nördlichen Fassade auf der Aareseite weisen eine Länge von 14,3 m und ein Gewicht von 43 t auf. Die 81 Dachträgerelemente weisen eine konstante Länge von 16,3 m und ein Gewicht von 49 t auf.

Die Stiel- und Dachträgerelemente werden durch bewehrte Ortbetonfugen und eine nach der Montage eingezogene Dachvorspannung zu 27 monolithischen Rahmeneinheiten mit einer Spannweite von 52,6 m miteinander verbunden. Die 30 mm breiten Spaltfugen entlang der Dachkante zwischen den Rahmeneinheiten werden mit Mörtel vergossen und mit verschweissten Stahlbauteilen mechanisch verbunden. Dadurch wird eine Schalenwirkung der Dachfläche für die veränderlichen Einwirkungen erzielt (Fig. 7). Sämtliche Fertigteile weisen eine Querschnittshöhe von 2,59 m und eine konstante Breite von 2,93 m auf. Die mittlere Stärke der Dachstruktur, bezogen auf die Grundrissfläche, beträgt lediglich 0,37 m.

Die Rahmeneinheiten stabilisieren das Gebäude in Querrichtung. Die am Rahmeneck unter Dach angeordneten Diagonalscheiben werden durch ein in Längsrichtung durchgehendes, horizonta-

Fig. 2
Die äusseren Stielflächen wirken auch als Rinnen für das abfliessende Regenwasser Längsseiten.
The columns function as drain gutters.
(© Studio Vacchini Architetti, Locarno).

Bauherrschaft
Kanton Aargau, vertreten durch die Immobilien Aargau, Departement Finanzen und Ressourcen, und Stadt Brugg
Planungsteam
Bauingenieur: Fürst Laffranchi Bauingenieure GmbH, Wolfwil; Architekt: Studio Vacchini Architetti, Locarno; Landschaftsarchitekt: Paolo Bürgi, Camorino
Ausführung
Arigon Generalunternehmung AG, Zürich mit den Subunternehmern Element AG, Veltheim (Betonvorfabrikation); VSL (Schweiz) AG, Subingen (Vorspanntechnik); Jäggi AG, Brugg (Baumeisterarbeiten)

Owner
Canton Aargau, represented by the Immobilien Aargau, Department of Finance and Resources, and Brugg
Planning team
Civil engineers: Fürst Laffranchi Bauingenieure GmbH, Wolfwil; Architect: Studio Vacchini Architetti, Locarno; Landscape architect: Paolo Bürgi, Camorino
Execution
Arigon Generalunternehmung AG, Zurich (main contractor) with the subcontractors Element AG, Veltheim (prefabricated concrete); VSL (Schweiz) AG, Subingen (post-tensioning); Jäggi AG, Brugg (construction work)

Fig. 3
Ansicht der Sporthallen von der Aareseite: Das durchgehende Foyer unterhalb der zwei Dreifachhallen ist gut erkennbar.
View of the sports hall from the river side: the foyer underneath the two triple gyms is recognisable.
(© Kanton Aargau, Foto: René Rötheli, Baden).

les Zugglied verbunden (Fig. 9). Dadurch wird die Form des Faltwerkdachs erhalten und die Stiele wirken als Teile eines über die gesamte Gebäudelänge aufgespannten Rahmens, der die Längsstabilisierung sicherstellt.
Der Horizontalschub der Rahmeneinheiten wird durch die vorgespannte Deckenplatte des Sporthallenbodens aufgenommen, die somit auch als Zugglied wirkt (Fig. 4). Die bei jedem Stiel angeordneten Ortbetonpfähle sind an ihrem Kopf in ein steifes Bankett eingebunden und übertragen die vertikalen Kräfte in die untere, steifere Kiesschicht. Jeder Pfahl weist einen Durchmesser von 0,80 m und eine Länge zwischen 7,0 und 11,0 m auf. Das Ortbetontragwerk ist unabhängig vom Faltwerk der Hallen in der oberen, mittelsteifen Kiesschicht flach fundiert. Setzungsdifferenzen bis zu 20 mm lassen sich durch die schlanken Verbindungen aus rostfreien Zugstäben zwischen den Stielen und der Sporthallendecke aufnehmen.

Technologie und Konstruktives

Vorspannung

Das Konzept für die Vorspannung des Faltwerks wurde ausgehend von der gewünschten Tragwerksform und mit Rücksicht auf die dünnwandigen Bauteile entworfen. Die kleinen Spannglieder im Verbund mit maximal 6 Litzen und einer Vorspannkraft bis P_0 = 1,1 MN finden innerhalb der Dach- und Stielscheiben Platz (Fig. 5). Die Breite der festen

desired shell action for live loads (Fig. 7). The precast roof and column beams have a constant section height of 2.59 m and a constant width of 2.93 m. The average thickness of the roof structure related to the covered horizontal surface is 0.37 m.
The FUs guarantee the stability of the structure in the transverse direction. In the longitudinal direction, the diagonal panels at the frame corners (Fig. 9) are concatenated and accommodate a continuous tension tie that ensures the folded form. In addition, they connect the column beams at the top to a continuous multiple frame, which guarantees the longitudinal stability.
The thrust of every FU is equilibrated by a tie connection to the post-tensioned concrete slab of the gym floor (Fig. 4). Thus, only vertical forces have to be carried from each column to the underlying compact gravel layer by a single concrete pile. All piles have a diameter of 0.80 m and variable lengths between 7.0 and 11.0 m. In order to reduce the risk of possible differential pile settlements, a rigid foundation beam (pile cap) connects the piles at the top. The in-situ sports hall structure is founded independently in the upper gravel layer on a ground slab. Relative settlements of up to 20 mm can be accommodated by

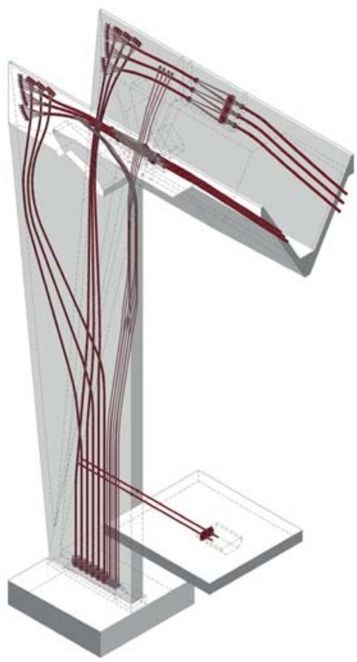

Fig. 4
Spannglieder und deren Verankerungen im Stiel und im anschliessenden Dachträgerelement sowie Zugverbindung zwischen Stiel und Sporthallendecke.
Tendons and anchorages in column and roof beams, tension ties connecting the column with the gym slab.

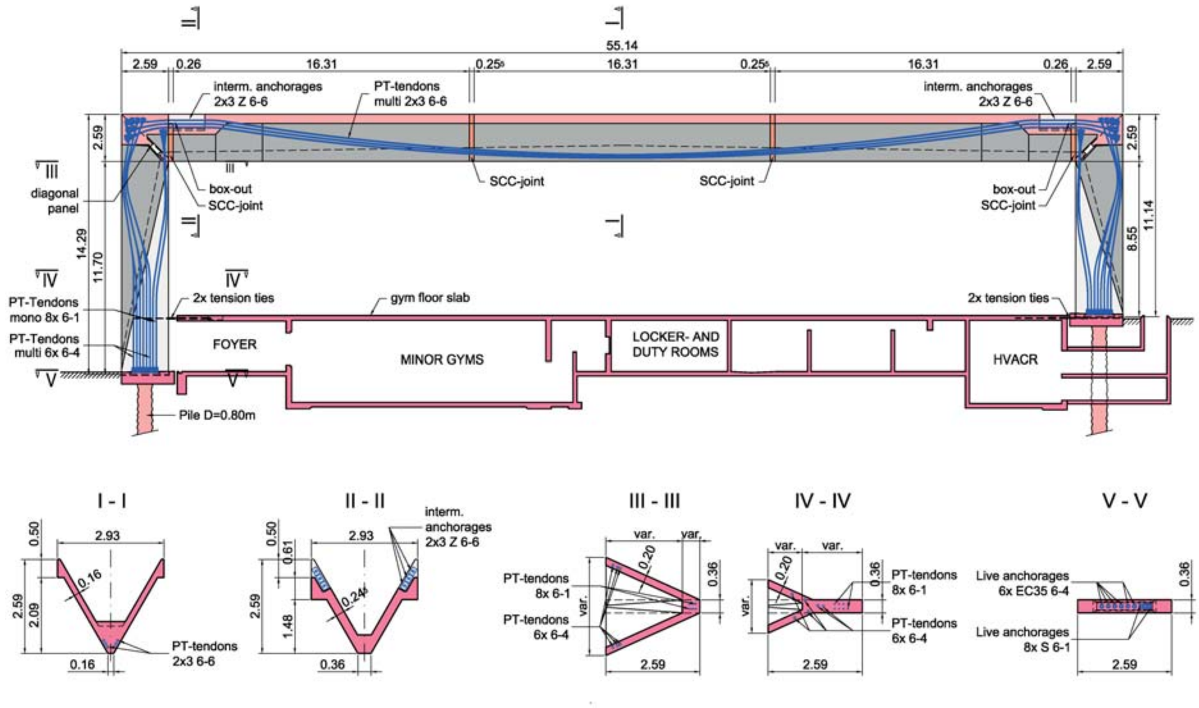

Fig. 5
Längsschnitt und Querschnitte einer Rahmeneinheit mit Elementeneinteilung, Ortbetonfugen und Spanngliedern.
Longitudinal and cross sections of a frame unit (FU) showing the precast roof and columns beams, the cast-in-situ joints and the post-tensioning tendons.

the slender stainless tie bars connecting the column beams to the gym's floor slab.

Technology and detailing

Post-tensioning

The conceptual design for the post-tensioning was developed taking into account the desired structural form and the thin-walled structural members. The grouted tendons are small sized with up to 6 strands and a prestressing force of P_0 = 1.1 MN, in order to fit into the member sections (Fig. 5). They require little space in the frame corner for the dead-end anchorages, specially developed for this project. The prefabricated column beams were post-tensioned at the factory (Fig. 10). Stressing of the tendons was carried out from the column base, where the live anchorages are located in a common block-out. Supplementary monostrand tendons had to be provided for the construction phase. On the other hand, the precast roof beams were post-tensioned only after erection of

Anker in den Rahmenecken wurde eigens für das Projekt minimiert. Die vorfabrizierten Stiele wurden im Werk vorgespannt (Fig. 10). Die vorkonfektionierten Spannglieder à 4 Litzen für den Endzustand sowie zusätzliche Monolitzenspannglieder ohne Verbund für die Transport- und Montagephase wurden aus einer Spannnische am Stielfuss aktiviert. Die Dachspannglieder wurden erst nach der Montage der Dachträgerelemente auf provisorischen Türmen und dem Betonieren respektive Vergiessen der Quer- und Längsfugen konfektioniert. Die Litzen wurden in die Hüllrohre eingestossen und aus einer Spannnische auf der Dachoberseite mit kompakten Zwischenverankerungen gespannt. Die Nischen wurden anschliessend ausbetoniert und sind im Endzustand nicht erkennbar (Fig. 11). Sämtliche Querschnitte sind unter den ständigen und den veränderlichen Einwirkungen vorgespannt. Die mittlere Betondruckspannung aus der Vorspannung beträgt $\sigma_{c,Ende}$ = – 4,6 MPa.

the whole structure and concreting of the joints in situ. The strands were inserted in the ducts and stressed at intermediate anchorages located in block-outs on the roof ridge, which were subsequently filled with concrete and are not visible in the final state (Fig. 11). All cross sections are compressed under dead and live loads: the average concrete compression stress due to post-tensioning is $\sigma_{c,End}$ = – 4.6 MPa.

Self-compacting concrete

All prefabricated elements are made of the same high-strength self-compacting concrete (SCC) of the strength class C50/60. The elements – with V- and Y-shape – were casted upside down to ensure both optimal filling of the formwork from above and a best-possible compact surface without air occlusions on the upper and outer faces of the members. For the same reasons, the maximum aggregate size was reduced to 8 mm. The fair-faced concrete surfaces are protected by hydrophobic impregnation. The directly

Fig. 6
Gerüsttürme für die temporäre Abstützung der Dachträger.
Temporary support of the roof beams using falsework.

Fig. 7
Nischen und Stahlplattenverbindung in den 3 cm breiten Längsfugen an der Dachoberkante.
Block-outs for welding plate connections in the 3 cm-wide gap on the roof ridge.

Fig.8
Rostfreie Zugglieder für die Verbindung zwischen Stiel und Sporthallendecke.
Stainless ties between the gym slab and one column beam.

Selbstverdichtender Beton
Sämtliche Fertigteile wurden aus dem gleichen hochfesten, selbstverdichtenden Beton (SVB) der Festigkeitsklasse C50/60 hergestellt. Die Fertigteile für Dach und Stiele mit V- respektive variablem Y-Querschnitt wurden mit den Sichtflächen nach unten betoniert, um die Schalung aus den Einfüllpunkten auf der Oberseite optimal zu verfüllen und damit die im Endzustand dem Regen und Wasserabfluss ausgesetzten Betonflächen möglichst kompakt und frei von Lunkern auszubilden. Aus den gleichen Gründen beträgt das Grösstkorn lediglich 8 mm. Sämtliche Sichtbetonflächen sind durch eine Tiefenhydrophobierung geschützt. Die direkt dem Regen ausgesetzten Stielflächen sind zusätzlich durch eine farblose Versiegelung geschützt. Auf der Dachaufsicht wurde vollflächig eine UV-resistente Flüssigkunststoffabdichtung auf Polyurethanbasis aufgebracht.

Konstruktive Durchbildung
Die Verbindungen zwischen dem vorfabrizierten Faltwerk und dem Ortbetontragwerk der Einbauten, rain exposed sides are treated with a supplementary transparent sealing. The roof surfaces are protected by a UV-resistant polyurethane liquid membrane.

Structural details
The connections between the precast members and the in-situ concrete structure had to be designed to take into account the estimated differential settlements and placement inaccuracies. The stainless tension ties (D = 40 mm) that transfer the thrust of every frame unit to the gym's floor slab, were previously inserted into tubular openings placed in the slab, then connected to the column beams after their erection (Fig. 8). A 20 mm space all around the tie allows for settlements.
The base point connection of the column beams is realised by corresponding steel plates encased in the column and in the foundation beam, which were welded during erection. The gap between the column base and the foundation was grouted afterwards by a high-strength cement grout. Inaccuracies were compensated in advance through levelling and if die unabhängig voneinander fundiert sind, mussten so entworfen werden, dass sie Ausführungstoleranzen und Differenzialsetzungen aufnehmen können. Die rostfreien Zugglieder (D = 40 mm), die den Horizontalschub von jedem Stiel zur Sporthallendecke übertragen, wurden vorgängig in Rohreinlagen der Decke eingeführt und nach Errichten des Stiels mit einem Muffenstoss angeschlossen (Fig. 8). Ein freier Zwischenraum von 20 mm um den Stab ermöglicht allfällige Differenzialsetzungen. Stiel und Fundationsbankett werden durch das Verschweissen von eingelegten Stahlplatten bei der Montage verbunden. Die schmale Fuge zwischen Stielfuss und der Fundamentaussparung wird anschliessend durch einen hochfesten Vergussmörtel verfüllt. Die Ausführungstoleranzen wurden vorgängig durch das Nivellieren der Einlagen im Bankett und bei Bedarf mit Schiftplatten ausgeglichen.

Herstellung
Für die Herstellung der kurzen und langen Stiele sowie für die

Fig.9
Zugglieder in der Diagonalscheibe unter Dach im Rahmeneck vor dem Betonieren der Aussparung.
Tension tie in the diagonal panel at the frame corner before concreting.

Fig. 10
Vorspannung der Stiele im Werk, Ankernische und Stahl-Fussplatten.
Post-tensioning of the column beams at the factory, block-out and steel base plates.
(© VSL AG, Subingen).

Fig. 11
Vorspannung der Dachträger auf der Baustelle dank Zwischenverankerungen an der Dachaufsicht.
Post-tensioning of the roof beams in situ by means of intermediate anchorages.
(© VSL AG, Subingen).

necessary by means of supplementary steel plates.

Construction

The short and long column beams as well as the midspan and lateral roof beams all required their specific steel formwork. Thus, the four element types were manufactured simultaneously in one factory and in a work cycle of 2 to 3 days per element. The concrete composition was optimised to achieve a rapid development of strength. The rotating of the heavy elements, which were cast upside down, required a special mechanism.
The handling and assembling of the precast elements was carried out with the aid of a 500 t crawler crane placed beside the hall. The roof beams were carried by temporary falsework until the post-tensioning was completed. The lowering of the falsework was controlled by a system of hydraulic jacks. The erection of the large span structure required a total time of 4 months.
The costs for the sports hall inclusive of technical equipment are of 25 million Swiss Francs.

mittleren respektive die seitlichen Fertigteile des Dachs waren insgesamt vier Schalungen erforderlich. Die vier Elementtypen konnten daher parallel in einem einzigen Werk in Zyklen von zwei bis drei Tagen pro Bauteil produziert werden. Die Betonmischung wurde für eine rasche Festigkeitsentwicklung optimiert. Das Drehen der schweren, auf der Kopfseite betonierten Fertigteile erforderte eine besondere Drehvorrichtung.
Montiert wurden die Fertigteile mithilfe eines 500-t-Raupenkrans, der neben der Sporthalle zusammengebaut worden war. Die Dachträger wurden bis zu ihrer Vorspannung von Gerüsttürmen getragen (Fig. 6). Diese wurden über ein System von hydraulischen Pressen abgesenkt, deren Kräfte sich fein steuern liessen. Die Errichtung des Faltwerks in zwei Phasen erforderte insgesamt vier Monate, die Kosten der Sporthalle inklusive Ausbau und technischer Ausrüstung betragen 25 Millionen Franken.

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Une nouvelle toiture pour le Musée Olympique à Lausanne
A new roof for the Olympic Museum at Lausanne

Aurelio Muttoni

Introduction

Dans le cadre de l'agrandissement du Musée Olympique à Lausanne (Suisse), une nouvelle toiture a été construite sur l'ancien bâtiment réalisé en 1990. La structure de cette toiture, fonctionnant aussi comme brise-soleil sur sa partie sud, est une grille de poutres en béton fibré à ultra-hautes performances. Une comparaison avec d'autres options (bois lamellé-collé, aluminium extrudé) a démontré que ce matériau peut être intéressant si tous les aspects sont considérés (économie, durabilité, facilité d'exécution, aspect architectural, poids et délais d'exécution). Des éléments de grandes dimensions (longueurs jusqu'à 21 m) ont pu être réalisés en assemblant éléments plus courts par la technique des joints conjugués et de la précontrainte par post-tension. En outre, le BFUHP permet de réaliser des éléments durables malgré leurs faibles dimensions et leur élancement.

Le Musée Olympique et sa nouvelle toiture

Le Musée Olympique, réalisé en 1990 sur la rive du Lac Léman à Lausanne, est devenu avec le temps le musée le plus fréquenté de la ville. La fréquentation accrue ainsi que des nouvelles exigences muséales ont rendu nécessaire un agrandissement et la réalisation d'une nouvelle toiture sur l'ancienne terrasse afin d'abriter un nouveau restaurant et une nouvelle salle de banquets. Sur la partie sud, la nouvelle couverture fonctionne aussi comme brise-soleil. La nouvelle structure couvre toute la partie du bâtiment orientée vers le lac sur un front de 71,25 m et une largeur de 21,00 m (Fig. 1). Les travaux ont eu lieu entre janvier 2012 et septembre 2013.

Introduction

As part of upgrading and extending the Olympic museum in Lausanne a new roof was built to cover the existing building (finished in 1990). The new roof also acts as an unusual canopy, shading natural light in the south part of the building. It consists of a grid of beams cast in ultra-high performance fibre-reinforced concrete (UHPFRC). A comparison with other options (timber and aluminium members) showed the UHPFRC solution to be the most competitive when considering all requirements of the structure (economy, durability, ease of construction, architectural expression, weight and construction details). The long lengths of the beams (up to 21 meters) were obtained by assembling shorter members using the match casting technique in combination with post-tensioning. In addition, the use of UHPFRC allowed the manufacture of durable elements, despite the limited thickness used and the pronounced slenderness of the beams.

The Olympic Museum and its new roof

The Olympic Museum was built in 1990 at the shores of Lake Geneva and today it is already the most frequently visited museum in Lausanne. It was decided to upgrade and extend the museum due to the large number of visits per year and at the same time to implement a number of new requirements for museums. An important part of the project was to build a new roof over the existing terrace to accommodate a new restaurant. On its south part, the new roof also acts as a canopy, shading natural light. The new structure covers the part of the building facing the lake with a total length of 71.25 m and a

Système porteur

La structure est une grille de poutres à trame régulière. 96 sommiers transversaux de 18 à 21 m de longueur, une hauteur de 1,00 m et un espacement de 0,75 m sont suspendus à des poutres métalliques longitudinales couvrant les 71,25 m de longueur (Fig. 2a). Ces dernières sont appuyées sur des murs en béton et sur des colonnes métalliques. Sur la partie sud orientée vers le lac, les sommiers transversaux sont en porte-à-faux sur une longueur de 4,50 m jusqu'à 9,00 m (Fig. 2b). La partie nord sur le restaurant et une partie au sud de la façade vitrée sont couvertes par une tôle à profil trapézoïdal qui assure le contreventement horizontal. A l'extrémité sud fonctionnant comme brise-soleil, la tôle est remplacée par des poutres longitudinales espacées de 0,75 m et de 0,55 m de hauteur appuyées sur les porte-à-faux fonctionnant comme raidisseurs et s'opposant ainsi au déversement des sommiers transversaux. Les sections de tous les éléments sont trapézoïdales avec largeurs variables entre 80 et 100 mm. Puisque la partie en porte-à-faux des sommiers transversaux se situe à l'extérieur, un joint en proximité de la façade Sud a dû être disposé pour les séparer thermiquement de la partie à l'intérieur. Ce joint thermique est conçu pour reprendre l'effort tranchant et le moment de flexion qui sont maximaux à cet endroit. Certains sommiers transversaux entièrement à l'extérieur ont une longueur ininterrompue de 21 m. Les autres, interrompus par le joint thermique, ont une portée variable entre 9,00 et 13,50 m à laquelle s'ajoute le porte-à-faux au-delà du joint (Fig. 2b).
Lors de la phase initiale du projet, plusieurs options ont été étudiées pour ce qui concerne la matériali-

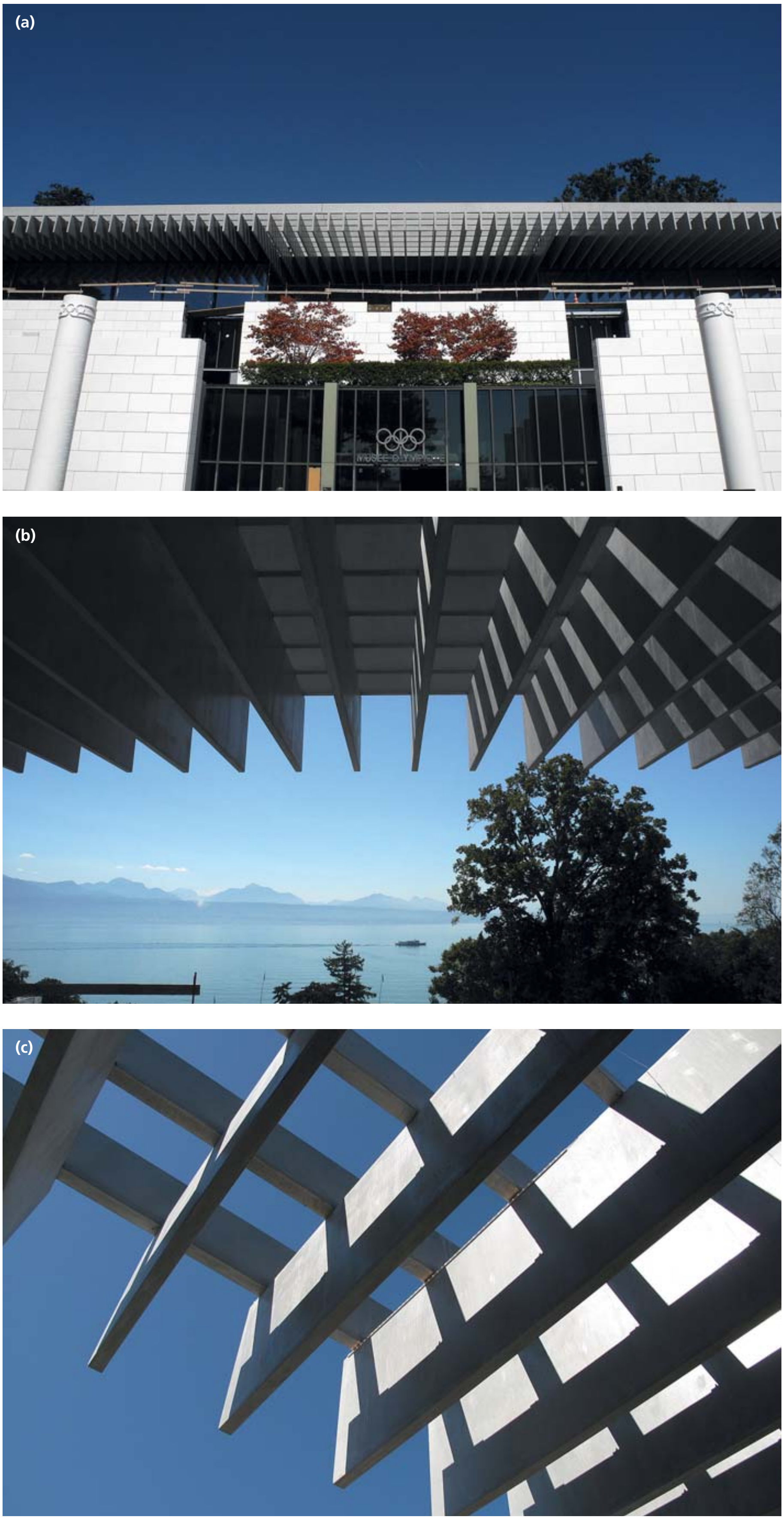

Fig. 1
a) Façade sud du Musée Olympique et nouvelle toiture pendant les travaux,
b) vue vers le lac depuis l'intérieur et c) vue de la partie fonctionnant comme brise-soleil.
a) South facade of the Olympic Museum and new roof during construction,
b) view towards the lake, and c) view of the canopy.

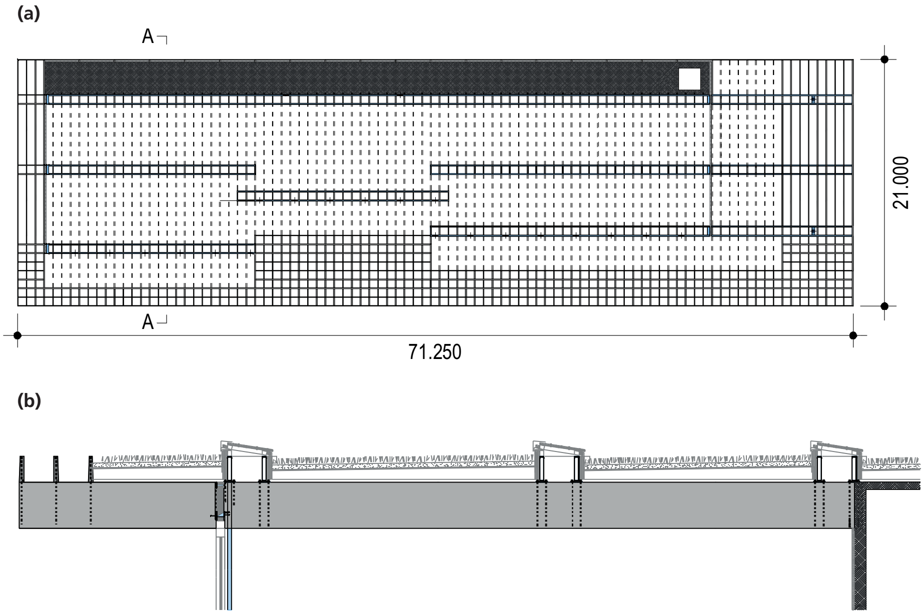

Fig. 2
a) vue en plan et b) schéma d'une poutre transversale avec portée de 13,50 m et porte-à-faux de 4,50 m (coupe A–A dans la vue en plan).
a) plan view and b) view of a transverse beam with a span of 13.50 m and a cantilever of 4.50 m (cross section A–A in plan view).

sation (BFUHP, bois lamellé collé, aluminium extrudé). En considérant tous les aspects économiques (coûts de production et d'entretien), techniques (complexité des détails constructifs), exécutifs (facilité de montage et possibilité d'assurer les délais), de durabilité et architecturaux, l'option en BFUHP a été retenue.

Préfabrication, montage et détails constructifs

Les éléments en BFUHP ont été préfabriqués dans une usine située à environ 80 km du chantier. Pour faciliter le décoffrage des moules métalliques (Fig. 4a), les sections des sommiers et des raidisseurs sont trapézoïdales avec épaisseurs variables entre 80 et 100 mm. A cause de l'élancement des éléments, pour faciliter le transport et le montage, les sommiers de 21 m de longueur ont été fabriqués et transportés en deux pièces de 10,50 m (Fig. 3b) et assemblées sur place au droit de joints conjugués précontraints par deux câbles mono-torons post-tendus (Fig. 3c). Pour les autres sommiers, le joint thermique a fait office de joint conjugué (Fig. 4c). Ce joint

width of 21.00 m (Fig. 1). The work was started in January 2012 and finished in September 2013.

Structural system

The structure of the roof consists of a regular grid of beams. In the transverse direction, the beams have a length varying between 18 and 21 m. They are 1.00 m in height and are spaced at 0.75 m. These beams are suspended from steel girders covering the 71.25 m length of the roof (Fig. 2a). The steel girders are supported on concrete walls and steel columns. In the south part, facing the lake, the transverse beams have cantilevers ranging from 4.50 m to 9.00 m (Fig. 2b). To ensure lateral stability, the north part (over the restaurant) and some regions of the south part are covered by a steel sheet of trapezoidal shape. This steel sheet is replaced in the outermost south regions by longitudinal stiffeners, spaced at 0.75 m and with a height of 0.535 m, supported on the cantilevers and acting as diaphragms (to resist lateral instability of the transverse beams). All sections were trapezoidal-shaped with a thickness

est composé de deux tubes en acier inoxydable dans lesquels trouvent place les deux torons dans la partie supérieure tendue. Ces tubes, avec les gaines des câbles de précontrainte, ont été injectés par coulis de ciment après mise en tension des câbles sur le chantier. Dans la partie inférieure des sommiers, la force de compression est reprise par des plaques en acier inoxydable soudé.
La fixation des raidisseurs sur les sommiers transversaux a été faite par le biais de plaques en acier inoxydable fixées aux raidisseurs par des tiges lors du bétonnage (Fig. 4a et 4b). Lors du montage, les plaques ont été boulonnées sur les sommiers inférieurs par le biais d'écrous vissés sur des tiges filetées en attente (Fig. 3d). Le même détail a été utilisé pour suspendre les sommiers transversaux aux poutres métalliques longitudinales (Fig. 4c).
La résistance à la flexion des sommiers transversaux est assurée essentiellement par les torons post-tendus. Afin de permettre la fixation des gaines vides lors du bétonnage, des barres de montage dans le sens transversal et longi-

varying between 80 and 100 mm. Taking into account the fact that the cantilevers are exposed to environmental conditions in the south part, a joint was provided close to the south façade for thermal insulation purposes. This joint transfers the shear and bending moments, which are highest at this region, of the transverse beams. Some of the transverse beams have a length of 21 meters without joints. The others are not continuous (with the thermally insulating joint), have inner spans ranging from 9.00 to 13.50 m (Fig. 2) and are followed by the cantilever region.
During the design of the structure, several options regarding the material to be used for the roof were investigated (UHPFRC, timber, extruded aluminium). Taking into account economic aspects

tudinal ont été utilisées. A ces armatures s'ajoutent les tiges verticales nécessaires pour les fixations des éléments et la suspension des sommiers (Fig. 4a et 4b).
Les raidisseurs longitudinaux ont la même largeur (80–100 mm) mais sont de moindre hauteur (535 mm) et leur longueur varie entre 3,00 et 12,00 m. Ils ont été coulés de la même façon et la résistance à la flexion des éléments dépassant une certaine longueur est assurée par deux torons prétendus en usine (méthode des fils adhérents).

Caractéristiques du BFUHP et dimensionnement

Le « béton spécial industriel » (BSI® avec 200 kg/m^3 de fibres métalliques, l_f = 20 mm, $ø_f$ = 0,3 mm) a été utilisé pour tous les éléments en BFUHP. Aucun traitement ther-

(both production and maintenance), technical aspects (complexity of construction details), construction technique (ease of construction and limited construction time), durability issues and/or architectural needs, UHPFRC was finally selected.

Precasting, erection and detailing

The UHPFRC members were precast in a specialized factory located 80 km from the construction site. In order to enhance the ease of demoulding (Fig. 3a), the cross-sections of the beams were trapezoidal with varying thicknesses between 80 and 100 mm. Due to the slenderness of the members, and to improve transportation and erection of the beams, the beams with a total length of 21 meters were fabricated as two pieces of

Fig. 3
a) préparation du coffrage métallique, b) manutention en usine des éléments de 10,50 m de longueur, c) joint conjugué des sommiers de 21 m et d) montage des raidisseurs longitudinaux sur les sommiers transversaux.
a) arrangement of steel formwork, b) handling of the 10.50 m elements, c) match-casted joint for the 21 m beams and d) erection of longitudinal stiffeners over the transversal beams.

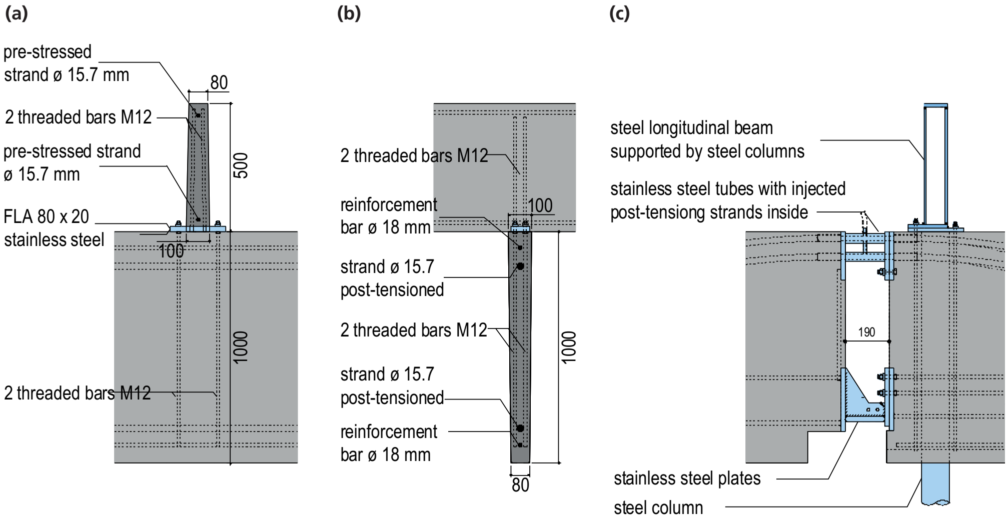

Fig. 4
a) et b) détail de la fixation des raidisseurs supérieurs sur les sommiers transversaux en dessous et c) détail du joint thermique et de la suspension des sommiers transversaux sur les sommiers métalliques longitudinaux.
a) and b) detail of joint between longitudinal and lower transversal beams and c) detail of thermally insulating joint and of the suspension of transverse beams from the steel beams.

mique n'a été appliqué lors du bétonnage. La conformité du béton a été contrôlée par des essais de compression sur cubes coulés (100x100x100 mm), par des essais de traction sur carottes (ø = 50 mm, *l* = 200 mm) prélevées d'un sommier d'essai et par essais de flexion sur prismes (90x90x400 mm) découpés du même élément (4 prismes dans le sens vertical et 4 dans le sens longitudinal) ou coulés sur place.
Les essais ont montré une variabilité relativement faible pour ce qui concerne la résistance à la compression après 28 jours (valeur moyenne = 188 MPa, écart type = 9,8 MPa à 28 jours) et la résistance à la traction de la matrice cimentaire (entre 8 et 12 MPa). Le comportement après fissuration est par contre relativement variable. Ceci est dû essentiellement à la distribution des fibres et à leur orientation peu homogène dues à la présence des armatures (barres de montage et tiges de fixation) ainsi que des gaines des câbles de précontrainte dans les sommiers transversaux et des torons prétendus dans les raidisseurs longitudinaux.
Pour cette raison, les armatures passives et prétendues ont été

10.50 m each (Fig. 3b). These pieces were later assembled at the construction site by means of the match-casted joints and post-tensioning (monostrand) tendons (Fig. 3c). For the other beams, the thermally insulating joint was used as a match-casted joint (Fig. 4c). This joint consisted of two tubes in stainless steel, where the two monostrand tendons were located (in the tension side of the member). These tubes, together with the ducts, were grouted with mortar after post-tensioning the strands. In the bottom side, the compression forces are transferred by means of welded stainless steel plates.
Assembling the transversal beams and longitudinal stiffeners was carried out using stainless steel plates fixed to the longitudinal members prior to concreting (Fig. 4a and 4b). During erection, the plates were bolted to the transverse stiffeners (by means of nuts screwed to bolts partly cast outside the concrete of the transverse beams, Fig. 3d). In the same way the transverse beams were suspended from the longitudinal steel girders (Fig. 4c).
The flexural strength of the transverse beams is ensured by the

dimensionées selon les règles classiques pour reprendre la totalité des efforts de traction à l'état limite ultime dus à la flexion, à l'effort tranchant et à la diffusion dans le plan due aux forces concentrées (forces d'ancrage des câbles post-tendus et forces introduites par les joints thermiques). La résistance assurée par les fibres a par contre été considérée pour

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post-tensioning strands. Transverse and longitudinal bars were also installed to ensure correct placing of the duct. In addition to this reinforcement, vertical bars were also arranged to fix or to suspend the beams (Fig. 4a and b). The longitudinal stiffeners have the same thickness (80–100 mm) but have a lower height (535 mm) and their length varies between 3.00 and 12.00 m. They were cast in the same way. The bending strength of the longest members was ensured by means of two strands prestressed in the factory before pouring the concrete.

UHPFRC properties and design criteria

The "béton spécial industriel" (BSI® with 200 kg/m^3 metallic fibres, l_f = 20 mm, $ø_f$ = 0.3 mm) was used for all members with UHPFRC. No thermal treatment was applied after concreting. The quality of the concrete was checked by compression tests on 100 x 100 x 100 mm cubic specimens, by tension tests on drilled cores from one specimen (ø = 50 mm, l = 200 mm) and 4-point-bending tests on prisms (90 x 90 x 400 mm) sawn from the same element. The tests exhibited a relatively low scatter with respect to the concrete compressive strength at 28 days (average value equal to 188 MPa, standard deviation equal to 9.8 MPa at 28 days) and the tensile strength of the cement matrix (between 8 and 12 MPa). The behaviour after cracking however was more variable. This is essentially due to the relatively inhomogeneous distribution and orientation of fibres because of the presence of linking threaded rods, post-tensioning ducts (with their support bars during pouring) and pre-tensioning strands.
For this reason, the ordinary and prestressed reinforcement have been designed according to the classical design methods so that they ensure resistance to all the tension forces at the ultimate state due to bending, shear and in-plane spreading of concentrated forces (anchorage forces of post-tensioning tendons and forces in the structural elements at the expansion joints). The contribution of fibres enhances (and was considered) in the bond properties of concrete and to ensure the strength of the out-of-plane spreading of the concentrated forces. The very high compressive strength was required in order to keep the dimensions very limited despite the significant concentrated forces (anchorage of tendons and thermic joints, see Fig. 4c).
In order to avoid lateral instability of the thin and slender beams, all supports were arranged on bottom of the longitudinal steel beams and the rods used to fix them to the longitudinal elements were designed accounting for the potential second order effects.

Conclusions

A detailed study of structural solutions for the new roof of the Olympic Museum allowed a comparison between a solution in ultra-high performance fibre reinforced concrete (UHPFRC) and others in timber and extruded aluminium. The timber and aluminium solutions were finally not selected for a number of reasons. This study has shown UHPFRC to be an interesting material for building roofs under demanding conditions. In addition, the UHPFRC allows producing durable elements despite thin dimensions and high slenderness. Nevertheless, it is still to be checked if the amount of fibres can be reduced accounting for the fact that for members with quite significant dimensions, the placing of ordinary or prestressed reinforcement is unavoidable to ensure sufficient strength to withstand the internal forces.

reprendre les forces d'adhérence des torons prétendus et assurer la diffusion hors du plan des forces concentrées. La résistance élevée à la compression a été indispensable afin d'assurer des épaisseurs réduites malgré l'introduction d'effort concentrés importants (ancrage des câbles et joints thermiques, voir Fig. 4c).
Afin d'empêcher le déversement des sommiers minces, tous les appuis sont disposés en dessous des poutres longitudinales métalliques (sommiers en BFUHP suspendus) et les tiges de fixation aux éléments longitudinaux (raidisseurs et poutres métalliques) ont été dimensionnées pour reprendre les efforts de second ordre qui pourraient en dériver.

Conclusions

Une étude de variantes a permis de comparer la solution en BFUHP à deux options en bois lamellé-collé et en aluminium extrudé. D'autres options avaient été écartées dans une phase préliminaire pour différentes raisons. Cette étude a démontré que le BFUHP est un matériau intéressant pour la réalisation d'une toiture avec des exigences accrues. En outre, le BFUHP permet de réaliser des éléments durables malgré leurs faibles dimensions et leur élancement. Cependant, il est encore à vérifier si le taux de fibres ne pourrait pas être réduit en considérant le fait que de toute façon, dans des éléments d'une certaine dimension, des armatures ordinaires passives ou actives sont indispensables pour reprendre les efforts plus importants.

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La salle de spectacle «Equilibre» à Fribourg

The new ”Equilibre” theatre in Fribourg

Henri Brasey, François Prongué, Jean-François Klein

Introduction

La nouvelle salle de spectacle de Fribourg sise entre l'Avenue de la Gare et le parc des Grand'Places constitue une prouesse technique. On se rend compte des caractéristiques de la salle de spectacle en considérant sa coupe longitudinale. D'une longueur totale de 63,55 m, elle se caractérise par deux porte-à-faux impressionnants; celui côté ville mesure 15,50 m à une hauteur de 21,50 m au-dessus de l'esplanade, celui côté parc des Grand'Places mesure 19,20 m pour une hauteur variant entre 7,0 m et 9,0 m (Fig. 1). Cette coupe met en évidence la logique et la simplicité du concept.

Particularités de l'ouvrage

Le porte-à-faux côté ville accueille en deux étages les bureaux de l'administration du théâtre ainsi que les salles de répétition. Il dégage une vaste place publique au pied du bâtiment (Fig. 2). Du côté parc la création du porte-à-faux est imposée par la présence d'un restaurant existant et d'un passage routier d'accès aux immeubles

Introduction

This new theatre in Fribourg, located between the Avenue de la Gare and the Grand'Places Park, is a real feat of engineering. The theatre's features are best appreciated by looking at its longitudinal section. With a total length of 63.55 m, it is characterised by two impressive overhanging parts (cantilevers); the one on the town side projects 15.50 m and is 21.50 m above the esplanade, the one facing the Grand'Places Park projects 19.20 m and is between 7.0 m and 9.0 m above the ground (Fig. 1). This view shows the logic and simplicity of the design.

The building's distinctive features

The cantilever facing the town comprises two floors and houses the offices for the theatre's administration staff and the rehearsal rooms. It makes space for a large public area at the foot of the building (Fig. 2). The cantilever on the park side was necessary because of an existing restaurant and road access to the neighbouring buildings, whilst its sloping

voisins tandis que sa forme en pente est dictée par la présence des gradins de l'auditoire de 700 places. La hauteur dégagée sous les dernières rangées permet l'aménagement d'un vaste foyer d'entracte merveilleusement ouvert sur le parc et offrant une vue imprenable sur les Alpes (Fig. 3).
Le porte-à-faux côté ville représentant un poids de l'ordre de 13 500 kN et celui du côté parc un poids d'environ 25 000 kN, la prouesse technique de l'ingénieur a été de mettre en équilibre ces deux efforts. Cela a été possible grâce à une collaboration étroite entre l'ingénieur et l'architecte ainsi que à l'utilisation de la précontrainte et celle de béton autoplaçant. La précontrainte est utilisée non seulement dans les sens longitudinal et transversal mais également dans le sens vertical. La longueur totale des câbles de précontrainte est de l'ordre de 3230 m (Fig. 4 et 6).
Une autre particularité de cet ouvrage réside dans l'utilisation de béton auto-plaçant pour limiter les nuisances lors des bétonnages et plus particulièrement ga-

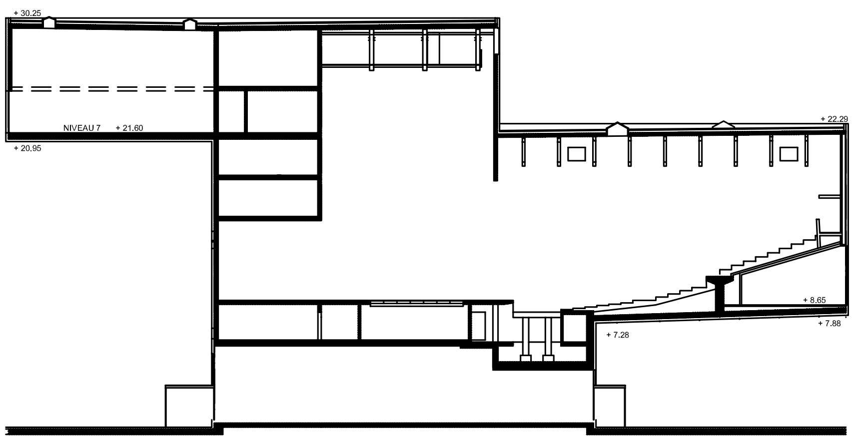

Fig. 1
Coupe longitudinale.
Longitudinal section.

Fig. 2
Le côté ville de la salle de spéctacle avec la place publique à son pied.
The side of the theatre facing the town with the public area in front.

Fig. 3
Le côté parc avec l'auditoire de 700 places surplombant le restaurant.
The park side with the 700-seat auditorium overhanging the restaurant.

shape is dictated by the tier system of the auditorium, which has a seating capacity of 700. The clear height underneath the last few rows makes space for a large interval foyer with a wonderfully open aspect, overlooking the park and offering an unrestricted view of the Alps (Fig. 3).
The cantilever on the town side produces a force of approximately 13,500 kN, and that on the park side approximately 25,000 kN, and the technical challenge facing the engineer was to balance these two forces. This was achieved by means of close collaboration between the engineer and the architect and the use of prestressing and self-placing concrete. Prestressing is used not only in the longitudinal and transverse directions but also vertically. The total length of the prestressing cables is around 3,230 m (Fig. 4 and 6).
Another particular feature of this structure is the use of self-placing concrete to minimise disruption during concreting and, even more importantly, to avoid inconvenience to the people using the multiplex cinema in the basement, on top of which part of the theatre is built.

rantir le confort des utilisateurs du cinéma multiplexe en sous-sol sur lequel se trouve partiellement la salle de spectacle.

Calcul sismique
Compte tenu de sa géométrie particulière et notamment de la présence de ses deux porte-à-faux formant un large de corps de bâtiment surélevé encastré sur une base de bâtiment étroite, un calcul sismique poussé a été réalisé. L'objectif était de déterminer les efforts dans les éléments porteurs principaux sous cette situation de risque accidentelle à l'aide de la méthode dite des spectres de réponse. Le bâtiment a donc été entièrement modélisé en trois dimensions (Fig. 5), afin d'en déterminer, dans un premier temps, ses fréquences propres de vibration et d'en tirer, après application des actions sur l'ouvrage et des accélérations sismiques prévues par la norme, les efforts internes.

Exécution
Techniquement, la construction présente quelques défis intéressants, le porte-à-faux supportant l'auditoire situé à 8 m du sol, mesure plus de 19 m de long. Un étayage provisoire pour la phase

Seismic study
Because of its unusual shape, especially the two cantilevers forming a wide elevated structure built on a narrow base, an exhaustive seismic study was carried out. The purpose of this was to determine the forces in the main load-bearing structures in this accidental risk scenario using the so-called response spectra method. The structure was modelled therefore in three dimensions (Fig. 5) in order to first determine its eigenfrequencies and, from these, to derive the internal forces and moments that would be induced in the structure under the seismic accelerations stipulated by the standard.

Execution
From a technical point of view, the building poses several interesting challenges, since the cantilever housing the auditorium is located 8 m above the ground and measures more than 19 m in length. As it was not possible to use temporary shoring during the construction phase, the construction stages were stabilised using prestressing cables, which were successively tensioned after coupling. These are arranged in the

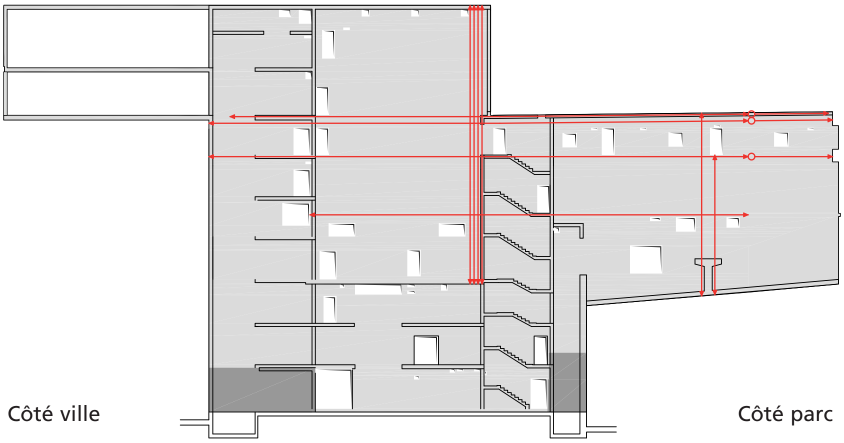

Fig. 4
Schéma de la précontrainte longitudinale et verticale.
Longitudinal and vertical prestressing layout.

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Entreprise/Contractor
Implenia Construction SA, Fribourg

de construction n'étant pas possible, c'est à l'aide des câbles de précontrainte mis en tension successivement après couplage que les étapes de construction ont pu être stabilisées. Ces derniers sont disposés dans les deux grands murs-voiles béton disposés de part et d'autre des gradins. L'unité choisie est 13 torons de 150 mm^2 (13T15S/P_0 = 2539 kN) et les mises en tension se font à l'extrémité dans le vide (Fig. 7).

Du côté ville, bien que plus haute (21 m au-dessus du sol) la construction s'est faite de manière traditionnelle à l'aide d'un échafaudage prenant emprise sur l'esplanade (Fig. 8). Les câbles de précontrainte d'unités plus petites soit 7 torons de 150 mm^2 (7T15S/P_0 = 1367 kN) sont quant à eux disposés dans les dalles constituants ainsi un élément travaillant à la traction rectiligne disposé horizontalement.

La réalisation s'est fait en 2 étapes principales: d'abord la construction classique du corps central de 28,55 m de long puis la réalisation par phases symétriques équilibrées des deux porte-à-faux côté ville et côté parc. Cette réalisation complexe a été faite en 9 phases pour le côté parc et 6 phases pour le côté ville. La méthode était celle de l'avancement avec couplage des câbles avant chaque phase.

two big concrete walls on either side of the seating tiers. The chosen unit is 13 x 150 mm^2 strands (13T15S/P_0 = 2,539 kN), whilst tensioning is carried out at the free end (Fig. 7).

On the town side, even though the cantilever is higher up (21 m from the ground) it was possible to build in the conventional manner, using falsework supported on the esplanade (Fig. 8). The prestressing cables of smaller units, i.e. 7 x 150 mm^2 strands (7T15S/P_0 = 1,367 kN) are laid in the slabs, thereby creating a horizontal element under tension.

The construction process was divided into 2 main stages: first of

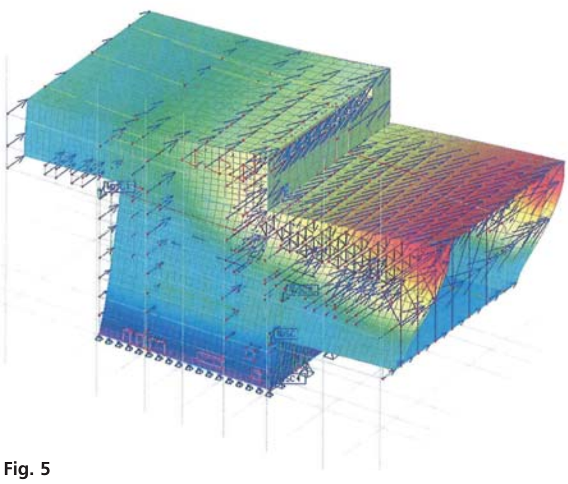

Fig. 5
Modèle statique tridimensionnel et mode vibratoire global transversal sous accélération sismique.
Structural model and global transverse vibration modes under seismic acceleration.

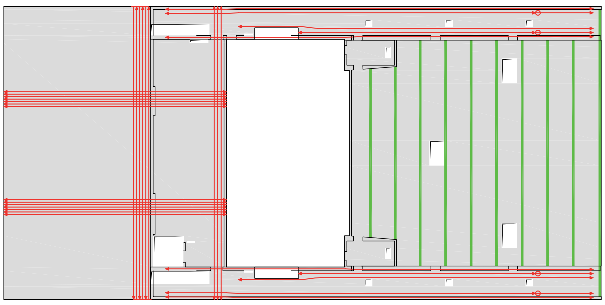

Fig. 6
Schéma de la précontrainte de la dalle du 6ème étage.
Prestressing layout for the 6th storey floor slab.

Fig. 7
Stabilisation des étapes de construction côté parc.
Supporting the construction steps on the park side.

Fig. 8
Construction traditionnelle avec échafaudage.
Conventional building with falsework.

all the conventional construction of the 28.55 m long central body and then construction of the two cantilevers on the town side and the park side in symmetrical and balanced phases. This complex procedure was achieved in 9 steps in the case of the cantilever facing the park and 6 steps for the cantilever facing the town. The method was to extend and couple the cables before each new step.

Thanks to digitisation and three-dimensional modelling, made possible by the relatively recent development of new calculation methods and computer software, it was possible to plan the 15 steps mentioned above with the requisite degree of safety.

As far as the sequence is concerned, the design studies started in October 2007 and construction lasted from June 2008 to July 2011. This three-year timescale to construct a volume of around 39,000 m^3 reflects the complexity of this building site, situated right in the centre of Fribourg.

La numérisation et les simulations tridimensionnelles rendues possibles par le développement relativement récent de nouvelles méthodes et programmes de calcul ont permis de planifier avec toute la sécurité nécessaire les 15 phases mentionnées ci-dessus.

Au point de vue historique, notons que les études ont débuté en octobre 2007 et que la réalisation a duré de juin 2008 à juillet 2011. Cette durée de 3 ans, pour un volume construit de l'ordre de 39 000 m^3, est le reflet de la complexité de ce chantier en plein centre urbain de Fribourg.

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Manufacture Cartier Horlogerie à Couvet
Cartier Horlogerie factory at Couvet

Philippe Menétrey, Jonathan Krebs

Introduction

Cartier Horlogerie, branche du groupe Richemont, est une des marques pionnières de l'horlogerie moderne dont les premières montres datent de la fin du XIX^{ème} siècle. Suite à la décision de Swatch de ne plus commercialiser ses mouvements aux marques externes au groupe, Cartier Horlogerie a choisi de renforcer son savoir-faire en matière de mouvements horlogers. Située à Couvet, dans le Val-de-Travers, la nouvelle manufacture Cartier Horlogerie est entièrement dévolue à la fabrication de mouvements horlogers et à leur assemblage.

Le bâtiment est conçu de façon à rassembler l'ensemble de la production sur un unique étage. Ce plateau, libre de toutes cloisons, est séparé en deux zones ; l'une dévolue à l'usinage des mouvements, l'autre à leur assemblage. Le bâtiment comporte trois étages. Le rez-de-chaussée est destiné aux parkings, vestiaires, installations techniques ainsi qu'à la réception. Le premier étage est l'étage de production et l'attique regroupe les espaces administratifs et le restaurant. Les dimensions du bâtiment, définies par la surface nécessaire à l'étage de production, sont importantes. La base du bâtiment est un rectangle de 80 m de long par 70 m de large. Le parti-pris architectural est d'atténuer la perception du rez-de-chaussée en réalisant un bâtiment sur pilotis. Les espaces nécessitant des murs sont ainsi regroupés dans la partie centrale du bâtiment tandis que les zones périphériques, qui accueillent les parkings, sont entièrement ouvertes.

Introduction

Cartier Horlogerie, a member of the Richemont group, is one of the pioneering brands of modern horology, the first watches of which date back to the end of the XIXth century. Following the decision of Swatch not to sell its movements to brands outside the group, Cartier Horlogerie has decided to strengthen its know-how of watch movements. Situated at Couvet, in the Val-de-Travers, the new Cartier Horlogerie factory is devoted entirely to the manufacture of watch movements and their assembly.

The building is designed in such a way that the whole of the production can be located on one floor. This area, free of all partitions, is separated into two zones, one devoted to the machining of parts of the movements and the other to their assembly. The building has three floors. The ground floor is designed for parking, cloakrooms, technical installations and the reception area. The first floor is the production floor and the attic contains the administrative offices and the restaurant.

The dimensions of the building,

Choix des matériaux et du système structural

Généralités

Hormis les enjeux inhérents au développement du projet architectural, plusieurs contraintes ont orienté la conception de la structure ; la mauvaise qualité du sol en place, la présence de radon, la charge utile importante à l'étage de production, les délais de construction du gros-œuvre limités à sept mois et les objectifs financiers du maître d'ouvrage en sont les principales.

Pour satisfaire au mieux à ces contraintes, la structure porteuse est

Fig. 1
Photomontage de la manufacture.
Photomontage of the manufacture.
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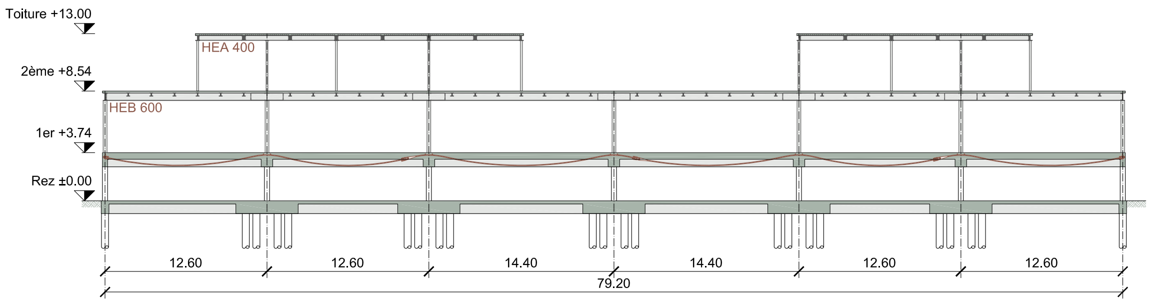

Fig. 2
Coupe longitudinale.
Longitudinal section.

defined by the surface necessary for the production floor, are large. The base of the building is a rectangle 80 m long by 70 m wide. The architectural bias is to reduce the perception of the ground floor by constructing a building on piles. The spaces necessary for the walls are thus grouped in the central part of the building while the peripheral zones, where the parking spaces are located, are entirely open.

Choice of materials and the structural system

General

Apart from the issues inherent in the development of the architectural project several constraints affected the design of the structure. The poor quality of the ground, the presence of radon, the high load-bearing capacity of the production floor, the time required to construct the building shell being limited to seven months and the financial objectives of the owner were the principle ones.
To satisfy these constraints as far as possible, the supporting structure has different functions on each level as shown on Figure 2.

Piles

The site, bordered by the Areuse to the north and the side, is characterised by a layer of about 30 m of lacustrian deposits and alluvial fluviates of soft consistency. A fringe of alluvia that is not very compact overhangs these deposits. The whole site is in a state of long-term consolidation. It was

différentiée par niveau comme visible sur la Figure 2.

Pieux

Le site, délimité par l'Areuse au nord et le versant au sud, est caractérisé par une couche d'environ 30 m de dépôts lacustres et alluvions fluviatiles de consistance molle. Une frange d'alluvions peu compactes surplombe ces dépôts.
L'ensemble du site est en consolidation lente. Il a donc été choisi d'appuyer la manufacture sur un réseau de 215 pieux à refoulement d'environ 30 m de longueur. Les conditions d'appui étant variable sur le site, les pieux ont été réalisés sur la base d'un « critère de fonçage » similaire pour tous les pieux (à l'instar des pieux battus), afin d'homogénéiser le niveau de travail des pieux et de limiter le risque de tassement différentiel. Le « critère de fonçage » a été calibré sur la base d'essais PDA.

Radier

Un radier général est disposé au-dessus des pieux. Il est renforcé par des longrines dans les deux directions. Afin de régler les problèmes d'étanchéité au radon, le radier de 25 cm est réalisé en béton étanche et ses armatures sont dimensionnées de manière à respecter les exigences accrues de fissuration. Etant donné les dimensions importantes du radier et afin de limiter la quantité d'armatures, ce denier à été réalisé en étapes dissociées puis clavé après durcissement du béton.

therefore decided to support the factory on a network of 215 piles inserted with vibratory pile divers about 30 m long. As the conditions of support varied throughout the site, the piles have been constructed on the basis of a settlement criterion similar for all the piles (following the example of the driven piles), in order to make the construction of the piles more uniform and to limit the risk of differential settlement. The settlement criterion has been calibrated on the basis of PDA tests.

Foundation slab

A general foundation slab is constructed on top of the piles. It is reinforced by longitudinal ribs in both directions. In order to control the problems of sealing to avoid radon the foundation slab of 25 cm is made of watertight concrete and its reinforcement is designed in such a way that it meets the increased requirements of cracking. In view of the large dimensions of the foundation slab and in order to limit the number of reinforcing bars, the slab has been made in separate steps and then connected together after hardening of the concrete.

Production floor slab

The production floor slab rests on columns spaced 14.40 m apart. The load-bearing capacity specified by the owner was 1 t/m^2 in order to allow plenty of flexibility in the production areas. The production floor slab is therefore made of pre-stressed concrete as described further down.

Fig. 3
Montage de la charpente sur atelier.
Assembly of the roof structure on the workshop.

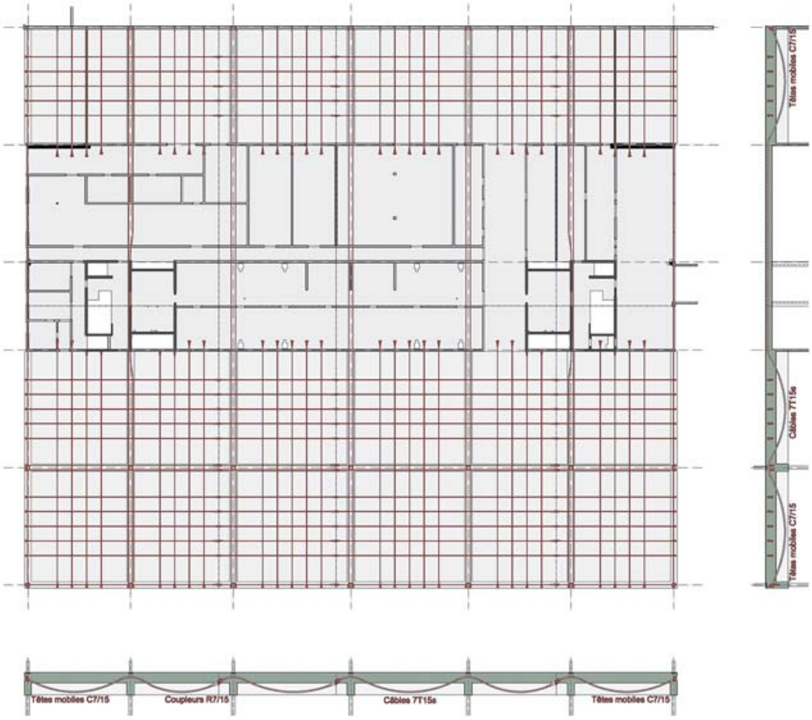

Fig. 4
Vue en plan et coupes de la précontrainte.
View in plan and sections of the pre-stressing.

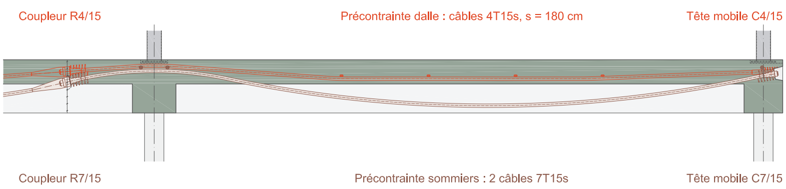

Fig. 5
Tracé et détails de la précontrainte dans le sommier et dans la dalle.
Trace and details of the pre-stressing in the beam and in the floor slab

Dalle de production
La dalle de production repose sur des colonnes espacées de 14,40 m. La charge utile définie par le maître d'ouvrage est 1 t/m^2 afin de permettre une grande liberté d'aménagement dans les espaces de production. La dalle de production est donc en béton précontraint, comme décrit ci-après.

Dalle et toiture en structure mixte acier-béton
La dalle sur atelier est une dalle mixte acier-béton permettant de franchir les grandes portées de 14,40 m. La dalle est constituée d'une charpente métallique dont les poutres principales sont de type HEB 600 disposées dans la trame des colonnes, comme illustré à la Figure 3. La charpente métallique est recouverte d'une tôle nervurée métallique. La dalle en béton a une épaisseur de 14 cm.
La dalle toiture est également une dalle mixte acier-béton avec des poutres de type HEA 400. Les colonnes du niveau administration/restaurant sont en acier. Leur section, rectangulaire, a été volontairement affinée de façon à les intégrer à la façade vitrée.

Cages d'escalier et d'ascenseur
Les cages d'escalier et d'ascenseurs sont en béton armé et font office de noyaux de stabilisation permettant de s'affranchir de contreventements en façade.

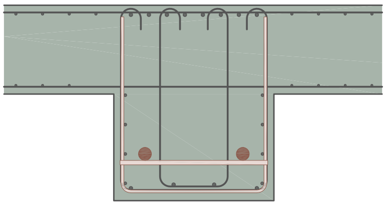

Fig. 6
Coupe transversale d'un sommier précontraint.
Transverse section through a pre-stressed lintel.

Fig. 7
Pose de la précontrainte.
Installing the pre-stressing.

Floor slab and roof in a composite steel-concrete structure

The floor slab on the workshop is a composite steel-concrete floor slab permitting the large spans of 14.40 m. The floor slab is formed of a metallic roof structure the main girders of which are of the HEB 600 type fitted in the framework of the columns, as illustrated in Figure 3. The metallic roof structure is covered by a ribbed metallic sheet. The concrete floor slab has a thickness of 14 cm.
The roof floor slab is also a composite steel-concrete floor slab with type HEA 400 girders. The columns of the offices/restaurant level are made of steel. Their rectangular cross section has been deliberately reduced so that they can be integrated into the glazed façade.

Stairwell and lift shaft

The stairwell and lift shafts are made of reinforced concrete and act as a centre for stabilisation which means that diagonal braces are not required.

Pre-stressed production floor slab

The production floor slab, designed to carry loads bidirectionally, is built on a frame of orthogonal supporting beams aligned to the frame of columns, as illustrated in Figure 4. This design enables the big spans to be bridged while providing the high load-bearing capacity. The lintels enable the efficiency of the pre-stressing to be increased and to simplify the fitting of the cables while eliminating the conflicts between the cables of the beam and of the floor slab. The floor slab has a thickness of 50 cm and the beam has a section of 90 cm wide and 110 cm high, floor slab included, which makes a very efficient structure.
The pre-stressing in the beam is made up of two cables of seven strands of T15S steel alloy Y1860S7-15.7 following a parabolic trace as shown in Figures 4 and 5. This pre-stressing, with only two cables, enables 60% of the permanent loads to be balanced.
The massive floor slab is equally pre-stressed in both directions. The pre-stressing is made up of T15S cables of four strands of the steel alloy Y1860S7-15.7 installed in flat ducts every 1.8 m. The pre-stressing of the floor slab follows a trapezoidal trace to simplify its introduction as shown in Figure 5.
The production floor slab of 5600 m^2 was constructed in eight steps, each of about 700 m^2, without any working joint. The longitudinal pre-stressing of the lintels and the pre-stressing in the floor slab is achieved with cables fitted in the factory and the connections to the concreting joints are made with couplers.

Dalle de production précontrainte

La dalle de production, conçue pour porter bidirectionnellement, est réalisée sur une trame de sommiers orthogonaux alignés sur la trame des colonnes, comme illustré à la Figure 4. Cette conception permet de franchir les grandes portées tout en supportant la charge utile élevée. Les sommiers permettent d'augmenter l'efficacité de la précontrainte et de simplifier la mise en place des câbles en supprimant les conflits entre les câbles des sommiers et de la dalle. La dalle a une épaisseur de 50 cm et les sommiers ont une section de 90 cm de large par 110 cm de haut, dalle comprise, ce qui en fait une structure très efficace.
La précontrainte des sommiers est composée de deux câbles de sept torons T15S de nuance d'acier Y1860S7-15.7 suivant un tracé parabolique comme illustré aux Figures 4 et 5. Cette précontrainte, avec seulement deux câbles, permet de balancer 60 % des charges permanentes.
La dalle massive est également précontrainte dans les deux sens. La précontrainte est composée de câbles de quatre torons T15S de nuance d'acier Y1860S7-15.7 installés en gaines plates tous les 1,80 m. La précontrainte de la dalle suit un tracé trapézoïdal afin de simplifier sa mise en œuvre, comme illustré à la Figure 5.

Fig. 8
Vue a) du dessus et b) du dessous de la dalle de production.
View a) of the upper side and b) of the underneath of the production floor slab.

La dalle de production de 5600 m^2 est réalisée en huit étapes, d'environ 700 m^2 chacune, sans aucun joint de travail. La précontrainte longitudinale des sommiers et la précontrainte en dalle sont réalisés avec des câbles montés en usine et les raccords aux joints de bétonnage sont réalisés avec des coupleurs.
La précontrainte des sommiers transversaux est réalisée avec des gaines vides dans lesquelles les câbles de précontraintes sont enfilés après bétonnage. Ceci permet de s'affranchir de coupleurs dans la zone centrale, dont l'épaisseur est réduite à 30 cm afin d'augmenter la hauteur utile dans les locaux techniques.
Pour des raisons de durabilité, les niches ont été cachetées une fois les injections des gaines réalisées.
L'utilisation de la précontrainte bidirectionnelle se justifie par les sollicitations de la structure à l'état limite ultime. Elle permet en outre d'améliorer considérablement le comportement de la dalle à l'état de service et également de diminuer de manière substantielle la quantité d'armatures passives mises en place. Le taux d'armature moyen de la dalle de production a ainsi pu être réduit à 110 kg/m^3.

Conclusion

La construction de la manufacture Cartier Horlogerie à Couvet était soumise à des exigences particulières au niveau de la surface, 5600 m^2 pour la surface de production, des portées avec une trame de 14,40 x 14,40 m, de la charge utile de 1 t/m^2, des caractéristiques géotechniques défavorables du site et de la durée de construction limitée à sept mois.
La structure porteuse a donc été différenciée par niveaux pour satisfaire à ces exigences. Une dalle en béton précontraint sur sommiers a été développée pour la dalle de production, dont la charge utile est de 1 t/m^2. Cette solution a montré toute son efficacité puisque la trame de 14,40 x 14,40 m a été franchie avec des sommiers de 1,10 m de hauteur et une dalle de 50 cm d'épaisseur.

The pre-stressing in the transverse beam is achieved with empty ducts in which the pre-stressing cables are threaded in after concreting. This makes it possible to do away with couplers in the central zone, the thickness of which is reduced to 30 cm in order to increase the usable height in the technical rooms.
The use of bidirectional pre-stressing is justified by the stresses in the structure at the ultimate limit state. It also permits the behaviour of the floor slab to be improved considerably in the service state and also to reduce substantially the quantity of passive rein-

Participants

Owner
Cartier Horlogerie, Branch of Richemont Intl. SA
Architects
A&A atelier d'architecture – Atelier A5
Civil engineers
INGPHI SA, ingénieurs en ouvrages d'art
Piles
Marti techniques de fondations SA
Reinforced concrete
F. Piemontesi SA
Pre-stressing
Freyssinet SA
Metallic roof structure and façades
Progin SA – Berisha SA

The factory in figures

Reinforced concrete	6900 m^3
Reinforcing steel	850 to
Pre-stressing steel	20 000 kg
Metallic roof structure	610 to
Volume constructed	60 000 m^3
Cost of the structures	CHF 10 600 000

Intervenants

Maitre d'ouvrage
Cartier Horlogerie, Branch of Richemont Intl. SA
Architectes
A&A atelier d'architecture – Atelier A5
Ingénieurs civils
INGPHI SA, ingénieurs en ouvrages d'art
Pieux
Marti techniques de fondations SA
Béton armé
F. Piemontesi SA
Précontrainte
Freyssinet SA
Charpente métallique et façades
Progin SA – Berisha SA

La manufacture en chiffres

Béton armé	6900 m^3
Acier d'armature	850 to
Précontrainte	20 000 kg
Charpente métallique	610 to
Volume construit	60 000 m^3
Coût des structures porteuses	CHF 10 600 000

Fig. 9
Vue du bâtiment réalisé.
View of the building constructed.

forcement installed. The average amount of reinforcement of the workshop floor slab could thus be reduced to 110 kg/m^3.

Conclusion

The design of the Cartier Horlogerie factory at Couvet was subjected to specific requirements regarding the surface area, 5600 m^2 for the production area, spans with a frame of 14.40 x 14.40 m, the load-bearing capacity of 1 t/m^2, the unfavourable geotechnical characteristics at the site and the duration of the construction time that had to be limited to seven months.
The supporting structure has therefore been designed differently for each level to satisfy these requirements. A ribbed floor slab made of pre-stressed concrete has been developed for the workshop floor slab, where the carrying capacity can be up to 1 t/m^2. This solution has shown its efficiency since the frame of 14.40 x 14.40 m has been covered with beams 1.10 m high and a floor slab 50 cm thick.
The floor slabs of the floors and of the roof have been constructed

Les dalles des étages et de toiture ont été réalisées avec une structure mixte acier-béton. Cette structure permet de franchir les portées de 14,40 m avec des profilés de type HEB 600 et une dalle de 14 cm d'épaisseur.
Ces dalles permettent ainsi de couvrir des surfaces libres de tous porteurs de 210 m^2 offrant ainsi une très grande flexibilité pour l'utilisateur.
De part sa complexité, le projet de la nouvelle manufacture Cartier Horlogerie a nécessité la mise en œuvre de structures porteuses différentes pour chaque étage, démontrant bien la puissance des structures en béton.

with a composite steel-concrete structure. This structure enables the spans of 14.40 m to be covered with sections of type HEB 600 and a floor slab of 14 cm thickness.
These floor slabs also enable a surface area of 210 m^2 to be covered thus offering great flexibility for the user.
Apart from its complexity, the project of the new factory for Cartier Horlogerie required the introduction of different load-bearing structures for each floor, demonstrating the potential of concrete structures very well.

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Erweiterungsbau Kongresszentrum Davos
Extension of the Davos Convention Centre

Joseph Schwartz

Einleitung
Das Kongresszentrum Davos wurde von renommierten Architekten in vier Bauetappen gebaut. Das erste Kongresshaus wurde 1969 von Ernst Gisel entworfen. Nach einem ersten Ausbau 1979 wurde es 1989 zum Kongresszentrum entwickelt. Mit der Erweiterung von 2009/10 durch den Basler Architekten Heinrich Degelo ist das Kongresszentrum neu gegen den Kurpark ausgerichtet und die verschiedenen Häuser sind durch eine repräsentative Wandelhalle direkt verbunden. Eine freischwebende, statisch spektakuläre Wabendecke überspannt den neuen Plenarsaal. Als Wahrzeichen für den weltberühmten Kongressort Davos wurde das markante Eingangsportal konzipiert.

Architektonisches Konzept
Die grösste Herausforderung beim 2008 durchgeführten Wettbewerb zur Erweiterung des Kongresszentrums Davos bestand darin, das bestehende Labyrinth aus

Introduction
The Davos Convention Centre was built by famous architects in four separate stages. The first Convention Hall was designed by Ernst Gisel in 1969. After the first extension in 1979 it was developed into a Convention Centre in 1989. With the extension of 2009/10 by the Basel architect Heinrich Degelo the Convention Centre is now located opposite the spa gardens and the different parts of the building are directly connected by a prestigious foyer. A statically freely suspended, spectacular honeycomb ceiling covers the new plenary hall. The striking entrance portal was designed as the emblem for the world famous Davos Convention Centre.

Architectonic design
The biggest challenge in the 2008 competition to extend the Davos Convention Centre consisted in bringing together the existing labyrinths from three buildings and an indoor swimming pool.

drei Häusern und einem Hallenbad mithilfe der neu zu ergänzenden Räume zu einem klaren Gebäudekomplex zusammenzuführen und neu zum südwestlichen Kurpark hin zu orientieren. Dies konnte nur gelingen, indem die teils chaotische innere Erschliessung in ein übersichtliches System umgewandelt wurde, das so in Erscheinung tritt, als sei es von Anfang an so geplant gewesen (Fig. 1).
Durch die Anordnung des Eingangs an die südliche Talstrasse entstand eine grosszügige Erschliessung des Gesamtkomplexes zum bestehenden Kurpark hin (Fig. 2a), mit der einzigartigen Sicht aus der Eingangshalle auf die Davoser Bergwelt. Die sechs neuen Säle im Innern ermöglichen eine zusätzliche Öffnung des Blicks auf die Schönheit der Landschaft. Das Prunkstück der Anlage ist ohne Zweifel der neue Plenarsaal für 1800 Teilnehmer, dessen elegante, freischwebende Wabendecke aus Stahlbeton sich

Bestehendes Kongresshaus
1 Eingangsbereich
2 Alter Saal
3 Hallenbad

Erweiterung Kongresshaus
4 Haupteingang/Anlieferung
5 Plenarsaal (grosser Saal)
6 Grossraumbüro
7 Wandelhalle
8 Kleine Säle
9 Lichthöfe

Fig. 1
Schematische Situation (© Degelo Architekten).
Schematic Situation (© Degelo Architekten).

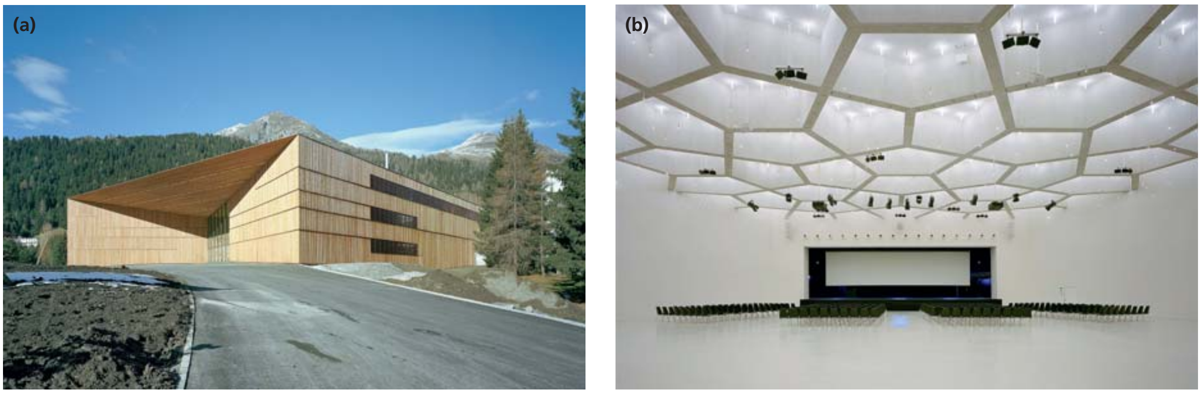

Fig. 2
Neuer Eingangsbereich des Kongresszentrums (a) und Wabendecke des Plenarsaals (b).
New entrance area of the Convention Centre (a) and honeycomb ceiling of the plenary hall (b).
© Ruedi Walti

This was done with the help of the new halls and those to be extended so that the complex forms a single building, which faces the spa gardens to the southwest. This could only be done by changing the rather chaotic inner situation into a coherent system that looks as if it had been planned like that from the start (Fig. 1).
By arranging the entrance at the southern end facing Talstrasse a large part of the total complex looked onto the existing spa garden (Fig. 2a), with a unique view from the entrance hall towards the mountains around Davos. The six new halls inside provide additional views of the beautiful countryside. The showpiece of the complex is without doubt the new plenary hall for 1800 participants. Its elegant, freely supported, honeycomb ceiling made of reinforced concrete is derived from a pentagonal shape (Fig. 2b). The model for intersecting hexagons, each dividing into four pentagons, is also known as "Cairo Pentagonal Tiling", which was used in antique mosaics.

General description of the work carried out

The existing Convention Hall was bordered on the south east side by the new building complex. In the area of the contact levels of the façade of the extension building some façade areas had to be demolished and replaced by new

von der fünfeckigen Grundrissform ableitet (Fig. 2b). Das Muster aus überkreuzten Sechsecken ist auch als «Cairo Pentagonal Tiling» bekannt, das bei den antiken Mosaiken angewendet wurde.

Allgemeine Beschreibung der Interventionen

Das bestehende Kongresshaus wurde südostseitig durch den neuen Baukörper umschlossen. Im Bereich der Berührungsebenen der Fassade zum Erweiterungsbau mussten einzelne Fassadenbereiche abgebrochen und durch neue tragende Bauteile ersetzt werden. Am Bestand waren ausserdem nur vergleichsweise geringe Anpassungen erforderlich. Im Bereich des Wandelhallenumbaus wurden einzelne Wände durch Unterzüge ersetzt.
Der Erweiterungsbau hat eine polygonale Grundrissfläche von ca. 4350 m^2 und enthält maximal vier unterschiedlich hohe Geschosse. Im Bereich des grossen Saals mit einer Fläche von ca. 1350 m^2 werden drei Geschosse zu einem

Bauherr
Landschaft Davos Gemeinde
Architekt
Degelo Architekten, Basel
Projektingenieur
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Bauausführung
Toneatti AG, Bilten

Fertigstellung
November 2010

load bearing components. For the remainder only comparatively small adjustments were necessary. In the foyer area some walls were replaced by floor beams .
The extension building has a polygonal floor plan of approx. 4350 m^2 and contains a maximum of four storeys of different heights. In the area of the large halls with an area of about 1350 m^2 three storeys were combined to give a hall with a height of about 8.5 m. The rooms arranged adjacent to the large hall and the platform can be used for various purposes. In the lowest floor on the Talstrasse side there are the technical rooms, a side stage, a catering room and cloakrooms. The remaining long area between the swimming pool and Convention Hall was left unchanged by the building work. The space under the floor slab of the large hall is used for technical ducts. There is no conventional basement.
Above the new building there are six small rooms in the spa garden storey. The rooms are separated

Owner
Landschaft Davos Gemeinde
Architect
Degelo Architekten, Basel
Project Engineer
Dr. Schwartz Consulting AG, Zug, in conjunction with DIAG Davoser Ingenieure AG, Davos
Contractor
Toneatti AG, Bilten

Completion
November 2010

Fig. 3
Die Wabendecke im Bau.
The honeycomb ceiling during construction.

ca. 8,5 m hohen Raum zusammengefasst. Die neben dem grossen Saal und dem Podium angeordneten Räume weisen unterschiedliche Nutzungen auf. Im untersten Talstrassengeschoss sind es Technikräume, eine Seitenbühne, ein Cateringraum und Garderoben. Die restliche längliche Fläche zwischen Hallenbad und Kongresshaus wird unverändert von der Anlieferung belegt. Der Raum unter der Bodenplatte des grossen Saals wird für Technikkanäle genutzt. Ein herkömmliches Untergeschoss ist nicht vorhanden.
Über der Anlieferung befinden sich sechs kleine Säle im Kurparkgeschoss. Die Säle sind durch drei innenliegende Höfe unterteilt. Die kleinen Säle sind vom südli-

by three internal courtyards. The small rooms are accessed from the southern entrance by a wide foyer. In the area of the old stage to be demolished, the new foyer is taking shape in the existing Convention Hall. Above the large hall there is a promenade floor with an open plan office and a technical room. The inclined canopy roof design consists of two slabs, which are connected by ribs that lie between them.
In general, the complete extension building is supported on a deep foundation incorporating reinforced piles made of cast-in-situ concrete with a diameter of 90 to 120 cm and lengths of between 7 and 9 m. In the area around the lift and stairs to the big hall, on

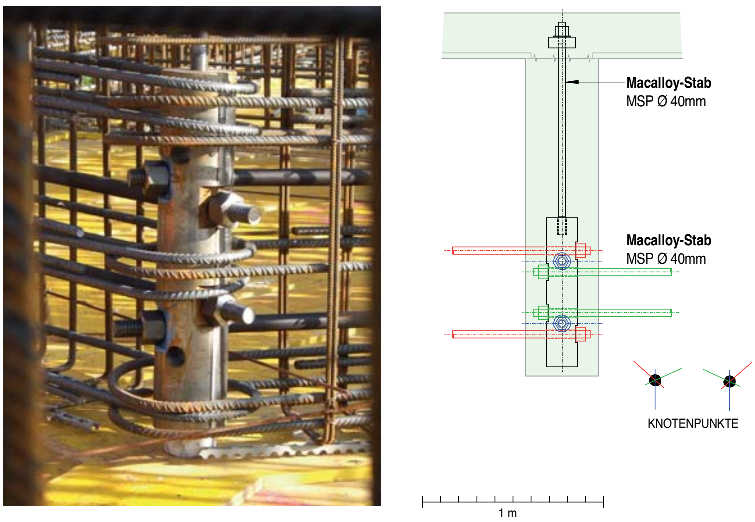

Fig. 4
Umlenkung der Zugkräfte im unteren Stegbereich: Foto und Detailskizze.
Deviating the tensile forces in the lower web area: photo and detailing.

chen Haupteingang durch eine breite Wandelhalle erschlossen. Im Bereich der alten abzubrechenden Bühne greift die neue Wandelhalle in das bestehende Kongresshaus ein. Über dem grossen Saal befinden sich im Promenadengeschoss ein offenes Grossraumbüro sowie ein Technikraum. Die geneigt verlaufende Vordachkonstruktion besteht aus zwei Platten, die durch dazwischenliegende Rippen verbunden sind.
Generell gründet der gesamte Erweiterungsbau auf einer Tieffundation mit bewehrten Bohrpfählen aus Ortbeton mit einem Durchmesser von 90 bis 120 cm und Längen zwischen 7 und 9 m. Im Bereich der Erschliessung des grossen Saals auf der Nordseite wurde eine kombinierte Pfahl-Plattengründung ausgeführt.

Besonderheiten des Tragwerks und konstruktive Lösungen

Die Tragstruktur ist in monolithisch verbundener Stahlbetonbauweise ausgebildet, wobei der Grossteil der Decken im Verbund vorgespannt wurde. Die neuen Gebäudeteile wurden zur Erzielung von Synergien betreffend Erdbebenwiderstand fugenlos mit jenen des Bestands verbunden.
Die Lastabtragung der Decke über dem grossen Plenarsaal erfolgt bei einer maximalen Spannweite von 45 m über eine räumliche Tragwirkung. Beteiligt an dieser Tragwirkung sind einerseits die als plattenbalkenartiger Rost wirkende Wabendecke selbst, sowie andererseits die Konsolwände in Kombination mit den Decken im Promenadengeschoss.
Die eigentliche Wabendeckenkonstruktion wurde auf einer horizontalen Schalungsebene in weissem Ortbeton C30/37 hergestellt (Fig. 3). Eine besondere Herausforderung stellte die Umlenkung der grossen Zugkräfte im unteren Bereich der zusammenlaufenden Stege dar. Gewählt wurde eine Lösung mit 40 mm starken Macalloy-Stäben, die schlaff eingelegt und jeweils an ihren Enden im Verbindungspunkt einen kräftigen Vollstahl-

the north side, a combined pile-raft foundation was used.

Features of the structure and design solutions

The structure consists of a monolithically connected reinforced concrete structure in which the majority of the floor slabs in the composite design are pre-stressed. The new parts of the building were connected without joints with those of the existing ones to achieve synergies regarding earthquake resistance.
The load transfer of the floor slab over the large plenary hall is achieved with a maximum span of 45 m exploiting a spatial structural system. Contributing to this support are firstly the honeycomb itself, acting as a slab-beam type of grid, and secondly the walls with corbels in combination with the floor slabs in the promenade floor.
The actual honeycomb construction was manufactured on a horizontal formwork in white cast-in-situ C30/37 concrete (Fig. 3). A particular challenge was the deviation of the large tensile forces in the lower area of the converging webs. The solution chosen involved 40 mm thick Macalloy rods, which are loosely laid and, at their ends at the connection point, pass through a strong solid steel mandrel to which they are anchored with nuts (Fig. 4). Full-scale tests on the joint were carried out in order to demonstrate the ductile behaviour of the tensile system. The slab above the honeycomb ribs was produced by means of lost formwork made of reinforced concrete elements, which had previously been fitted with lamp openings and further technical installation elements.
The flat slab above the promenade storey carries the vertical loads acting on the honeycomb ceiling. In addition to the bending effects of the honeycomb ceiling a further 3D system is activated in the open plan office of the promenade storey (Fig. 5). The lower edges of the three corbel walls made of C30/37 concrete in the hall plan form cantilevering linear sup-

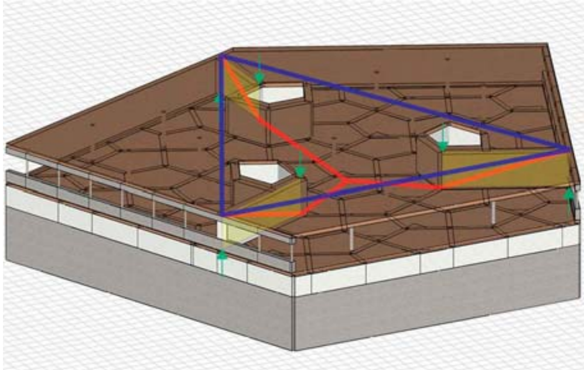

Fig. 5
Zusätzliches, räumliches Tragsystem aus Konsolwänden und Wabendecke.
Additional 3D structural system of corbel walls and honeycomb ceiling.

dorn durchdringen, an dem sie mit Muttern verankert sind (Fig. 4). Um das duktile Verhalten des Zugsystems nachzuweisen, wurden 1:1-Versuche der Verbindung durchgeführt. Die Platte oberhalb der Wabenrippen wurde mit verlorenen Schalungselementen aus Stahlbeton hergestellt, die vorgängig mit den Lampenöffnungen und weiteren technischen Installationselementen versehen wurden.
Die Flachdecke oberhalb des Promenadengeschosses trägt die Vertikallasten auf die Wabendecke ab. Zusätzlich zur Biegetragwirkung der Wabendecke wird ein weiteres räumliches Tragsystem im Grossraumbüro des Promenadengeschosses aktiviert (Fig. 5). Die Unterkanten der drei konsolartigen Wände aus Beton C30/37 bilden in den Saalgrundriss auskragende linienförmige Auflager, an denen die Wabendecke zusätzlich aufgehängt ist. Die horizontalen Auflagerkräfte dieser Konsolwände werden in die Decken eingeleitet. Die resultierenden Membrankräfte schliessen sich dabei in den Decken infolge der quasi-axialsymmetrischen Anordnung der Konsolwände in einem Kräftedreieck im Grundriss kurz. Die Membranzugkräfte werden vollständig mit Vorspanngliedern aufgenommen, die mehrheitlich dem entstehenden Kräftedreieck zwischen den Konsolwänden folgen und im Bereich hinter den Konsolwänden verankert sind

ports, on which the honeycomb ceiling is also suspended. The horizontal support forces of this corbel walls are transmitted to the roof slab. The resulting membrane forces converge in the slabs due to the quasi axisymmetric arrangement of the corbel walls in a force triangle in the ground plan. The membrane tensile forces are completely absorbed by means of prestressed members, the majority of which follow the resulting force triangle between the corbel walls and are anchored in the area behind them (Fig. 6). Both strand systems that can take tension forces of up to 3500 kN and wire systems that can take tension forces of up to 3700kN were used as tendon cables. The corbel walls are also prestressed with diagonally running tedons that produce favourable deviation forces in the serviceability state (Fig. 7). Additional central prestressing is arranged in the slab over the promenade storey, in order to control or deviate the membrane compressive forces.
The big canopy roof in the entrance area is designed as a 3D structural system, consisting of a lower and an upper reinforced concrete slab as well as the intermediate connecting webs. To ensure that the building met the physical requirements regarding thermal insulation, most of the canopy webs were made of thermally insulating lightweight concrete LC12/13.

(Fig. 6). Als Spannkabel kamen sowohl Litzensysteme mit Spannkräften bis zu 3500 kN als auch Drahtsysteme mit Spannkräften bis zu 3700 kN zum Einsatz. Die Konsolwände sind ihrerseits ebenfalls mit diagonal verlaufenden Spanngliedern vorgespannt, die für das Verhalten im Gebrauchszustand günstig wirkende Umlenkkräfte erzeugen (Fig. 7). In der Decke über dem Promenadengeschoss ist eine zusätzliche zentrische Vorspannung angeordnet, um die Membrandruckkräfte kontrolliert zu führen bzw. umzulenken.

Concluding remarks

The requirements placed on the design, the planning and the development for the extension of the Davos Convention Centre were considerably higher than for a conventional building and reached the level of bridge construction. The excellent cooperation of all parties concerned in the project – owner, architect, consulting engineer, other technical planners, building site manager, contractor, concrete supplier and post-tensioning company – enabled the building to be successfully completed.

Das grosse Vordach im Eingangsbereich ist als räumliches Tragwerk ausgebildet, bestehend aus einer unteren und einer oberen Stahlbetonplatte sowie den dazwischenliegenden verbindenden Stegen. Zur Gewährleistung der bauphysikalischen Ansprüche betreffend Wärmedämmung wurden die an den Innenbereich angrenzenden Vordachstege weitgehend in Wärmedämm-Leichtbeton LC 12/13 ausgeführt.

Schlussbemerkungen

Die Anforderungen an den Entwurf, die Projektierung und die konstruktive Ausbildung waren beim Erweiterungsbau des Kongresszentrums Davos wesentlich höher als bei einem konventionellen Hochbau und erreichten das Niveau eines Brückenbauwerks. Die hervorragende Zusammenarbeit aller am Projekt Beteiligter – Bauherr, Architekt, Bauingenieur, weitere Fachplaner, Bauleiter, Unternehmer, Betonlieferant und Spannfirma – ermöglichten die erfolgreiche Realisierung des Bauwerks.

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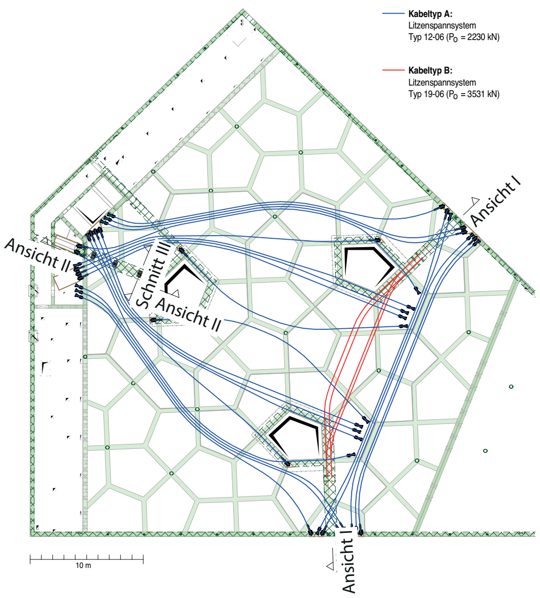

Fig. 6
Vorspannung zur Aufnahme der Membranzugkräfte.
Prestressing to withstand the membrane tensile forces.

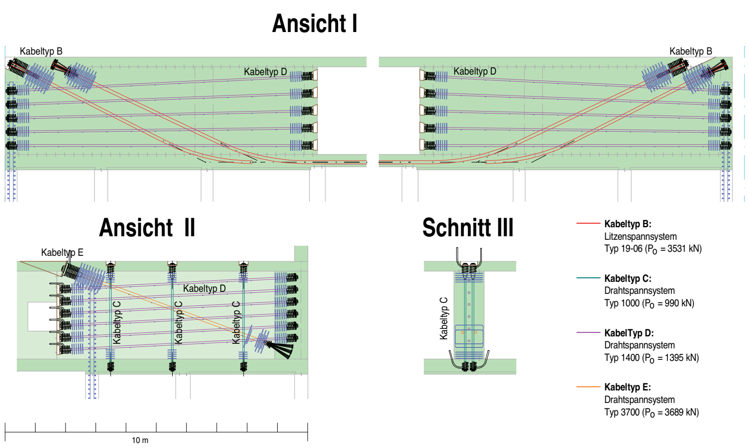

Fig. 7
Anordnung der Vorspannung in den Konsolwänden.
Arrangement of the prestressing system in the corbel walls.

Schulhaus Grono – Kraft und Form
Grono School – Force and Form

Patrick Gartmann

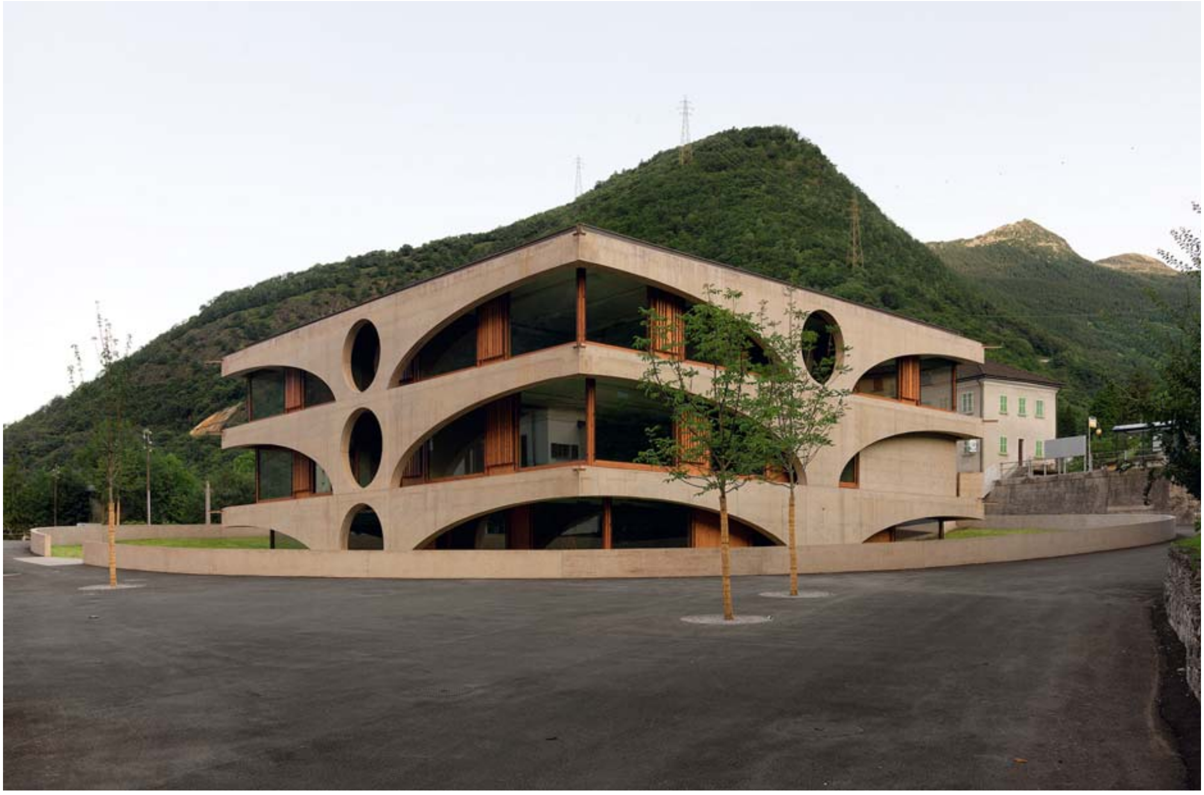

Fig. 1
Ansicht des fertiggestellten Schulhauses Grono aus Osten (© Miguel Verme).
View of the finished school at Grono from the east (© Miguel Verme).

Einleitung
In der Gemeinde Grono haben ein Architekt und ein Ingenieur gemeinsam ein Schulhaus realisiert, das durch ein aussergewöhnliches Tragwerkskonzept seinen prägnanten architektonischen Ausdruck erhält. Das Bauwerk wurde mit dem «Architektur- und Ingenieurpreis erdbebensicheres Bauen 2012» der Schweiz ausgezeichnet.

Situation
Das Dorf Grono liegt südlich des San Bernardino im Misox, einem italienischsprachigen Teil des Kantons Graubünden. Das neue Schulhaus liegt unterhalb der Hauptstrasse. Das einfach gegliederte Volumen, quadratisch im Grundriss und allseitig orientiert, ist von einer kreisrunden Mauer umgeben. Die Hanglage wird genutzt, um Schule und Kindergarten eigene Zugänge und Aussenräume zu ermöglichen: Jeweils durch eine mittige, kreisrunde Öffnung der aussenliegenden Gebäudestruktur betritt man auf der Hangseite über eine Brücke das Schulhaus und auf der Talseite den ein Geschoss tieferliegenden Kindergarten mit Hort. Im mittleren Stockwerk – dem Eingangsgeschoss des Schulhauses – befinden sich Bibliothek, Saal und Lehrerzimmer, darüber liegen die Klassenzimmer. Im Untergeschoss befinden sich Werk- und Technikräume.

Introduction
In the municipality of Grono an architect and an engineer have collaborated to build a school with an unusual supporting framework of striking architectural appearance. The structure was honoured with the Architecture and Engineer prize for earthquake-resistant buildings in Switzerland 2012.

Situation
The village of Grono lies to the south of San Bernardino in the region of Misox, an Italian speaking part of the Grisons. The new school building is located below the main street. The subdivision of the building's volume is simple; its outline is a square facing all directions, surrounded by a circular wall. The slope of the location

Zusammenarbeit Ingenieur und Architekt
Das Ziel der intensiven Zusammenarbeit zwischen Ingenieur und

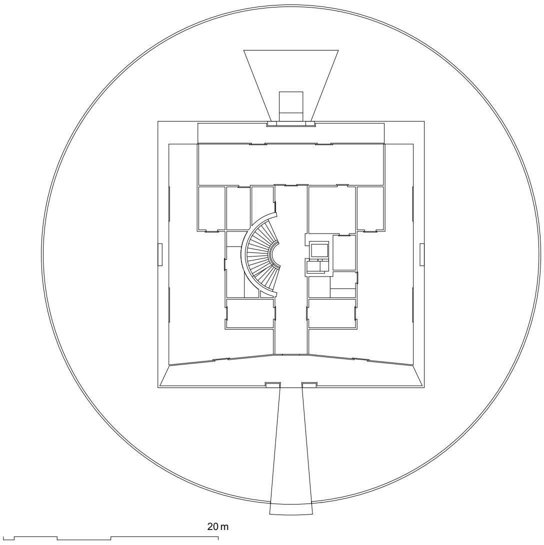

Fig. 2
Situation und Erdgeschoss (© Raphael Zuber).
Situation and ground floor plan (© Raphael Zuber).

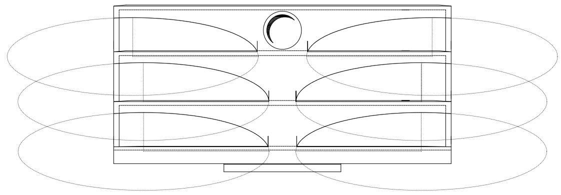

Fig. 3
Die tragende Fassadenstruktur ergibt sich aus dem Einschreiben von Ellipsen in die Wandscheiben.
The load-bearing façade structure is achieved by the production of ellipses in the wall elements.

has been used to give both the school and the kindergarten their separate entrances and outside spaces. Each entrance consists of a central, circular opening in the exterior structure of the building. While the school can be accessed on the sloping side across a bridge, the entrance to the kindergarden and nursery is located one floor below on the valley side. The middle floor of the building – where the school's entrance is located – features a library, a hall and a staffroom. The classrooms are located on the top floor. The basement contains workshops and building services.

Cooperation between engineer and architect

The objective of the close cooperation between engineer and architect was the creation of both efficient and aesthetically attractive structures. Flexibility in use, the equivalence of kindergarten and school, as well as economic aspects were given priority already at the competition design stage of the project. The engineer and the architect co-developed the building with its curved cantilevering walls, which can also be seen as arched openings straddling the corners. The shape of the façade reflects the consistent rea-

Architekt sind effiziente und zugleich formal ansprechende Strukturen. Die Flexibilität, die Gleichwertigkeit von Kindergarten und Schule sowie die Ökonomie standen bei diesem Bauwerk bereits beim Wettbewerbsentwurf im Vordergrund. Ingenieur und Architekt entwickelten gemeinsam das Bauwerk mit den geschwungenen Kragarmscheiben, die auch als übereck geführte Bogenöffnungen gelesen werden können. Die Fassade ist die konsequente formale Umsetzung eines in sich stimmigen Tragwerks. Dieser Struktur aus eingefärbtem Ortbeton sind alle anderen Elemente untergeordnet. Die Zwischenwände aus geschlämmtem Backstein, die Verglasungen und die Haustechnik folgen dabei pragmatisch den gegebenen Bedingungen. Der Ingenieur reagiert hier nicht nur auf den Architektenentwurf, sondern beeinflusst und prägt ihn, um sowohl formal wie statisch seinen Ausdruck zu schärfen. Dies ergibt einen Bau mit hoher visueller Präsenz, der die architektonischen wie ingenieurmässigen Kriterien miteinander verbindet und zeigt, wie sich die beiden Disziplinen gegenseitig bedingen. Ein starker Rohbau, der sich immer wieder seinen Nutzern anpassen

lisation of a harmonious load-bearing structure. All other elements are subordinate to this structure, which is made of coloured cast-in-situ concrete. The intermediate walls, which are made of white-washed brick, as well as the glazing and the building services follow the given conditions pragmatically. Here, the engineer not only reacts to the architectural design, but he influences and shapes it in order to sharpen its appearance both aesthetically and statically. The resulting structure has a strong visual presence that combines architectural and engineering-related criteria and demonstrates how the two disciplines are mutually dependent. The strong shell can be repeatedly adapted to meet the requirements of its users and is able to accommodate different fashion trends and preferences for the internal layout while remaining open to changes in the building's purpose.

Design of the load-bearing structure

Material

The complete load-bearing structure of the building uses concrete with a fire resistance of R 60. Concrete is extremely varied in its appearance, and if it is employed

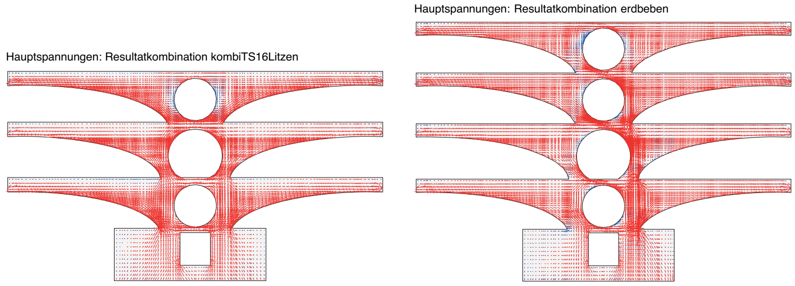

Fig. 4
Auftretende Hauptspannungen in den Fassaden.
Principal stresses in the façades.

Bauherrschaft/Client
Comune politico di Grono
Architekt/Architect
Raphael Zuber, Chur
Bauingenieur/Civil engineer
Conzett Bronzini Gartmann AG, Chur
Baumeister/Contractor
Casada SA, Malvaglia

Preis/Prize
Architektur- und Ingenieurpreis erdbebensicheres Bauen 2012/
Architect and Engineer prize for earthquake-resistant buildings 2012

kann, verschiedene Modeströmungen und Vorlieben des Innenausbaus überlebt und auch spätere Umnutzungen zulässt.

Konzept des Tragwerks

Material

Die gesamte Tragkonstruktion des Bauwerks wird in Beton mit einem Brandwiderstand von R 60 erstellt. Beton ist in seiner Erscheinung äusserst vielfältig und kann über den differenzierten Einsatz die architektonisch-räumliche und die strukturell-konstruktive Bedeutung des einzelnen Gebäudeteils unterstützen. Beton wird als tragendes und raumbildendes Material eingesetzt. Mit Farbpigmenten lässt sich der Beton zudem auf einfache Weise einfärben. Der Betonmischung wurden 3,0% gelbe und 0,6% schwarze Eisenoxidpigmente in Pulverform zugemengt, um den gewünschten Farbton der Umgebung zu erhalten.

Struktur

Die kompakte und quadratische Grundform von 25 x 25 m ist ideal für den Tragwerksentwurf dieses viergeschossigen Bauwerks, das für eine spätere Aufstockung mit einem fünften Geschoss konzipiert wurde. Vertikale Bewehrungsanschlüsse mit Kupplungen wurden dafür bereits eingebaut. Die Tragstruktur besteht aus einem Erschliessungskern, einer halbkreisförmigen Treppenwand sowie vier tragenden, vorgespannten Fassaden. Die vertikalen Lasten werden direkt über den Erschliessungskern, die Treppenwand sowie über die Fassaden in

consciously, it can highlight the architectural space and the structural significance of the individual parts of the building. At Grono School, concrete is used both as load-bearing and space-shaping material. Moreover, by adding coloured pigments, concrete can be dyed quite easily. In the case of Grono school, 3.0% of yellow and 0.6% of black iron oxide pigments in powder form was added to the concrete mixture in order to adjust it to the colour of the surrounding area.

Structure

The compact, square outline of the building measures 25 by 25 metres. It ideally supports the fra-

das Fundament geleitet. Die Kräfte der Fassade laufen zentrisch auf jeder Seite zusammen und werden somit an nur vier Stellen abgeführt. Dies ermöglicht stützenlose Stockwerke, die je nach Bedürfnis frei eingeteilt werden können, und spielt die Ecken der Geschosse frei. Die Fassaden sind damit so angeordnet, dass Verkürzungen der Decken aus Schwinden, Temperaturänderung und Vorspannung (elastisch und Kriechen) nicht zu erheblichen Zwangsbeanspruchungen der Decken führen. Diese werden als schlaff bewehrte Flachdecken in einer Stärke von 36 cm ausgeführt. Die Spannweiten betragen maximal 11 m. Vom Erdgeschoss

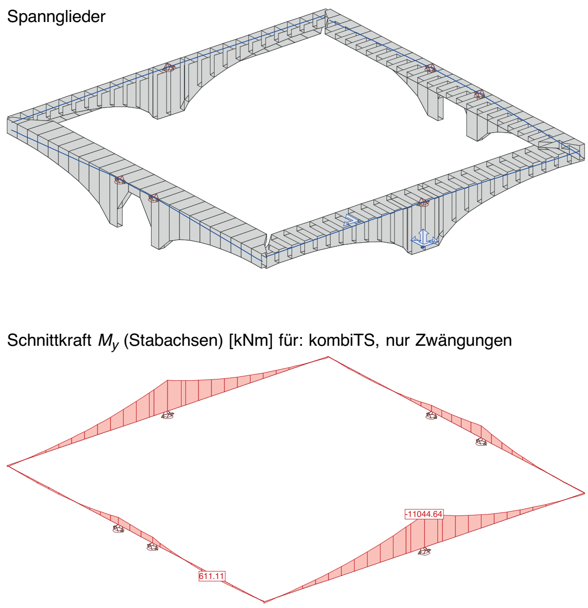

Fig. 5
Statisches Modell eines aus acht Kragarmen gebildeten Stockwerks.
Statical model (showing section forces) of storey formed of eight cantilever elements.

mework design of the four-storey structure, which has been designed so a fifth storey can be added at a later point. The necessary vertical reinforcement connections with the corresponding couplings have already been installed. The load-bearing structure itself consists of a central access core, made up of a semi-circular staircase wall and four load-bearing, prestressed façades. Vertical loads are transmitted into the foundations directly via the core, the staircase wall and the façades. The forces on the façade converge towards the centre on each side of the building and are thus transmitted in only four places. As a result, none of the floors requires any columns and can thus be organised freely according to the users' specific needs. At the same time, the corners of each storey are freed of any structural elements. This arrangement of the façades ensures that contractions of the floor slabs due to shrinkage, temperature changes and pre-stressing (elastic and creep) do not lead to significant stresses in the floor slabs. The floor slabs are constructed as reinforced flat slabs with a thickness of 36 cm and a maximum span of 11 m. All floor areas from the ground floor up to the 2nd floor can be subdivided with any number of non-load-bearing walls.

bis zum 2. Obergeschoss können die Räume mit beliebigen, nichttragenden Wänden aufgeteilt werden.

Mit tragenden Stockwerkrahmen als Fassaden lassen sich die drei Zielsetzungen an die Stabilität, die Torsionsverdrehung und die Zwängungen lösen. Die breiter aufsitzenden Fassaden (mit den kreisförmigen Aussparungen) übernehmen dank ihres grossen horizontalen Widerstands die Aussteifung gegen horizontale Einwirkungen, wie Wind und Erdbeben, in x-Richtung. In y-Richtung übernehmen dies die Treppenwand und der Erschliessungskern. Diese Anordnung erfüllt die folgenden Anforderungen:

– Für die Stabilität müssen mindestens drei vertikale Scheiben vorhanden sein.
– Zur Vermeidung grosser Torsionsbeanspruchung aus Wind (Windkraftzentrum) und Erdbeben (Massenzentrum) ist die quadratische Grundrissform ideal. Die Lage und Querschnitte der Stockwerkrahmen, des Treppenwandsegments und des Liftkerns generieren das Steifigkeitszentrum (Schubmittelpunkt und Drehzentrum). Für eine minimale Beanspruchung der Rahmen und eine geringe Gebäudetorsion liegen Windkraft-, Massen- und Steifigkeitszentrum nahe beisam-

Thanks to the use of load-bearing multi-storey frames as façades, the building's structural stability, torsional stresses and secondary bending moments are well under control: The wide-based façades with their circular cut-outs provide significant horizontal resistance in x-direction and thus stiffen the structure against horizontal effects such as wind and earthquakes. Any effects in y-direction are absorbed by the staircase wall and the central access core. This arrangement meets the following requirements:

– It ensures the stability of the structure by employing at least three vertical wall elements.
– To avoid large torsional stresses due to wind (wind force centre) and earthquakes (mass centre), a square outline is ideal. The position and cross sections of the multi-storey frames, the staircase segments and the elevator core form the structure's stiffness centre (shear centre and centre of rotation). The centres of wind force, mass and stiffness lie close together and thus ensure minimum loading of the frames and low building torsion. The actual eccentricity in the x-direction is e_x = 0.17 m and in the y-direction e_y = 2.22 m.

The structure was calculated as non-ductile with q = 2.0. The façades taper elliptically to the outside into cantilevers and thus reproduce the static forces of the concrete. The large vertical normal force (N_k = −6600 kN) from permanent loads has a very positive effect on the reinforcement content of the façade at the level of embedment of the basement (M_y = −9300 kNm). Due to the prestressing of the symmetrical cantilevers with 16 strands of 150 mm^2 cross sectional area (P_0 = 3125 kN) each, the complete façade cross section is overcompressed. The design of the structure in the front elevation thus corresponds to the stresses.

Fig. 6
Bauaufnahme: Einige Wände des Erdgeschosses sind geschalt und bewehrt.
Picture of building: Some walls of the ground floor show the formwork and the reinforcement.

Design details

The moveable and fixed anchorages for the prestressing require space. Therefore, the 40 cm

Fig. 7
Blick ins zentral angeordnete, halbkreisförmige Treppenhaus (© Miguel Verme).
View of the centrally arranged semi-circular staircase (© Miguel Verme).

men. Die tatsächlichen Exzentrizitäten betragen in x-Richtung e_x = 0,17 m und in y-Richtung e_y = 2,22 m.

Das Bauwerk wurde als nicht duktil mit q = 2,0 gerechnet. Die Fassaden verjüngen sich ellipsenförmig gegen aussen zu Kragarmen und zeichnen dadurch die statischen Kräfte des Betonbaus nach. Die grosse vertikale Normalkraft (N_k = −6600 kN) aus ständigen Lasten wirkt sich beim Einbindehorizont über dem Untergeschoss (M_y = –9300 kNm) sehr positiv auf den Bewehrungsgehalt der Fassade aus. Durch die Vorspannung der symmetrischen Auskragungen mit 16 Litzen à 150 mm^2 Querschnittfläche (P_0 = 3125 kN) wird der gesamte Fassadenquerschnitt überdrückt. Die Gestaltung der Konstruktion im Aufriss entspricht somit den Beanspruchungen.

Konstruktive Einzelheiten

Die beweglichen und festen Verankerungen der Vorspannung benötigen Platz. Die 40 cm starke Fassade wurde im Grundriss jeweils in den Ecken auf einer Länge von 4,90 m linear von 40 cm auf 90 cm nach innen verbreitert. In der Ansicht wurde die ideale Form der kubischen Parabel durch Ellipsen ersetzt, die annähernd deckungsgleich sind. Durch das Verschieben des Scheitelpunkts der Ellipse vom Eck um knapp 2,20 m zum Zentrum entstand nun die benötigte Mehrhöhe für die Verankerungen.

Die Form und Geometrie der Ellipse bleibt für alle Geschosse immer dieselbe, obwohl die Geschosshöhen unterschiedlich sind. Die Auswirkung lässt sich am jeweiligen Fassadenfuss erkennen.

Konstruktiv kann man sich die geschwungenen Kragscheiben auch als um 90° übereck geführte Bogen vorstellen. Der klassische Schluss- oder Scheitelstein bei Bogenkonstruktionen wird durch die beiden übereck gekreuzten Vorspannkabel ersetzt. Die stati-

strong façade was linearly widened in ground plan and in each of the corners over a length of 4.90 m from 40 cm to 90 cm on the inside. In elevation, the ideal shape of the cubic parabola was replaced by nearly congruent ellipses. The necessary additional height for the anchoring is produced by displacing the vertex of the ellipse from the corner by almost 2.20 m towards the centre.

The shape and geometry of the ellipse remains the same for all storeys, although the heights of the storeys vary. The effect can be seen on each of the façade feet. From the design point of view, the curved cantilevered walls can be imagined as curves that extend around the corners at right angles. The classical keystone or centre key known from traditional arch designs is replaced by the two prestressing cables crossing at right angles, reproducing the same static effect within the concrete.

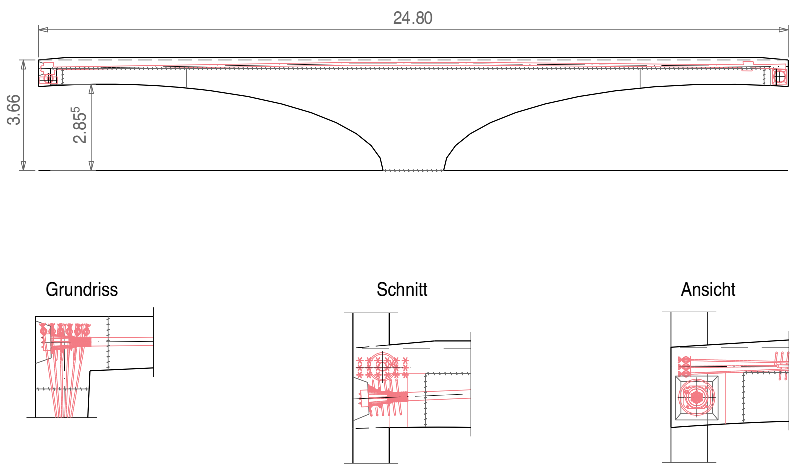

Fig. 8
Längsschnitt durch einen Fassadenbogen mit Vorspannung sowie Detail der Verschränkung der Vorspannung in der Fassadenecke.
Longitudinal section through a curved façade element with prestressing as well as a detail of the crossing of the prestressing cable in the corner of the façades.

Another special feature of the structure can be found in the floor slabs which are running from the inside to the outside. The monolithic connection of façades and floor slabs makes sense from a design point of view and does not require any cantilever plate joints. To still meet the requirements as far as building physics are concerned, an edge insulation of 1.20 m width on the soffit and full surface insulation under the supporting floor are used.

Construction work

The accurate construction of Grono school in fair-faced concrete represented a task which required close cooperation between the design and execution teams. Psychological aspects in relation to the contractors must not be neglected, for it is they who convert the ideas of the designers into an actual building. It is obvious that motivated teams who specialise in fair-faced concrete deliver better and more accurate work and thus rediscover their professional pride. Also, experience shows that the best results in a construction team are achieved if each participant accepts responsibility for the technically correct implementation of his part.

sche Wirkung im Beton ist dieselbe.

Eine andere Besonderheit sind die von innen nach aussen durchlaufenden Geschossdecken. Die monolithische Verbindung von Fassaden und Deckenplatten ist konstruktiv sinnvoll, benötigt keine Kragplattenverbindungen und lässt sich bauphysikalisch mit Randdämmungen von 1,20 m Breite an der Untersicht und vollflächigen Dämmungen unter dem Unterlagsboden lösen.

Bauausführung

Die präzise Herstellung dieses Schulhauses in Sichtbetonqualität stellte eine Aufgabe dar, die in intensiver Zusammenarbeit von Planern und Ausführenden zu lösen war. Die psychologischen Aspekte in Bezug auf die Ausführenden dürfen nicht vernachlässigt werden, denn sie sind es, die die Ideen der Planenden umsetzen. Es ist naheliegend, dass motivierte «Sichtbeton-Bauteams» bessere und präzisere Arbeit liefern und so ihren Berufsstolz wieder entdecken. Die besten Bauteamerfahrungen werden zudem erreicht, wenn jeder Ausführende die Verantwortung für die fachgerechte Umsetzung selber übernimmt.

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Canopée en béton armé à la «Maison de l'Écriture» à Montricher
Concrete Canopy of "Maison de l'Ecriture" in Montricher

Aurelio Muttoni, Miguel Fernández Ruiz

Introduction
Cet article présente une canopée en béton armé bâtie à la maison de l'Écriture à Montricher. Cette canopée est appuyée sur des colonnes préfabriquées en béton armé centrifugé avec des hauteurs allant jusqu'à 18 mètres. La forme de la canopée a été déterminée sur la base du champ de cisaillement d'un plancher dalle avec la même trame de colonnes. Les principes utilisés pour la conception et le dimensionnement de la canopée sont expliqués dans cet article, de même que les aspects principaux concernant les technologies utilisées pour sa construction.

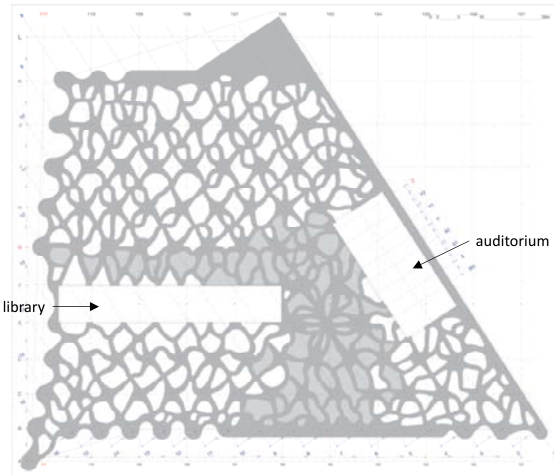

Fig. 1
Vue de la canopée de la Maison de l'Écriture
Top view of the canopy of the MdE.

La Maison de l'Écriture et sa canopée
La Maison de l'Écriture (MdE) est un centre dédié à la promotion de la littérature. Le projet est composé de deux bâtiments (une bibliothèque et un auditoire) de même que de plusieurs cabanes suspendues (actuellement en construction). La canopée de la MdE est l'un de ses symboles. Elle occupe environ 4500 m^2 et a une épaisseur de 400 mm. Elle s'appuie sur des colonnes très élancées en béton centrifugé avec des hauteurs variant entre 9 et 18 mètres. La canopée connecte les différentes parties de la MdE (Fig. 1) et offre aux cabanes suspendues des points pour leur accrochage (Fig. 2).

Introduction
An innovative concrete canopy has been designed and constructed for the Maison de l'Ecriture in Montricher. This canopy is supported by slender prefabricated columns up to 18 metres high and was designed according to the shear field of an ideal flat slab. In this paper, the principles used for the shear field design are presented as well as the main aspects concerning the various technologies used for its construction.

Conception de la canopée
Dans la forêt, une canopée n'est ni un ensemble de brins, ni une masse continue. De la même façon, la canopée de la Maison de l'Écriture n'est ni une grille de poutres, ni une dalle plate, mais

Fig. 2
Photos de la canopée avec ses colonnes préfabriquées.
Photos of the canopy of the MdE with its slender precast columns.

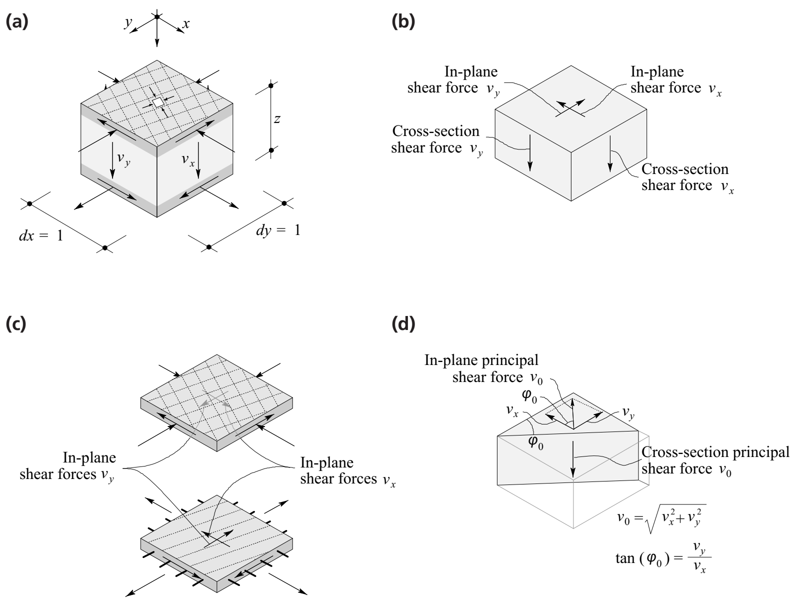

Fig. 3
Concept de champ de cisaillement et modèle sandwich : a) vue d'un élément ; b) forces agissant dans le noyau ; c) forces agissant dans les panneaux ; et d) direction et intensité de l'effort tranchant principal.
The concept of shear field. Sandwich model of a reinforced concrete slab element: a) general view of the element; b) forces acting on the core; c) forces acting on the panels; and d) magnitude and direction of the principal shear force.

The "Maison de l'Ecriture" and its canopy

The "Maison de l'Ecriture" (MdE) is a centre dedicated to literature, with the aim to preserve and to promote it. It consists of two buildings (a library and an auditorium), and a number of suspended residences (currently under construction). The canopy of the MdE is one of its symbols. This 400 mm thick structure covering 4,500 m^2 is supported on slender centrifuged concrete columns with heights varying between 9 and 18 m. It links the different parts of the MdE (Fig. 1), and offers the anchorage points for suspending the residences (Fig. 2).

Conceptual design

In a forest, a canopy is not an assembly of branches, neither a continuum mass. In the same manner, the canopy of the MdE is neither a flat slab nor an assembly of beams. Its shape expresses the theoretical location and shape where the shear forces are transmitted inside a slab towards the supports (its shear field). Thus the regions near the columns quelque chose d'intermédiaire. Sa forme transcrit la position idéale des membrures qui devraient être noyées à l'intérieur d'une dalle continue pour transmettre des charges gravitaires sur les colonnes et les murs qui la supportent. La position théorique des « brins » de cette forme complexe a été déduite à partir d'un calcul numérique, selon un modèle élastique linéaire des champs de cisaillement : le béton marque l'espace qui définit les directions principales de l'effort tranchant d'une dalle qui couvrirait l'ensemble de la Maison de l'Écriture.

Le concept du champ de cisaillement d'une dalle

Le champ de cisaillement est un champ vectoriel qui représente la direction (φ_0) et l'intensité (v_0) de la direction principal de l'effort tranchant unitaire d'une dalle. Pour des dalles en béton armé, un modèle sandwich est particulièrement adapté afin d'expliquer la signification physique de ces paramètres. Il considère la dalle divisée en trois régions (Fig. 3a) : un noyau transmettant l'effort (where shear forces are larger) become continuous. However, at a certain distance from the columns the shear forces are moderate, resulting into linear members.

The concept of the shear field of a slab

The shear field is a vector field representing the direction (φ_0) and magnitude (v_0) of the principal shear force per unit length in a slab. With respect to reinforced concrete slabs, a sandwich model is particularly useful to explain the physical meaning of such parameters. It considers a slab divided into three regions (Fig. 3a): a core carrying shear forces (Fig. 3b) and two outer panels (Fig. 3c) carrying in-plane shear and normal forces (thus equilibrating internal bending and torsional moments). With respect to the core, the shear forces per unit length acting in the cross-section (v_x and v_y) are in equilibrium with the in-plane shear forces developed in the upper and in the lower faces of the core, see Figure 3b. Such in-plane shear forces are in

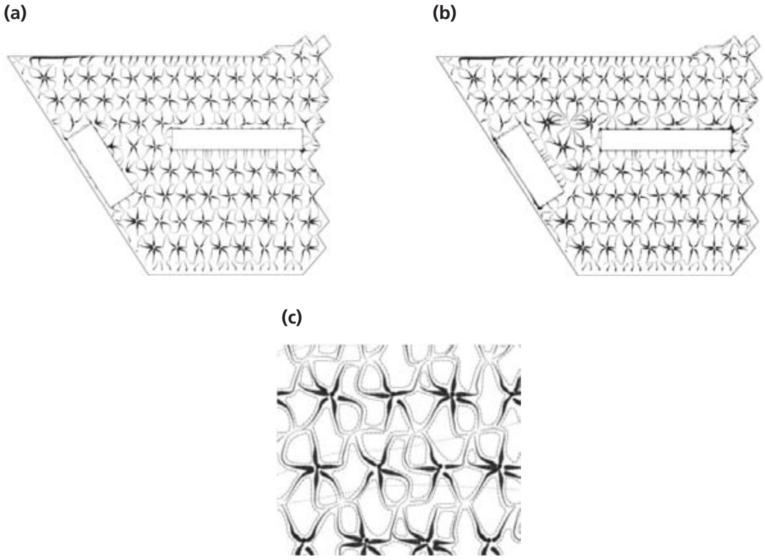

Fig. 4
Analyse du champ de cisaillement de la canopée: a) trame regulière; b) trame sans colonne entre la bibliothèque et l'auditoire; et c) détermination de la géométrie à partir du champ de cisaillement.
Shear field analysis of the canopy: a) regular column spacing; b) one column eliminated between library and auditorium; and c) obtaining the canopy geometry.

tranchant et deux panneaux extérieurs (Fig. 3c), qui équilibrent la torsion et les moments de flexion. Au sein du noyau, les deux composantes de l'effort tranchant unitaire (v_x et v_y) sont en équilibre avec les efforts rasants développés dans les panneaux supérieur et inférieur (Fig. 3b).

Les composantes unitaires de l'effort tranchant (v_x et v_y) sont ainsi les composantes de l'effort tranchant unitaire agissant, qui peut également être défini par son intensité (v_0) et sa direction (φ_0), voir Figure 3d. Une manière graphique compacte de l'exprimer consiste à dessiner l'enveloppe des directions principales d'effort tranchant avec une épaisseur du trait proportionnelle à son intensité. Un tel dessin s'avère très pratique afin de comprendre le comportement statique de la structure.

Recherche de la forme pour la canopée de la MdE

Afin de trouver la forme de la canopée de la MdE, le champ de cisaillement d'une dalle plate appuyée sur la même trame de

turn in equilibrium with the force-increments acting in the panels as shown in Figure 3c. The in-plane shear forces (v_x and v_y) are two vectors whose resultant is the principal shear force, defined by its magnitude (v_0) and by its in-plane direction (φ_0), see Figure 3d.

It can be noted that the in-plane principal shear force is in equilibrium with the cross-section principal shear force in the core of the sandwich, which has the same magnitude (v_0) and develops in a plane perpendicular to the direction φ_0 (Fig. 3c,d). The shear field can be represented by a set of lines with direction φ_0 at each point and whose thickness is proportional to its magnitude (v_0). Such plots help understanding the shear forces developing in a slab and thus how the forces are transmitted to the supports of the slab.

Form-finding in the canopy of the MdE

In order to determine the shape of the canopy of the MdE the shear field of a flat slab support-

colonnes et des bâtiments était calculé. Cette analyse a permis d'adapter la forme de la canopée aux conditions de bord réels et de raffiner sa forme pour l'adapter aux besoins de l'architecture.

Après plusieurs essais, la forme de la Figure 4 était retenue comme satisfaisante des points de vue de l'architecture et du comportement statique. Une colonne entre la bibliothèque et l'auditoire était finalement enlevée afin d'améliorer la qualité de l'espace dans cette zone. Comme la Figure 4b le montre, la géométrie du champ de cisaillement était alors adaptée à cette circonstance. Une fois

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ed on the existing buildings and columns was computed. This allowed tailoring the canopy to the boundary conditions as well as refining its shape to adapt it to the architectural needs.

After a number of preliminary designs, the shape of Figure 4a was selected as satisfactory from an engineering and architectural point of view. A column between the library and auditorium was eventually removed to enhance the space in the place between them (Figure 4b), and the resulting shear field thus adapted to this situation.

Once the shear field was selected, only the required material for carrying shear was kept (refer to Fig. 4c). The moment field (bending and torsion moments) of the resulting structure was thus modified with respect to that of the continuous slab (as the top and bottom layers of the sandwich model are no longer present everywhere). However, the shear field of the continuous slab and of the canopy is still the same, as the forces in the canopy are transmitted to the supports by following the direction of the beams (thus justifying the selected procedure to find the shape of the structure).

Buildings

Besides the canopy, two conventional buildings were introduced as part of the construction, a library and an auditorium. The buildings have continuous walls of 12 m height above the soil level. Thus, the canopy also turned out to be a continuous slab at their interface (Fig. 2).

Columns

A forest canopy is supported by slender trunks in the same manner as the canopy of the MdE is also supported on a number of slender columns. These columns have variable height varying between 18 and 9 meters and with diameters varying between 450 and 350 mm. They were prefabricated in centrifuged concrete. The mechanical slenderness of the columns was kept approximately constant ($L_{cr}/Ø \approx 30$). This optimized the mechanical behaviour of the members and was in agreement with the architectural expression (refer to Fig. 2). To do so, some columns required to be clamped in the foundations while others were simply supported on the foundations (to enhanced ease of construction of the prefabricated members).

Detailed design

Once the final geometry was established, the canopy was designed by using a 3-D model of the structure (accounting for the plane and linear regions). This allowed determining the internal forces in the structure, as its structural behaviour depends on the actual placing of the members and on their linkage. On that basis, the reinforcement was designed.

The reinforcement was adapted to the various regions of the structure. For typical spans of approx. 7 m, concrete was reinforced by using ordinary reinforcement and steel fibres (20 kg/m^3, Fig. 5a). The fibres allowed reducing the required minimum reinforcement amount and helped in zones where ordinary reinforcement was difficult to place due to complex geometries. Non-prestressed strands were also used (1/4" diameter, $f_{p0.1k}$ = 1,770 MPa) to suitably reinforce the member and to provide continuous reinforcement in the nodal regions.

In the linear members, reinforcement followed the shape of the members and was composed of groups of up to 4 bars of diameter 10 mm bent "in situ". In order to resist shear, torsion and deviation forces, transverse pins were arranged on the sides of the members. This reinforcement was developed in the nodal (continuous) regions, with a classical orthogonal reinforcement layout (see Fig. 5a).

Steel heads were placed over the columns (Fig. 5b). These elements were specifically designed to provide sufficient punching shear strength and anchorages required for suspending the residences. The shear heads were composed

que le champ de cisaillement était déterminé, seulement la matière nécessaire afin de transmettre l'effort tranchant était retenue (voir Fig. 4c). Le champ des moments (moments de flexion et moment de torsion) de la structure était ainsi modifié par rapport à celui d'une dalle plate (les nappes supérieures et inférieures du sandwich n'étant plus continues partout). Cependant, le champ de cisaillement de la dalle continue et celui de la canopée restent identiques car les forces de la canopée sont amenées jusqu'aux supports suivant la direction des brins (ainsi justifiant la démarche suivie pour trouver la forme de la canopée).

Bâtiments

Outre la canopée, deux bâtiments conventionnels ont été bâtis, une bibliothèque et un auditoire. Les bâtiments ont des murs de 12 mètres de hauteur sur le niveau du sol, ce qui imposait à la canopée de devenir une surface continue à leur engagement.

Colonnes

Une canopée forestière est appuyée sur des troncs élancés de la même façon que la canopée de la MdE est appuyée sur des colonnes en béton élancées. Ces colonnes ont une hauteur variable entre 9 et 18 mètres et avec des diamètres variant entre 450 et 350 mm. Les colonnes sont préfabriqués en béton centrifugé et ont un élancement mécanique à peu près constant ($L_{cr}/Ø \approx 30$). Ce dernier critère permettait d'optimiser le comportement statique tout en respectant l'expression architecturale de l'ensemble (Fig. 2). Pour ce but, certaines colonnes étaient encastrées dans leur fondation tandis que d'autres étaient simplement appuyées (afin de simplifier la construction et la pose des éléments).

Dimensionnement

Une fois la géometrie de la structure définie, la canopée a été calculée à l'aide d'un modèle 3D de la structure (tenant compte des régions en forme de dalle et des

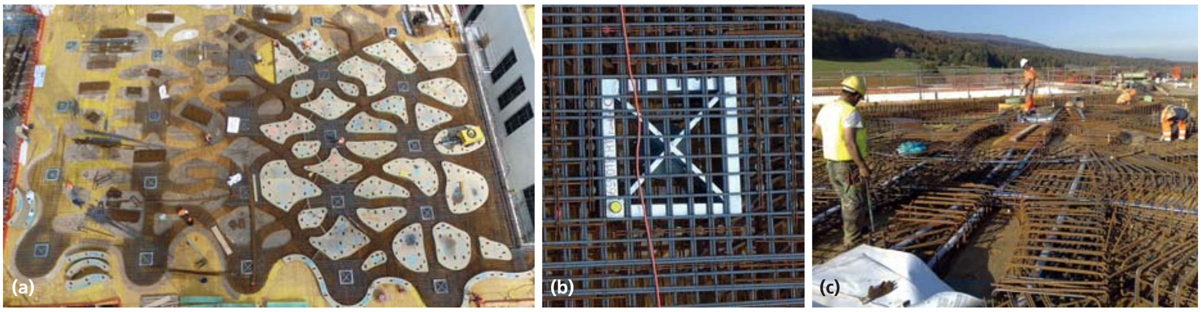

Fig. 5
Construction : a) armature ordinaire et torons non-précontraints ; b) têtes métalliques ; et c) gaînes de précontrainte.
Construction: a) placing of ordinary reinforcement and non-prestressed strands; b) steel heads; and c) post-tensioning tendons.

brins). Sur la base de ce modèle, les efforts de réduction étaient déterminés et l'armature finalement dimensionnée. L'armature a été adaptée aux différentes régions de la structure. Pour des portées typiques de 7 mètres, le béton était renforcé avec des barres d'armature ainsi que 20 kg/m^3 de fibres (voir Fig. 5a). Les fibres ont permis de réduire l'armature minimale et ont aidé dans les zones difficiles à armer. En outre, des torons non-précontraints étaient également disposés (1/4" de diamètre, $f_{p0.1k}$ = 1770 MPa) afin d'armer correctement l'élément et d'avoir une armature continue sur les zones nodales. Dans les brins, l'armature suivait la forme des membres et était composée par des groupes de 4 barres de diamètre 10 mm pliées sur place. Afin de résister à l'effort tranchant, la torsion et les forces de déviation, des épingles transversales étaient disposées de chaque côté de l'élément. Cette armature était ancrée dans les zones nodales (sur les colonnes) où l'armature était disposée suivant une trame orthogonale classique (Fig. 5a).

Sur les colonnes, des têtes métalliques étaient disposées (Fig. 5b). Ces éléments étaient conçus spécifiquement afin d'offrir une résistance suffisante au poinçonnement ainsi que pour permettre l'ancrage des cabanes suspendues. Les têtes métalliques avaient quatre profils prismatiques avec des trous filetés à l'intérieur permettant de visser les pièces d'ancrage des cabanes. Ces éléments prismatiques permettaient en outre la soudure des profils latéraux et des tôles de la tête (de 35 mm d'épaisseur).

Une zone particulière se trouve dans l'engagement entre la bibliothèque et l'auditoire, où une colonne avait été enlevée (Fig. 4b). Dans cette zone, la portée correspond au double de la portée typique et les flèches étaient très importantes par rapport au reste de la canopée. En conséquence, des câbles de post-tension furent disposés (Fig. 5c) permettant de balancer une fraction adéquate des charges permanentes et menant ainsi à des flèches comparables au reste de la structure.

of four prismatic members with threaded holes (for screwing the anchorage pieces of the residences). These prismatic members also served to weld the lateral profiles as well as the main plates (35 mm thick).

A special zone was the region between the library and auditorium, where a column was removed (Fig. 4b). In this zone, therefore, the span length was thus doubled and deflections were significantly larger than in the rest of the canopy. As a consequence, post-tensioning tendons were also placed (Fig. 5c), which allowed suitably balancing a considerable fraction of the permanent loads leading to deflections similar to those for the rest of the structure.

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Construction d'une coque en béton armé à Chiasso

Construction of an ellipsoidal concrete shell in Chiasso

Aurelio Muttoni, Franco Lurati, Miguel Fernández Ruiz

Introduction

Cet article résume les propriétés principales ainsi que les méthodes utilisées pour la construction d'une coque à forme ellipsoïdale en béton armé avec une portée maximale de 92,8 m et avec une épaisseur type de 10 cm. La structure était bétonnée avec du béton projeté (dans les zones de pente importante) et avec du béton coulé en place (pour les zones avec pente modérée ou faible). Le renforcement de la coque est composé de barres d'armature conventionnelles, de même que de fibres métalliques, de précontrainte et de clous en acier selon les différents zones et les efforts de réduction agissant.

Conception et dimensionnement

Géométrie et propriétés principales de la coque

Les dimensions de la coque ellipsoïdale sont 92,8 m (axe long) ×

Introduction

This paper summarizes the main properties and building techniques used for the construction of an ellipsoidal concrete shell with a maximum span of 92.8 m and a general thickness of 10 cm. The structure was cast both in sprayed concrete (for large slopes) and in ordinary concrete (for moderate or low slopes) and was reinforced with conventional reinforcement, metallic fibres, post-tensioning and shear studs depending on the location and internal forces of the shell.

Design

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

51,8 m (axe court) × 22,5 m (hauteur). L'ellipsoïde est sectionné par un plan horizontal qui est constitué d'un socle en béton composé de voiles verticaux transversaux, avec une hauteur totale de 18,24 m, voir Figure 1. Une description plus détaillée de la géométrie et des raisons qui la justifient peut être consultée sous [1].

L'épaisseur de la coque était variable. Une valeur de 100 mm était tenue par défaut, justifiée par de raisons constructives (enrobage minimal) et afin d'écarter les risques de voilement. Quatre nappes d'armature étaient disposées, deux à l'intrados et deux à l'extrados de la coque. Les nappes d'armature étaient orientées suivant les directions radiales (méridiens) et tangentielles (parallèles) de l'ellipsoïde. Ceci était justifié comme étant la disposition la plus efficace pour des raisons statiques. La disposition de quatre

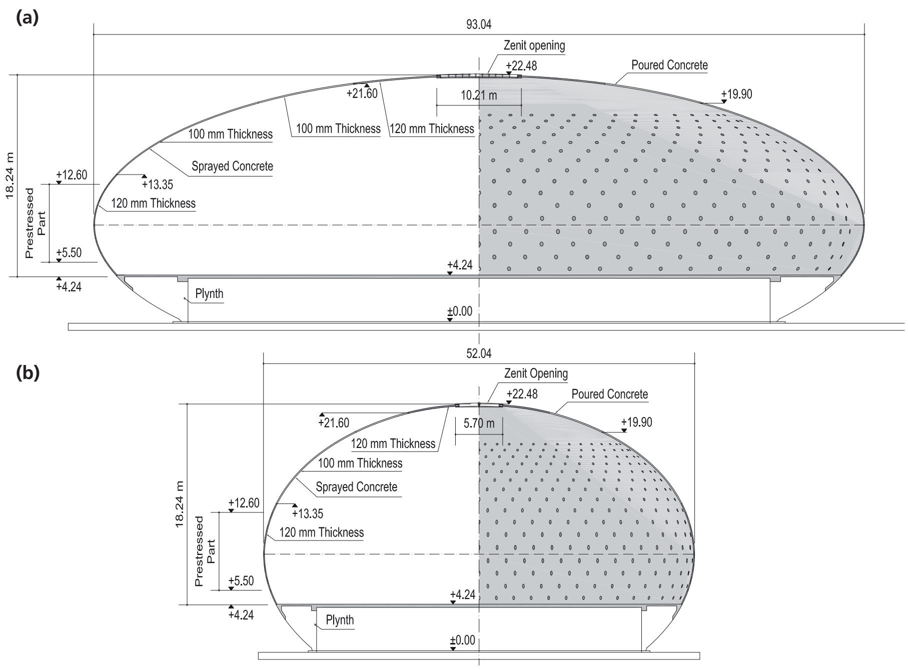

Fig. 1

Dimensions principales : a) coupe selon l'axe longue ; et b) coupe selon l'axe court.

Main geometrical dimensions: a) section along long axis; b) section along small axis.

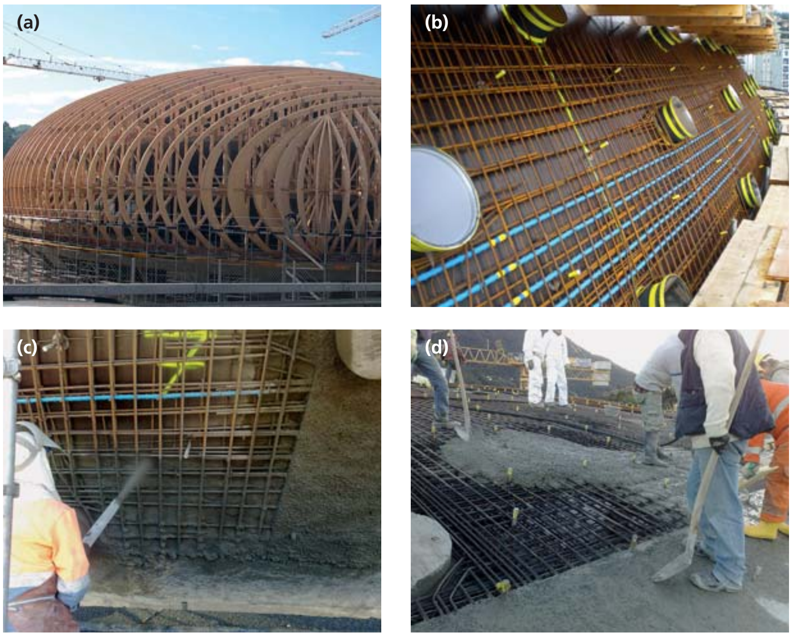

Fig. 2
Construction de la coque : a) étayage ; b) disposition de la précontrainte ; c) projection du béton ; et d) béton coulé sur place.
Construction of the shell: a) temporary scaffolding; b) placing of prestressing tendons; c) spraying of concrete; and d) pouring of concrete.

height for the shell of 18.24 m, see Figure 1. An extended description on the geometry and its reasons can be consulted elsewhere [1].
The thickness of the shell was variable. A value of 100 mm was selected as the default thickness. This was justified by constructional reasons (minimum thickness of necessary reinforcement cover) and also to ensure 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 in the radial (meridian) and tangential (parallel) directions. This was selected as the most effective layout for statical reasons. The arrangement of the four layers was required to control bending moments and shear forces developed at the basement connection, near the prestressed zone and for connecting to the steel piece placed at the zenith opening (Fig. 1). Bending moments and shear forces in other regions were quite small. Four reinforcement layers were nevertheless arranged in all regions for con-

nappes d'armature permettait le contrôle des moments de flexion et des efforts tranchants se développant à la base de la coque (intersection avec le socle), dans la zone précontrainte et pour connecter les profils en acier d'une pièce disposée dans l'ouverture zénithale de la coque (Fig. 1). Dans d'autres régions, les moments de flexion et les efforts tranchants étaient très modérés, voir négligeables. Cependant, la disposition de quatre nappes d'armature était respectée pour des raisons constructives, pour assurer un comportement correct vis-à-vis de la fissuration (pouvant se développer pour certains cas de charges) et pour écarter des risques de voilement.
En plus de l'armature conventionnelle, 35 torons de précontrainte (mono-torons de 0,6" de diamètre) étaient disposés à proximité de l'équateur de la coque (depuis le niveau +5,50 m jusqu'au niveau +12,60 m, voir Fig. 1) afin de reprendre les efforts membranaires dans la direction horizontale. L'épaisseur de la coque était, pour des raisons constructives, augmentée dans cette zone à 120 mm

structional reasons, to ensure suitable crack control (which may potentially occur 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, see Fig. 1) to carry membrane tension 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 m).
At the level of the connection to the concrete basement (between levels +4.24 and +5.14 m) shear studs were also installed 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 m to the zenith opening. On the top

Fig. 3
Ouvrage fini.
Completed work.

(entre les niveaux +4,24 m et +13,35 m).
Au niveau de la connexion avec le socle en béton (entre les niveaux +4,24 m et +5,14 m), des clous (armature transversale avec têtes d'ancrage) étaient disposés afin de garantir une résistance et capacité de déformation suffisante dans cette région (soumise à des moments de flexion et à des efforts tranchant accrus). L'épaisseur de la coque dans cette zone était augmentée à 120 mm, tout comme depuis le niveau +21,60 m jusqu'à l'ouverture zénithale (10,21 m × 5,70 m). Cette dernière augmentation d'épaisseur permet de connecter une structure métallique à travers laquelle la lumière entre dans l'espace intérieur de la coque. En outre, entre les niveaux +4,81 m et +18,78 m, des ouvertures circulaires de 0,40 m de diamètre étaient disposées, voir Figure 1.

Propriétés du béton
La structure était bétonnée avec du béton projeté depuis le niveau +4,24 m jusqu'au niveau +19,90 m. Ceci permettait d'utiliser du coffrage conventionnel (seulement d'un côté) pour toute la coque. Quand la pente était suffisamment faible (moins de 20°, depuis le niveau +19,90 jusqu'au niveau +22,48 m), le béton était coulé ordinairement. Pour les deux types de béton, la résistance caractéristique à la compression (f_{ck}) à 28 jours était spécifiée à 30 MPa.
Dans la région du béton projeté, entre les niveaux +4,24 m et +13,36 m, des fibres métalliques avec crochet (30 kg/m^3) étaient utilisées. Ces fibres avaient une

part, the increased thickness allowed the concrete shell to be connected to a steel structure placed at the zenith opening (10.21 m × 5.70 m), allowing daylight 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 Figure 1.

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 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 level +4.24 m and level +13.36 m, hooked metallic steel fibres (30 kg/m^3) were used. The

longueur de 30 mm et un rapport longueur-sur-diamètre de fibre de 80. Les fibres étaient disposées afin d'améliorer le contrôle de la fissuration (particulièrement dans la région précontrainte) et pour améliorer la capacité de déformation du béton soumis à des efforts tranchants et des forces normales accrues (zone de liaison avec le socle). Le béton projeté comportait 300 kg/m^3 de ciment et 25 kg/m^3 de chaux. Cette dernière était disposée afin d'améliorer la mise en place du béton. Des tailles de granulats entre 0 et 4 mm étaient utilisés pour 70% de la structure, le reste étant entre 4 et 8 mm. L'addition d'eau était effectué dans le pistolet de projection (projection par voie sèche).

Construction de la coque

Le coffrage de la coque était disposé sur des cintres et un étayage en bois, Figure 2a. Le coffrage était composé de panneaux en bois pliés sur place et vissés (Fig.

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Entreprise de construction/contractor
Muttoni SA, Bellinzona

Coûts/Costs
CHF 5 300 000.– (TVA et honoraires inclus/VAT and design costs inluded)

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fibres had a length of 30 mm and a length-to-diameter ratio of 80. The fibres were introduced to enhance crack control (in the post-tensioned region) and to improve the ductility of concrete under high normal and shear forces (at the connection to the basement). The sprayed concrete comprised 300 kg/m^3 of cement and 25 kg/m^3 of lean lime. The latter was to enhance the 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.

Construction of the shell

Formwork was placed against a wood scaffolding, Figure 2a. The formwork consisted of panels bent in situ and fixed in their corresponding position (Fig. 2b). The reinforcement was then placed and concrete was sprayed or poured in situ (Fig. 2c and 2d). The time required for placing of the reinforcement and concreting the shell was about 3 months in total.
After concreting, decentering of the shell was carried out. This is probably the most critical phase and has led in some cases to the collapse of a shell structure. In the present case the shell was constructed in a number of phases in order to avoid decentring 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

2b). L'armature était ensuite disposée et le béton était projeté ou coulé sur place (Fig. 2c et 2d). Le temps requis pour disposer les armatures et pour bétonner la coque était d'environ 3 mois au total.
Après le bétonnage, la coque était décintrée. Ceci est probablement l'opération la plus délicate ayant mené plusieurs fois dans le passé à des effondrements totaux ou partiels des coques. Dans le cas de la coque de Chiasso, une série de phases étaient définies afin que le décintrage ne soit pas la situation du projet déterminante. D'abord, la moitié des câbles de post-tension était mis en tension (un sur deux). Ensuite, la partie du coffrage en contact avec la zone précontrainte était enlevée, suivi par la mise en tension de la totalité des câbles de post-tension. Cette démarche permettait d'assurer le transfert correct et effectif des forces de précontrainte à la coque. Finalement, les étais verticaux étaient graduellement descendus, menant au décintrage complet de la coque. Les flèches mesurées pendant ce procédé étaient en bon accord avec les valeurs prédites. Quelques photos de la structure finalisée peuvent être observés dans la Figure 3.
Les coûts de l'ouvrage sont répartis à 49% pour le coffrage et l'étayage, 21% pour l'armature ordinaire, 5% pour la précontrainte, 24% pour le béton projeté et 1% pour le béton coulé sur place. Ceci montre que les coûts associés au coffrage et à l'étayage demeurent très importants et que des techniques plus efficaces sont encore à développer.

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 recorded during the process were in good agreement with predicted values. Some pictures of the completed work can be seen in Figure 3.
The cost of the concrete structure corresponded to 49% for the scaffolding and formwork, 21% for ordinary reinforcement, 5% for post-tensioning, 24% for the sprayed concrete and 1% for the poured in-situ concrete. This shows the relatively large cost of scaffolding and formwork for these types of structures, and points to a fruitful future research topic to obtain more efficient techniques.

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Palestra doppia a Chiasso
Double gymnasium in Chiasso

Andrea Pedrazzini, Eugenio Pedrazzini, Roberto Guidotti

Introduzione
La costruzione in oggetto s'inserisce nel contesto del campus scolastico e museale della città di Chiasso. L'edificio si compone di due elementi: una copertura a sé stante con pianta quadrata corrispondente alla sala ginnica e uno zoccolo rettangolare parzialmente interrato che si relaziona con gli edifici scolastici e il parcheggio presenti sul lato nord per mezzo di una terrazza e l'entrata, con il giardino verso sud attraverso un'ampia gradinata,con il campo stradale e la piazza verso est mediante uno sbarramento e con il m.a.x Museo e lo Spazio Officina verso ovest con un'ampia fontana.
La copertura è costituita da un solaio a cassettoni in calcestruzzo armato precompresso sospeso nel vuoto mediante quattro travi pareti appoggiate centralmente su un cavalletto a V quindi congiunte al basamento in un solo punto. Lo zoccolo funge da appoggio alla copertura e si confronta con i particolari problemi d'impermeabilità e di galleggiamento imposti dalla falda la cui quota può superare di quasi due metri la platea dello stabile.
Di seguito saranno esposte le particolarità strutturali e di messa in opera della copertura e dello zoccolo.

Introduction
The building in question is part of the school and museum complex of the city of Chiasso. The building consists of two elements: a separate square roof over the gymnasium and a partially buried rectangular plinth-like base that connects with the school buildings and parking lot on the north side by means of a terrace and the entrance, with the garden to the south by means of a wide flight of steps, with the street and square to the east by means of a barrier and with the M.A.X Museum and Workshop Space to the west with a large fountain.
The roof consists of a prestressed, reinforced concrete lattice (grid form) suspended in space by four wall beams centrally supported by V-shaped elements (pillars) joined to the base at a single point.
The plinth supports the roof and deals with the special problems regarding waterproofing and uplift due to the groundwater table, whose level can exceed the floor level of the building by almost two metres.

Considerazioni strutturali
La copertura
La sala ginnica è coperta mediante un solaio a cassettoni con 7 travi precompresse di luce 32,85 m, altezza 1,24 m (rapporto di snellezza L/H = 26,5) e spessore 35 cm in ciascuna direzione portante e un solaio di spessore compreso tra i 16 e i 18 cm staccato termicamente dalle pareti perimetrali. La precompressione delle travi è calibrata agli sforzi a flessione riscontrati nel comportamento elastico della struttura (22 trecce da 150 mm^2 nelle tre travi centrali, 19 trecce nelle due travi seguenti e da 9 trecce nelle travi di bordo) ed è disposta con andamento poligonale con deviazioni a $\frac{1}{4}$ e $\frac{3}{4}$ della luce in modo tale da prevenire l'intersezione dei cavi nei punti d'incrocio delle travi.

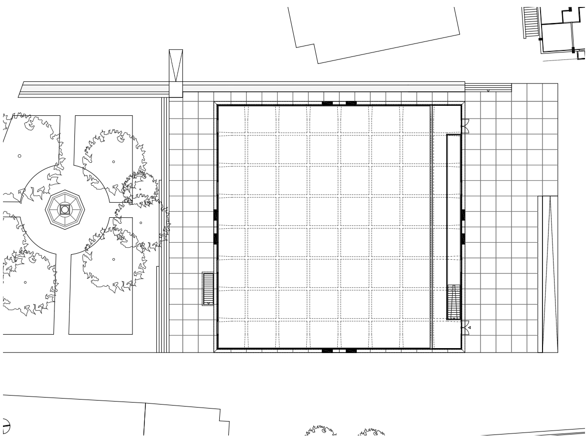

Fig. 1
Pianta piano terra.
Ground floor plan.

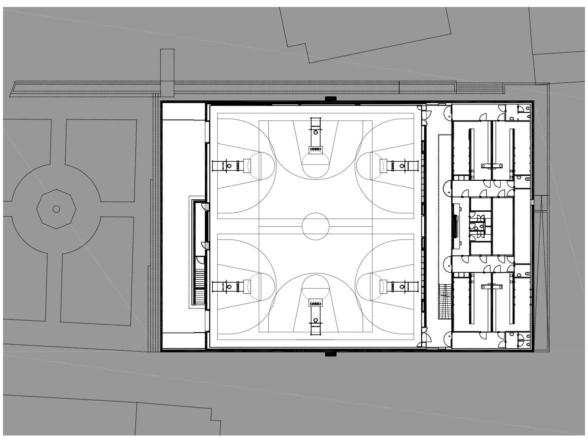

Fig. 2
Pianta livello –1.
Level –1 plan.

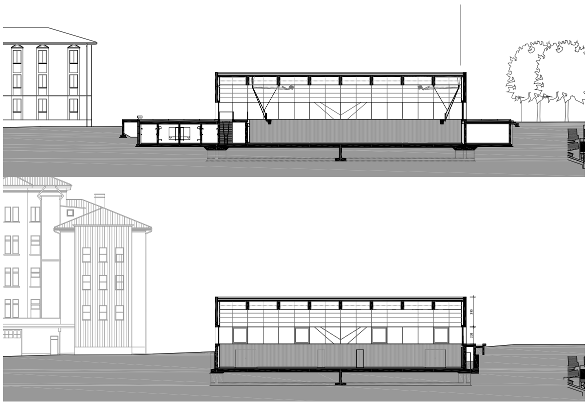

Fig. 3
Sezioni.
Sections.

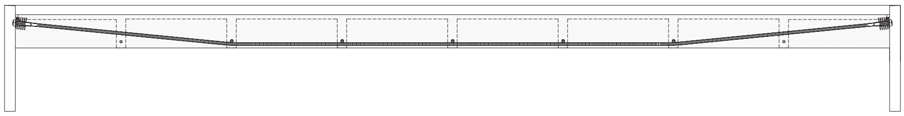

Fig. 4
Andamento cavi graticcio.
Lattice cable orientation.

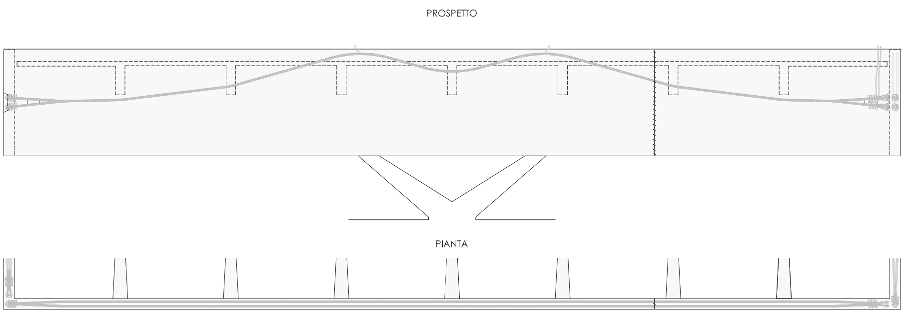

Fig. 5
Andamento cavi travi parete.
All beam cables orientation.

Below, we explain the structural and installation features of the roof and the plinth.

Structural considerations

The roof

The gymnasium is covered by a lattice ceiling with 7 prestressed beams with a span of 32.85 m, height of 1.24 m (slenderness ratio L/H = 26.5) and thickness of 35 cm in each load-bearing direction, and a slab between 16 and 18 cm thick, thermally insulated from the perimeter walls. The pre-compression of the beams is calibrated to the flexural stresses

Lungo il perimetro le travi del solaio appoggiano su 4 travi parete di altezza 3,95 m e spessore 40 cm con una precompressione tale da prevenire la fessurazione del calcestruzzo (2 cavi di 8 trecce da 150 mm^2 ciascuno).
Al centro ciascuna trave parete è disposta su una forcella (pilastro a forma di V) con struttura mista di tubolari d'acciaio (ROR 273.25/S355) e calcestruzzo capace di accogliere le installazioni tecniche e i pluviali.
Se considerate singolarmente le travi pareti risultano instabili ma la loro unione determina una

encountered in the elastic behaviour of the structure (22 strands of 150 mm^2 in the three central beams, 19 strands in the next two beams and 9 strands in the outer beams) and is arranged in a polygonal layout with offsets at $\frac{1}{4}$ and $\frac{3}{4}$ across the open space in such a way as to prevent the intersection of the cables at the points of intersection of the beams.
Along the perimeter, the roof beams rest on 4 wall beams 3.95 m high and 40 cm thick with pre-compression sufficient to prevent the cracking of the concrete (2 cables of 8 strands of 150 mm^2 each).

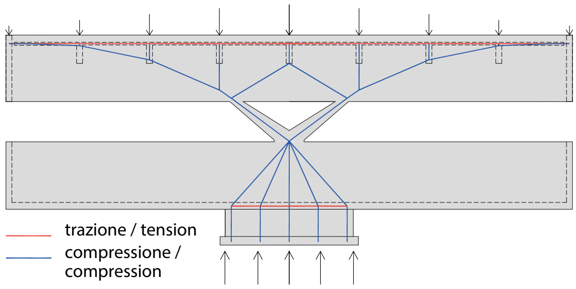

Fig. 6
Funzionamento facciata.
Force transfer in the façade.

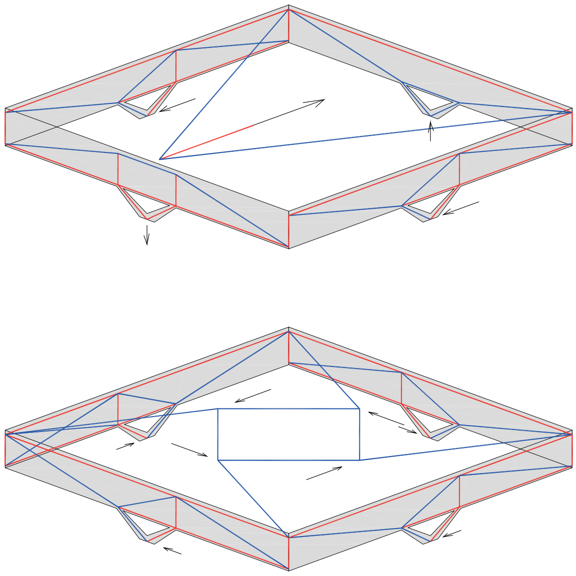

Fig. 7
Funzionamento stabilità.
Stability function.

struttura capace di resistere alle sollecitazioni orizzontali (vento e sisma). Considerando una forza incidente su una facciata, le pareti ortogonali allo sforzo impediscono la rotazione delle travi complanari allo stesso le quali trasmettono la forza a terra mediante una sollecitazione assiale delle forcelle. Una situazione analoga la si riscontra anche nel caso di una sollecitazione a torsione della struttura dove le singole facciate si sostengono reciprocamente. La sovrapposizione di questi campi tensionali determina la stabilità della struttura ad ogni genere di sollecitazione orizzontale.

Lo zoccolo

Da un punto di vista statico lo zoccolo si confronta con la ridistribuzione delle forze d'appoggio della copertura al terreno e la resistenza alle sollecitazioni idrostatiche risultanti dalla falda la cui quota massima si pone 1,80 m al di sopra della platea.
In un bilancio globale delle sollecitazioni verticali agenti ne risulta che una platea con spessori standard non è sufficiente a contrastare la spinta idrostatica agente e che il conseguimento dell'equilibrio mobilita necessariamente anche il carico risultante dalla copertura. Da questa riflessione ne consegue l'opportunità di una distribuzione del carico della copertura in platea il tutto a beneficio anche della tipologia della fondazione disposta in superficie sul terreno alluvionale piuttosto che su pali.
La distribuzione delle forze d'appoggio della copertura in platea è ottenuta mediante l'irrigidimento della stessa con due travi incrociate disposte sotto platea. Assieme alle pareti di bordo gli architravi disposti sotto platea suddividono la stessa in quattro porzioni quadrate di ca. 16,50 m di lato.
L'esecuzione di contrafforti verticali in continuità con la trave estendono lo stesso principio d'irrigidimento adottato per la pla-

At the centre of each wall beam there is a fork (V-shaped pillar) with a combined structure of steel tubes (ROR 273.25/S355) and concrete that can accommodate technical services and downspouts (drainpipes).
Individually, the wall beams are unstable but together they form a structure able to withstand horizontal loads (wind and earthquake). If one considers a force acting on one façade: the walls at right angles to the force prevent the rotation of its ground beams, which transmit the force to the ground through axial stressing of the V-shaped pillars. The same situation exists in the case of torsional behaviour on the structure where the single façades interact to provide mutual support. The overlap of these tension fields renders the structure stable under any kind of horizontal load.

The plinth

From a static point of view, the plinth handles the redistribution of the forces supporting the roof to the ground and the resistance against the hydrostatic pressures resulting from the groundwater table whose maximum level is 1.8 m above the floor level.

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Fig. 8
Facciata Sud (© Filippo Simonetti).
South Façade (© Filippo Simonetti).

In the global balance of vertical stresses, a floor of standard thickness is not sufficient to withstand the hydrostatic force and the load applied by the roof is also necessary to achieve equilibrium. This consideration marks the suitability of distributing the roof load to the floor, bringing only benefits to this type of foundation, which is placed on the surface of the flood plain rather than being supported on piles.
The distribution of the forces supporting the roof on the floor is achieved by stiffening it with two crossed beams under the floor. Along with the outer walls, the under-floor beams divide the floor into four square areas of approx. 16.50 m on each side.
The creation of vertical buttresses as a continuation of the beam extends the same stiffening principle adopted for the floor to the east and west walls of the building.

tea anche alle pareti est ed ovest dello stabile.

Aspetti costruttivi
La copertura
La messa in opera della copertura è stata vincolata dai particolari requisiti estetici delle travi parete eseguite in calcestruzzo a faccia a vista senza alcuna nicchia di tesatura dei cavi precompressi.
La soletta di copertura a cassettoni è stata gettata e precompressa prima delle facciate perimetrali. Questo modo di procedere ha condizionato la centina del graticcio dimensionata in modo tale da sostenere sia il peso distribuito del calcestruzzo durante la fase di getto che il carico concentrato risultante alle estremità delle travi dopo la tesatura dei cavi di precompressione.
A causa della capacità limitata dell'impianto di betonaggio del cantiere il getto della soletta a cassettoni di copertura (ca. 330 m^3)

Construction aspects
The roof
The installation of the roof was constrained by the particular aesthetic requirements of the wall beams, which are made of fair faced concrete without any tensioning niches for the prestressing cables.
The latticed roof slab was cast and prestressed before the perimeter façades. This procedure required dimensioning the rib of the gridwork in such a way as to support both the distributed weight of the concrete during the casting step and the concentrated load at the ends of the beams after the tensioning of the prestressing cables.
Due to the limited capacity of the concrete mixer at the construction site, the casting of the lattice roof (approx. 330 m^3) was carried out on two consecutive days: first the 14 beams of the gridwork, followed by the roof slab.

Fig. 9
Casseratura graticcio di copertura.
Roof lattice formwork.

Fig. 10
Struttura grezza dopo il disarmo.
Bare structure after removal of formwork.

è stato realizzato in due giorni consecutivi: dapprima le 14 travi del graticcio in seguito la soletta di copertura.
La tesatura dei cavi é avvenuta in due fasi: 30% a 3 giorni dal getto e i restanti 70% dopo 21 giorni.
Le travi pareti perimetrali sono state gettate in 4 tappe, ciascuna a forma di L, con interruzione di getto nella zona mediana della facciata.
I cavi di precompressione, disposti parallelamente e con teste fisse e mobili contrapposte agli angoli, sono stati tesati in tre fasi: 30% a 3 giorni dal getto, 70% dopo 14 giorni e 100% dopo ca. 28 giorni del getto dell'ultima porzione della trave.
Le forcelle sono state prefabbricate in cantiere e posate mediante autogru nei rispettivi punti di appoggio.

Lo zoccolo
La platea e le pareti perimetrali dello zoccolo dell'edificio sono concepite con il sistema «vasca bianca». Sia in platea che nelle elevazioni sono stati predisposti degli elementi di fessurazione controllata iniettati qualche mese dopo i getti in modo tale da garantire l'impermeabilità della costruzione.

The tensioning of the cables was carried out in two steps: 30% 3 days after casting and the remaining 70% after 21 days.
The perimeter wall beams were cast in 4 stages, each in an L shape, with casting interruptions in the middle area of the façade.
The prestressing cables, which are arranged in parallel with fixed and moveable heads counterpoised at the corners, were tensioned in three phases: 30% 3 days after casting, 70% after 14 days and 100% approx. 28 days after casting the last portion of the beam.
The V-shaped pillars were prefabricated at the construction site and positioned with a truck crane at the respective support points.

The plinth
The floor and the perimeter walls of the plinth of the building were conceived with the "white tank" waterproofing system. Both in the floor and in elevations, elements of controlled cracking were grouted several months after casting in order to guarantee that the underground structure remains waterproof.

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Abitazione sul lago di Sarnen
House on Lake Sarnen

Mario Monotti

Introduzione

La costruzione è un edificio di piccole dimensioni, ca. 100 m^2 abitabili, ubicato a Wilen sulle rive del lago di Sarnen nella Svizzera centrale. L'abitazione è concepita per approfittare in modo ottimale della bellezza del paesaggio visibile senza interruzioni dai quattro lati. Organizzati su diversi livelli, gli spazi interni offrono tre diversi sguardi del lago: quello orizzontale sull'acqua dal soggiorno (quota 0.00), quello a pelo d'acqua dalla camera (quota –1.00) e quello dall'alto dalla cucina (quota +1.00). Contro terra, sotto la cucina, con accesso dalla camera, trovano spazio un servizio e il locale tecnico. Ai locali interni si aggiunge un ampio spazio esterno coperto che si estende in modo asimmetrico sui quattro lati dell'abitazione. Oltre alla particolare disposizione interna, la costruzione ricava la propria valenza dalle tensioni risultanti tra la struttura portante in calcestruzzo faccia a vista con singolare ricchezza geometrica e le sgarberie architettoniche che, rompendo le simmetrie e variando le quote degli spazi interni, rendono ancor più arduo l'esercizio estremo della costruzione costretta a sorreggere con soli due pilastri il carico dell'intera copertura.
Di seguito sono esposte alcune riflessioni strutturali e gli aspetti costruttivi che hanno accompagnato la messa in opera dell'abitazione.

Riflessioni strutturali

L'organizzazione degli spazi interni, la copertura con tetto a falde e la tipologia delle colonne portanti sono state assunte come parametri inderogabili. La progettazione strutturale è stata focalizzata sulla disposizione dei pilastri e sugli aspetti costruttivi della copertura presupponendo l'impiego del calcestruzzo quale unico materiale da costruzione.
L'analisi delle colonne ha evidenziato come un'intersezione troppo elevata da terra dei puntoni obliqui e una disposizione asimmetrica dei pilastri allontanasse le traiettorie delle forze dalla struttura portante determinando delle sollecitazioni a flessione degli elementi portanti e, conseguentemente, un movimento rotatorio dell'intera copertura. Per migliorare il comportamento strutturale, i pilastri sono stati posti in asse con il tetto verso il lago e in corrispondenza del sostegno del tetto sopra la cucina verso monte. Nella configurazione scelta tre punti d'appoggio della copertura risultano fissi e l'origine del movimento della struttura è confinato in un solo elemento portante, il

Introduction

The structure is a small dwelling with around 100 m^2 of living space, located in Wilen on the shore of Lake Sarnen in Central Switzerland. The house was designed to take full advantage of the beautiful landscape, with uninterrupted views from all four sides of the structure. Organised on various levels, the internal spaces offer three different views of the lake: horizontally over the water from the living room (0.00 altitude), at surface level from the bedroom (–1.00 altitude) and looking down on it from above in the kitchen (+1.00 altitude). A bathroom and utility room are located underground beneath the kitchen, with access from the bedroom. In addition to the internal rooms there is a generous amount of covered

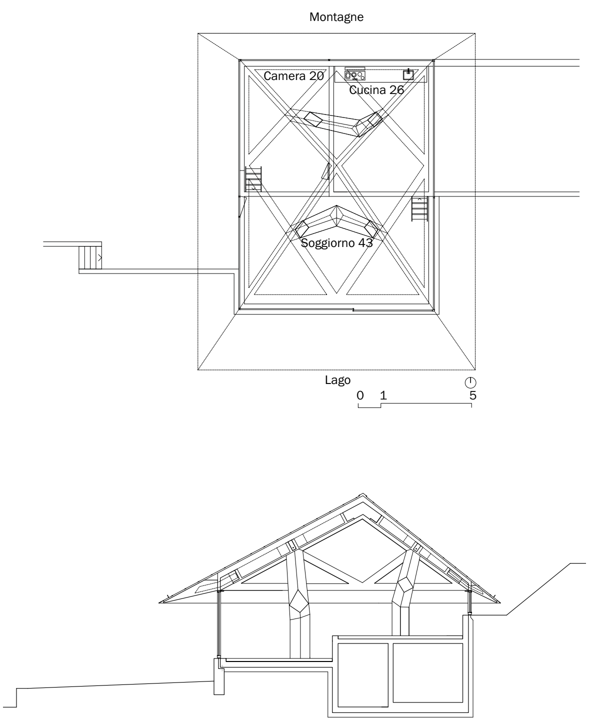

Fig. 1
Pianta e sezione sud/nord.
Plan and south/north section.

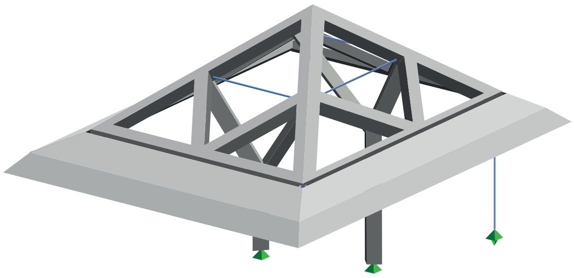

Fig. 2
Modello statico.
Static model.

Thoughts about the structure

The organisation of the internal spaces, the pitched roof and the type of load-bearing columns were taken as fundamental parameters. The structural design focused on the positioning of the pillars and the constructional aspects of the roof, presupposing that concrete would be the only construction material.
Analysis of the columns showed that an intersection of the diagonal struts too high above the ground and the asymmetrical location of the pillars would detach the flow of forces from the load-bearing structure, lead to bending stresses in the load-bearing elements and a rotation of the entire roof. To improve the structural behaviour, the pillars were placed in alignment with the roof facing the lake and in correspondence with the roof support over the kitchen facing the mountain. In the chosen configuration, three supports of the roof are fixed and the source of structural movement is confined to a single load-bearing element, the diagonal strut located above the bedroom. The rotation of the roof induced by the deformation of the pillar above the bedroom is

outdoor space, which extends asymmetrically on all four sides of the building. In addition to the unique internal design, the structure owes its appeal to the interplay resulting from the load-bearing fair-faced concrete structure, with a singular geometric richness and startling architecture, which by breaking up symmetries and varying the height of the internal spaces makes the construction work even more difficult, since the entire weight of the roof is supported by just two pillars.
Some information is given below on the structure and the design aspects involved in erecting the building.

puntone obliquo sovrastante la camera. La rotazione del tetto indotta dalla deformazione del pilastro al di sopra della camera è stata controllata mediante la precompressione del braccio inclinato del pilastro e una forza ausiliaria di bilanciamento indotta attivamente per mezzo di un tirante posto in cucina nell'angolo della facciata. L'efficacia dei provvedimenti intrapresi ha trovato ampio riscontro con la deformazione verticale misurata lungo la facciata al termine della costruzione.
Più che sugli aspetti statici, la progettazione della copertura si è focalizzata sulle problematiche costruttive determinate dal getto del calcestruzzo su superfici ob-

Fig. 3
Cassero e armatura pilastri.
Formwork and pillar reinforcement.

Fig. 4
Cassero e elementi prefabbricati copertura.
Formwork and prefabricated roofing elements.

lique con pendenze fino all'80%. In continuità con i pilastri, la struttura portante del tetto è stata ridotta a singole aste interconnesse rigidamente. La geometria di questi elementi è stata ricavata mediante pannelli prefabbricati di calcestruzzo armato di spessore minimo (6 cm) rifiniti in modo tale da fungere da cassero a perdere. Oltre a semplificare la messa in opera gli elementi prefabbricati hanno contribuito in modo sostanziale al contenimento delle imprecisioni esecutive permettendo al contempo di ridurre il peso proprio della struttura. Separata termicamente dal tetto, la gronda è stata gettata sul posto quale elemento conclusivo della costruzione.

Aspetti costruttivi

Le diverse fasi del cantiere sono descritte brevemente corredate dalle cifre essenziali che caratterizzano la costruzione.
L'edificio è stato inserito nel pendio naturale mediante uno scavo di 300 m^3 con altezza 5 m e sviluppo 15 m. Lo scavo è stato assicurato mediante calcestruzzo spruzzato e 13 ancoraggi passivi di lunghezza 8 m e resistenza 190 kN.

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controlled by pre-stressing of the inclined arm of the pillar and an auxiliary balancing force actively induced through a tie rod located in the kitchen in the corner of the façade. The effectiveness of the measures taken was clearly demonstrated through the vertical deformation measured along the façade when construction was completed.
More than on statical aspects, the design of the roof focused on construction problems caused by the need to cast concrete on slanting surfaces with slopes of up to 80%. In continuity with the pillars, the load-bearing structure of the roof was reduced to individual rigidly-connected rods. The shape of these elements was obtained using thin (6 cm) prefabricated reinforced concrete panels, finished so as to serve as lost formwork. In addition to simplifying the work, the prefabricated elements made a large contribution to limiting execution imprecisions, while simultaneously making it possible to reduce the weight of the structure. Thermally separate from the roof, the eaves were cast on site, as the final structural element.

Construction aspects

The various construction stages are described briefly below, together with the basic figures.
The building was inserted in the natural slope with a 300 m^3 excavation, height of 5 m and length of 15 m. The excavation was reinforced using cast concrete and 13 passive anchors with a length of

L'edificio è stato fondato su 16 pali battuti in ghisa duttile di lunghezza compresa tra 10 e 15 m e resistenza di 800 kN. La scelta di una fondazione profonda è stata determinata dalle caratteristiche del terreno, dalla particolare vicinanza al lago e dalla concentrazione dei carichi.
Lo zoccolo dell'edificio (locale tecnico e camera) è stato concepito con il sistema «vasca bianca» (iniezione dei giunti di lavoro) in calcestruzzo impermeabile faccia a vista in modo tale da resistere alla spinta del terreno e alle sollecitazioni idrostatiche relative a un' immersione di 1,75 m.
I pilastri presentano una sezione a quadrilatero concavo invariata (di 0,29 m^2 lato lago e 0,25 m^2 lato monte) sul primo tratto e rastremata nei bracci (da 0,27 a 0,20 m^2 lato lago rispettivamente da 0,23 a 0,16m^2 e da 0,25 a 0,18 m^2 lato monte). Il cassero dei pilastri è stato concepito con due lati fissi e due mobili in modo tale da favorire la posa dell'armatura disegnata in ogni singola posizione. Entrambi i pilastri sono stati messi in opera in una sola tappa mediante pompaggio di calcestruzzo tipo SCC 30/37 dal piede. Il ramo più obliquo del pilastro lato monte è stato precompresso con 2 cavi di 4 trefoli cadauno aventi testa fissa al piede del pilastro e mobile sul lato esterno della copertura. Quale tirante di bilanciamento è stata impiegata una barra Stafix M27.
Le travi portanti della copertura, ottenute mediante la posa di elementi prefabbricati su un telaio di

Fig. 5
Vista sud (© Karin Gauch, Fabien Schwartz).
South view (© Karin Gauch, Fabien Schwartz).

Fig. 6
Vista nord (© Karin Gauch, Fabien Schwartz).
North view (© Karin Gauch, Fabien Schwartz).

8 m and a capacity of 190 kN. The building has a foundation of 16 ductile cast iron piles, with lengths ranging from 10 to 15 m and a load capacity of 800 kN. The decision to opt for a deep foundation was determined by the characteristics of the ground, in particular the nearness to the lake and the concentration of the loads.
The basement (utility room and bedroom) was designed with a "white tank" system (injection of working joints) in waterproof fair-faced concrete, so as to resist the earth pressures and the hydrostatic pressures associated with a depth below the water level of 1.75 m.
The pillars have an unvarying concave quadrilateral section (0.29 m^2 lake side, 0.25 m^2 mountain side) in the first part, which tapers in the arms (from 0.27 to 0.20 m^2 lake side, from 0.23 to 0.16 m^2 and from 0.25 to 0.18 m^2 mountain side, respectively). The formwork for the pillars was designed with two fixed sides and two moveable sides, so as to facilitate the casting of the reinforcement in each individual position. Both pillars were rendered operational in a single casting, by pumping SCC 30/37 type concrete from the base. The most oblique part of the mountain-side pillar was prestressed with 2 cables of 4 strands each, with the head attached to

legno, sono state gettate in opera in un'unica tappa con chiusura progressiva del cassero sul lato superiore.
La gronda è stata fissata al telaio della copertura mediante 12 profili in acciaio con peso complessivo di 2,1 t parzialmente rivestiti con materiale isolante. Analogamente ai pilastri, la gronda è stata messa in opera in una sola tappa con calcestruzzo tipo SCC 30/37 pompato all'interno di un cassero ermetico.
Complessivamente, per la costruzione sono stati impiegati 135 m^3 di calcestruzzo (55% nello zoccolo, 2% nei pilastri, 24% nella copertura e 19% nella gronda) e 28 t d'armatura (40% nello zoccolo, 5% nei pilastri, 36 % nella copertura e 19% nella gronda).

Conclusioni

Malgrado le modeste dimensioni, l'edificazione dell'abitazione sul lago di Sarnen ha richiesto ogni genere di lavoro specialistico del genio civile. La riuscita dell'opera è stata conseguita grazie ad un grande impegno e un rapporto di amicizia tra tutti gli addetti ai lavori.

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the base of the pillar and moveable on the external side of the roof. A Stafix M27 bar was used as a balancing tie rod.
The load-bearing beams of the roof, obtained by placing prefabricated elements on a timber frame, were cast in a single operation with progressive closing of the formwork on the upper side.
The eaves were attached to the roof frame using 12 steel sections with a total weight of 2.1 t, partially covered with insulating material. Similarly to the pillars, the eaves were cast in a single operation using SCC 30/37 concrete, pumped inside a hermetically-sealed formwork.
In total, 135 m^3 of concrete was used in construction (55% for the plinth, 2% for the pillars, 24% for the roof and 19% for the eaves), as well as 28 t of reinforcement (40% for the basement, 5% for the pillars, 36% for the roof and 19% for the eaves).

Conclusions

Despite its modest size, the construction of the house on Sarnen Lake required the use of every type of specialist civil engineering work. The success of the work was achieved thanks to notable efforts and a friendly working relationship between all involved.

Elefantenpark «Kaeng Krachang» im Zoo Zürich – Betontragwerk

Elephant House "Kaeng Krachang" in the Zurich Zoo – Concrete structure

Fabian Persch

Einleitung

Im Frühjahr 2014 wird der Elefantenpark «Kaeng Krachang» als neue Attraktion des Zürcher Zoos für das Publikum geöffnet. Kernstück des 10 000 m^2 grossen Geheges ist eine kreisförmige Halle, die von einer spektakulären Holzdachkonstruktion überspannt wird. Das riesige Dach mit einer Fläche von 6000 m^2 und einem Durchmesser von 85 m wirkt dabei als freitragendes Schalentragwerk, dessen Lasten über einen Stahlbetonringbalken in lokale Dachfundationen abgeleitet werden. Die Geometrie des Dachs wurde mithilfe eines FE-Schalenmodells in einem statischen Formfindungsprozess schrittweise entwickelt. Mit seiner zufällig wirkenden geschwungenen Form soll es sich einerseits harmonisch in die Landschaft einfügen, gleichzeitig ist der gekrümmte Rand aber auch dem Nutzungsprogramm des Gebäudes angepasst. Die Bogenhöhen zwischen den Auflagern ergeben sich aus architektonischen Vorgaben, wie den vorgesehen Besuchereinblicken, den Höhen von Tordurchfahrten und der Grösse der Elefanten.

Als besonders komplex stellt sich der 270 m lange Ringbalken dar, der der unregelmässigen Form des Schalenrands folgt. Obwohl er Spannweiten von bis zu 40m zwischen den einzelnen Auflagerpunkten zu überbrücken hat und dabei die Durchbiegungen für die Fassadenplanung in einem engen Rahmen begrenzt bleiben müssen, soll er als filigranes, kaum sichtbares Bauteil im Dachrand integriert werden. Dieses Ziel lässt sich nur durch den Einbau von Vorspannkabeln realisieren. Die tatsächlich auftretenden Verformungen lassen sich bei diesem Prototypen im Vorfeld nicht exakt definieren, sondern nur über Sensitivitätsanalysen eingrenzen. Nicht nur planerisch, auch in der

Introduction

In spring 2014 the elephant park "Kaeng Krachang" in the Zurich Zoo will be opened as a new attraction to the public. Centrepiece of the 10,000 m^2 large compound is a roundish concrete hall roofed by an impressive curved wooden structure. The giant roof with an area of 6,000 m^2 and 85 m in diameter acts as a self-supporting shell-structure, whose loads are transferred via a reinforced concrete ring beam to the local roof foundations. The roof's geometry was incrementally developed with the aid of an FE shell model. With an irregular appearance and a flowing shape its aim is to harmoniously fit into the surrounding landscape. At the same time, the curved edge is geared to the building's utilisation concept. The rises of the arches between the supports are determined by architectural requirements, such as the planned visitors' views,

Fig. 1
Künstlerischer Entwurf (Quelle: Markus Schietsch Architekten).
Artist's impression (source: Markus Schietsch Architekten).

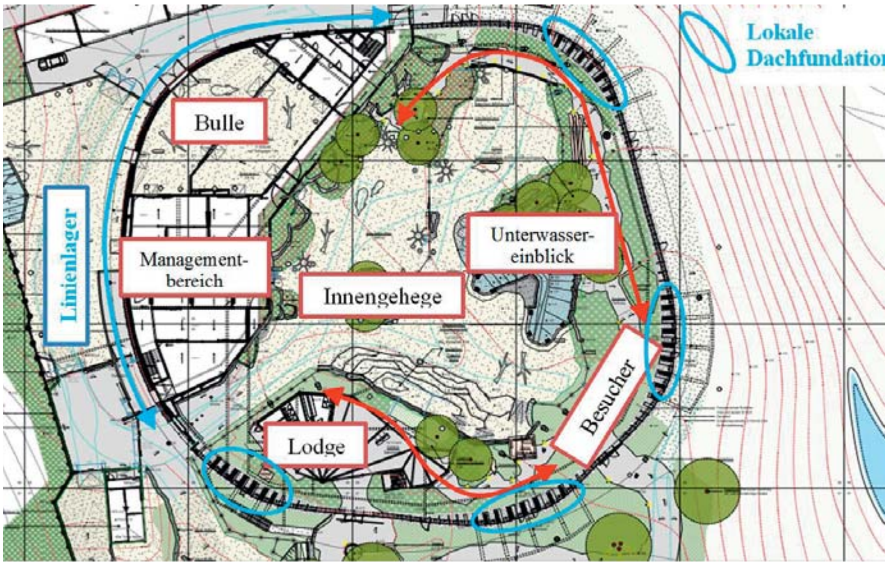

Fig. 2
Nutzungskonzept Elefantenhaus (Quelle: Walt+Galmarini).
Utilisation concept for Elefantenhaus (source: Walt+Galmarini).

clearance heights and size of elephants.

A particularly complex component of the structure is the 270 m long ring beam that follows the uneven form of the roof edge. While bridging spans of up to 40 m with only very small deflections permitted, the beam is meant to be thin and invisibly integrated in the roof. These requirements can only be met by the use of post-tensioning. Predictions of actual deflections of this prototype are difficult and can only be limited by means of sensitivity analyses.

Ausführung stellt das Betontragwerk eine grosse Herausforderung dar. So müssen grosse Teile der Bewehrung und die Vorspannkabel aufgrund der dreidimensionalen Geometrie des Bauwerks mithilfe von 3-D-Plandarstellungen mit Koordinatenangabe millimetergenau eingemessen werden.

Betontragwerk

Die Anordnung des Tragwerks ergibt sich aus den architektonischen Vorgaben für die Nutzung der Halle. Ziel bei der Tragwerks-

Not only with regard to the design but also with the execution, the concrete structure presents a major challenge. Due to the 3D geometry of the structure, large sections of the reinforcement and the post-tensioning tendons have to be measured to millimetre accuracy with the aid of 3D plans with detailed information of the coordinates.

Concrete structure

The layout of the structure is given by the architectural requirements for the utilisation of the

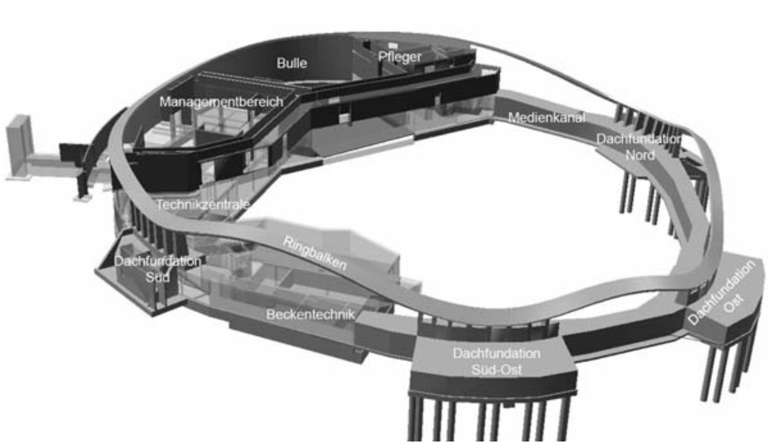

Fig. 3
Betontragwerk (Quelle: Walt+Galmarini).
Concrete structure (source: Walt+Galmarini).

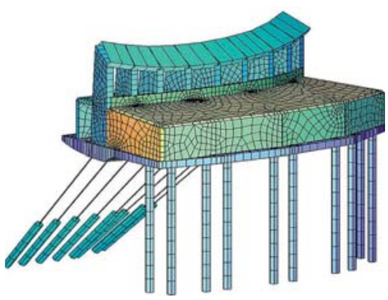

Fig. 4
Modell Dachfundation (Quelle: Walt+Galmarini).
Model roof foundation (source: Walt+Galmarini).

Fig. 5
Bewehrung einer Dachfundation mit Spannkabeln für die Wandscheiben.
Reinforcement of a roof foundation with bracing cables for the shear walls.

Fig. 6
Holzdachkonstruktion mit Wandscheiben.
Timber roof construction with shear walls.

planung war es, alle ohnehin benötigten Betonbauteile zur Abtragung der Dachlasten hinzuzuziehen und so auf zusätzliche Elemente möglichst zu verzichten. So dient beispielsweise die Aussenwand der Stallungen als lineares Auflager für den Ringbalken und die vorgesehenen Regenwasserspeicher sind als lokale Dachfundationen ausgebildet und an den unterirdisch verlaufenden Medienkanal angebunden.

Lokale Dachfundationen mit vorgespannten Wandscheiben

Die Schalenkräfte werden vom Ringbalken über vorgespannte Wandscheiben in die lokalen Dachfundationen abgetragen. Die horizontale Komponente wird dabei durch permanente Felsanker, die vertikale Komponente über Pfähle in den Untergrund abgeleitet.
Von zentraler Bedeutung für die Formstabilität des Dachs ist, dass die Wandscheiben nur geringe horizontale Verschiebungen erfahren. Jede hier auftretende Verformung hat ein Abflachen der Schale zur Folge. Daher ist jede Wandscheibe für Beanspruchungen auf Gebrauchslastniveau mit einem vertikalen Spannglied vorgespannt. Die festen Verankerungen der Drahtkabel vom Typ

building. The aim of the design was to integrate all existing concrete members of the structure for the transfer of the roof load, and thus avoid additional elements as much as possible. Thus, the exterior wall of the stables, for instance, serves as a linear bearing for the ring beam, and the planned rainwater tanks are designed as local roof foundations and connected to the underground services duct.

Local roof foundations with prestressed shear walls

The ring beam transfers the roof loads to the roof foundations, which are anchored by piles to resist the vertical load component, and by permanent rock anchors for the horizontal load component.
It is essential for the roof's stability that the shear walls only permit very small horizontal movements. Any deflection of the walls results in a flattening of the roof shell. For this reason, each shear wall is prestressed with a vertical post-tensioning tendon to the level of the live load. The fixed-end anchorages of the wire cables BBRV 3700 together with the cages with threaded reinforcement are measured to millimetre

BBRV 3700 sind jeweils mit Körben aus schraubbarer Bewehrung millimetergenau relativ zur Bodenplatte der Fundation eingemessen. Das Betonieren der Wandscheiben erfolgt in mehreren vertikalen Etappen. Die Wandköpfe werden zusammen mit dem Ringbalken betoniert.

Ringbalken

Um die planmässigen Beanspruchungen aufnehmen zu können sowie im aussergewöhnlichen Bemessungszustand – wie beispielsweise beim Ausfall von Felsan-

Projektdaten/Project data

Bauherr/Owner
Zoo Zürich AG
Gesamtleitung/General direction
cga Consulting Group Aeberhard, Dättlikon
Architekt/Architect
Fischer Architekten AG, Zürich
Landschaftsarchitekt/ Landscape architect
vetschpartner Landschaftsarchitekten AG, Zürich
Bauingenieur/Civil engineer
Walt+Galmarini AG, Zürich, BlessHess AG, Luzern
Ausführung/Contractors
Landolt & Co. AG, Kleinandelfingen

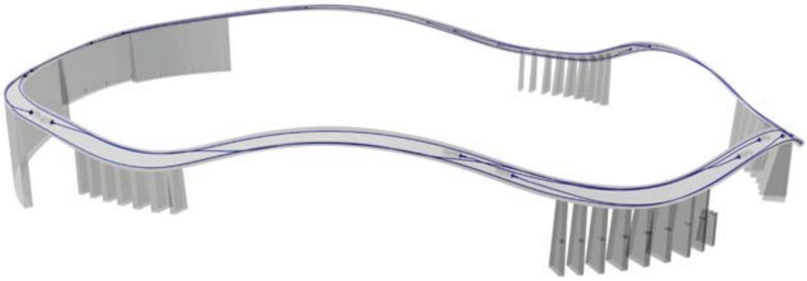

Fig. 7
Spanngliedverlauf im Ringbalken (Quelle: Walt+Galmarini).
Geometry of curved tendon in the ring beam (source: Walt+Galmarini).

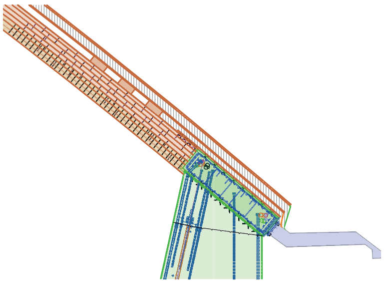

Fig. 8
Regelschnitt Ringbalken.
Standard section of ring beam.

accuracy relative to the bottom slab of the foundation. The concreting of the shear walls is carried out in several vertical stages. The tops of the walls are cast at the same time as the ring beam.

Ring beam

To withstand the design loads as well as to act as a tension flange

kern und damit der horizontalen Stützung eines Widerlagerbereichs – wie ein Zuggurt wirken zu können, ist der Ringbalken mit insgesamt neun Drahtspanngliedern vom Typ BBRV 1900 mit Längen von 70 bis 120 m und einer Vorspannkraft von jeweils 190 Tonnen vorgespannt. Die Spanngliedführung ist dabei drei-

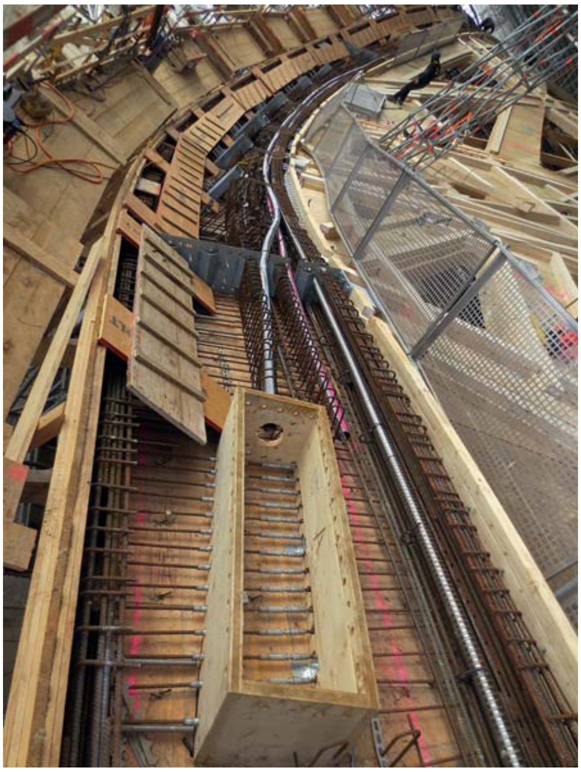

Fig. 9
Verlegte Hüllrohre und Spannnische.
Placed sheath and prestressing niche.

in accidental design situations – such as the failure of rock-anchors and hence the horizontal support of an abutment zone – the ring beam is prestressed with nine type BBRV 1900 post-tensioning tendons with lengths of up to 120 m and a prestressing force of 190 t per tendon. The tendon layout is three-dimensional, i.e. the cables are arranged in such a way that, vertically, the ring beam lifts the roof in the spans, and, horizontally, is tied to the wall abutments. Two tendons are always arranged in parallel to avoid twisting of the ring beam. The position of the tendons and the prestressing niches had to be specified and measured on to millimetre accuracy. Since the tendons run over several concreting stages, the empty ducts were installed prior to concreting and the wire bundles were eventually pulled in using a winch.
Reinforcement and post-tensioning tendons were installed in a "wooden channel" that was subsequently closed with the shuttering to a "wooden pipe" and filled with self-compacting concrete. The lowest layer of the multi-layer wooden roof served as the soffit formwork and together with the upper formwork and the formwork sides was also left in the structure.
The casting of the ring was carried out in 15 stages with a weight of up to 50 t of fresh con-

Fig. 10
Einziehen der Drahtbündel in vorverlegte Hüllrohre.
Pulling in the bundle of wires in the previously placed sheath.

Fig. 11
Befestigen der Konterschalung.
Fixing the upper formwork.

dimensional, d.h. die Kabel sind derart angeordnet, dass sie in vertikaler Richtung eine Dachanhebung im Feldbereich und in horizontaler Richtung ein Zusammenspannen mit den Wandscheiben bewerkstelligen. Dabei sind jeweils zwei Kabel parallel angeordnet, um eine Verdrillung des Ringbalkens zu verhindern. Die Position der Spannkabel als auch der Verankerungsnischen musste millimetergenau bestimmt und eingemessen werden. Da die Vorspannkabel über mehrere Betonierabschnitte verlaufen, wurden jeweils zuerst die leeren Hüllrohre verlegt und anschliessend die Drahtbündel mit einer Winde eingezogen.
Die Bewehrung und Vorspannung wurde in eine «Holzrinne» eingebaut, anschliessend mit einer Konterschalung zu einer «Holzröhre» geschlossen und mit selbstverdichtendem Beton (SCC) ausgefüllt. Als Bodenschalung diente dabei die unterste Lage des Mehrschichtholzdachs, und auch die Konterschalung an der Oberseite und die Randabschalungen verblieben anschliessend im Bauwerk. Das Betonieren des Rings erfolgte in 15 Etappen mit bis zu 50 Tonnen Frischbetongewicht. Aufgrund der Rezeptur erreichte der verwendete Beton vom Typ Holcim Selfpact 3716CL trotz der tiefen Aussentemperaturen bereits nach drei Tagen die zum Vorspannen benötigte Festigkeit.

crete. Due to its formula, the installed concrete Holcim Selfpact 3716CL, despite of the low temperatures, was able to achieve the necessary strength for stressing after only three days. After stressing the tendons, which was carried out in several steps, the roof that so far had been supported by falsework, was lowered, and the loads were transferred to the ring beam and the roof foundations, respectively. The deflections of the structure are monitored with fibre-glass sensors such that conclusions can be drawn as to the actual forces or moments and comparisons with the results of the calculation models can be made.

Nach dem mehrstufigen Anspannen der Kabel wurde das Dach, das bis dahin von einem Lehrgerüst gestützt wurde, abgesenkt und die Belastung auf den Ringbalken beziehungsweise die Dachfundationen übertragen. Mithilfe von Fiberglassensoren werden die Dehnungen überwacht und so Rückschlüsse auf die tatsächlich vorhandenen Kräfte und Momente gezogen und mit den Berechnungsmodellen verglichen.

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