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# SUSTAINABLE RESTORATION OF THE ROOF OF A LARGE INDOOR POOL

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**Abstract:** During an inspection of the roof of an indoor pool from 1971 in Potsdam (Germany) it was observed, that the roof and the column structures of the building are partially damaged by corrosion. The roof construction is a suspended simple-bent pre-stressed shell between two stiff edge beams. The span of the roof is 39.5 m. The edge beams are supported by A-frames. After further investigation on the damaged areas (including tension tests, tension tests on notched tendons with different load cycles, tests of the bearing capacity of sleeve splices and screw nuts) the conclusion was that the joint of the tension rod and the concrete column had to be restored. This was accomplished through the attaching of a new supporting construction to the existing parts, which took over the load from the old connection. At both sides of the columns pre-stressed DYWIDAG-single-tendons (St1080/1230, diameter of 36 mm) were used. They have to bear a maximal tension force of 835 kN. With this unusual restoration technique, a fast and relative inexpensive retrofitting of the construction could be carried out. The existing construction had to be modified only marginal, and the typical appearance of the building was preserved. This solution stands for sustainability.

*Keywords*: .

## 1. History

In 1969 began the construction of an indoor swimming pool in Potsdam, Germany (Fig. 1). It was built simultaneously with the buildings in Rostock, Leipzig and Dresden. Similar objects were built shortly after in Erfurt and Halle. All of these indoor pools have a, for their time modern, suspended roof construction, which turned out to be a new territory for the constructing companies. The results of this are the till this day emerging constructional defects. During the building the constructors tried to build an exceedingly compact roof, which was necessary for the suspended roof construction.

Fig. 1. The indoor swimming pool.

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The indoor pool was completed in 1971. Already at the end of the 80´s the roof construction turned out to be leaky. That is why first restorations and modernisations were done at the beginning of the 90´s. In 2006 the indoor pool was temporarily closed because of heavy damages on the construction.

The paper will describe an unusual restoration technique. A fast and inexpensive retroffiting of the roof will be presented, which does allow to extend the life cycle of the indoor pool for at least 6 years.

## 2. The Structure

The building consists of two parts. The hall part with a 50 m long swimming pool and the outbuilding with the non-swimmers pool, the technical facilities, the sanitary rooms and a sauna. Over the hall area spans the suspended roof construction. The outbuilding is a traditional reinforced concrete construction.

The roof construction over the hall area is a suspended simple-bent prestressed shell between two stiff edge beams (Fig. 2). The span width of the roof is 39.5 m. The edge beams are supported by A-frames. The walls are entirely separated from the roof. Between the edge beams cables are hang up. The roof cladding is made from prefabricated concrete panels which are connected with suspension hooks to the cables and have small steel pins at the cables to provide a shear-proof connection between each other. The longitudinal groove is made of C25/30 concrete. In the groove are 4 longitudinal reinforcing rods (2 ø10 + 2 ø12) with a yield stress of 400 N/mm$^2$ and every 20 cm flat ø6 binder with a yield stress of 220 N/mm$^2$. The concrete panels are 4.5 cm thick. The corrosion protection of the steel parts in the roof construction should be ensured by the prescription of a minimal concrete coverage and a coating with epoxide resin containing paint.

The cables are steel rods with the quality class St 60/90 and a diameter of 26 mm. The length of the cables is varies between 36.88 m and 40.56 m, depending on the place of installation. The maximal length of the rods just 22.0 m is, so the cables had to be extended by sleeve splices. The preset sag of the cables in the middle point is 3.166 m in the completed construction. The cables are pretensioned with an initial load. The value of this initial load was set so, that in the completed roof are even in case of a total snow load are just compressive stresses. This way the danger of a rip of the grooves is banned. The horizontal component of the cable-force in the completed roof is 128 kN.

The edge beams are 6.0 m long prefabricated single-span reinforced concrete beams. 8 cables are connected to each girder. The beams are made of C25/30 concrete and reinforcement with a yield stress of 400 N/mm$^2$.

The columns (Fig. 3) on which the edge beams bear on are prefabricated A-frames. They are 9.22 m high and below the height of 2.69 m divided into a tension rod from steel and a compressed rod form concrete, with a span of 5.98 m at their foot. The head part and the concrete bar are rectangular concrete sections which are restricted to their foot. The tension rod is a steel eye bar with

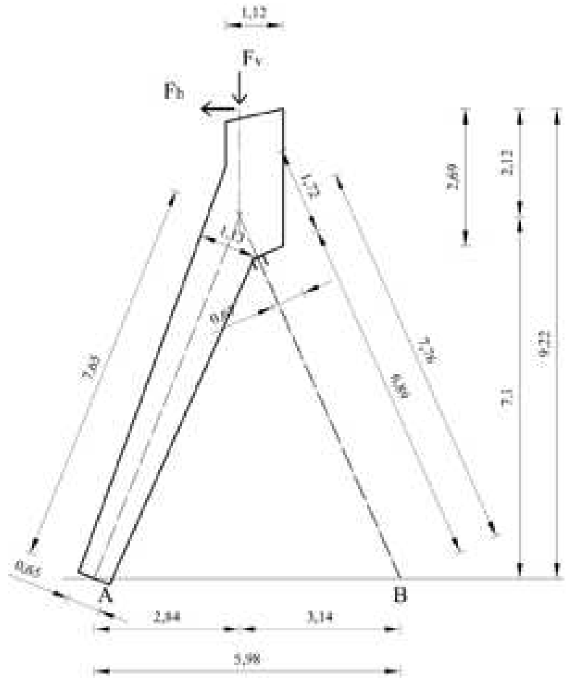


Fig. 3. The geometry of the columns.

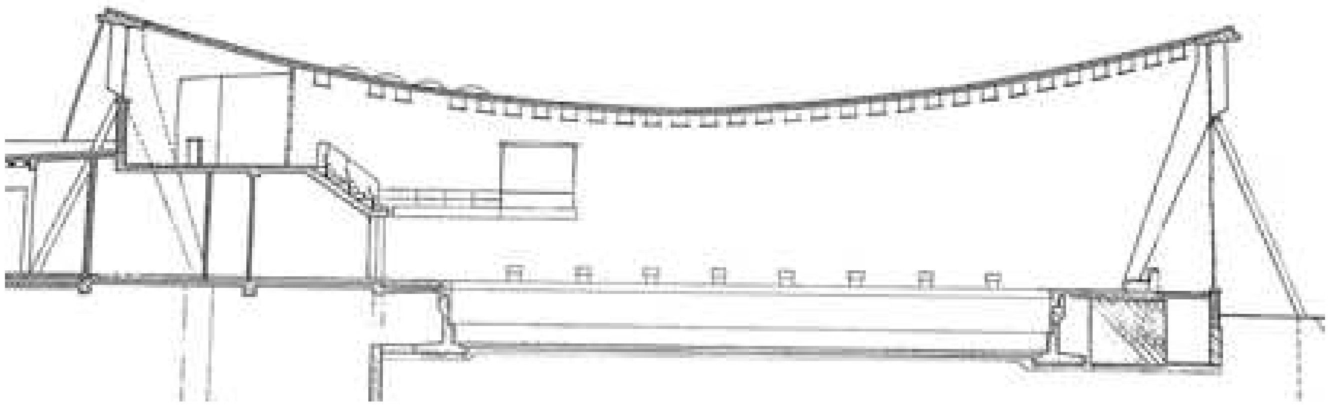

Fig. 2. The section of the roof structure of the building.

a 200×150 mm cross section in the middle of the bar and 200×355 mm cross sections with 122 mm diameter pinholes on its ends. The head part has a length of 2.177 m, the compressed bar a length of 7.65 m and the tension rod a length of 7.76 m.

The horizontal loads of the roof construction are transmissioned to the columns at the column head. Together with the forces that are acting directly on the columns, the following loads result for them:

– Maximal tensile force of 1630 kN,
– Maximal compressive force in the compressed bar of 3432 kN with a simultaneous moment of 819 kNm,
– Maximal moment in the compressed bar of 3177 kNm with a simultaneous compressive force of 3334 kN,
– Maximal moment in the top part of 4021 kNm with a simultaneous compressive force of 3275 kN.

The head-section and the compressed bar are fabricated as one member. The connection of the tension rod to the concrete member is solved trough a hinge (d = 120 mm) between the eye bar and an anchor plate. The anchor plate is fixed with 8 ø26 stay rods to the concrete member.

The foundations are equipped with a tilting plate at their top to allow a potential rotation of the column. The acting transversal forces are transmissioned by tholepins. For more details see Franke & Heinze (1969), Quade (1969) and Acker & Heinze (1969).

## 3. The damages

During an inspection of the indoor pool in February 2006 was observed, that at the connection between the tension rod and the concrete column heavy corrosion damage occurred on the screwings (Fig. 4). The damages were especially heavy on the northern facade of the building. Here were the joints, which lie outside the façade are covered by a sheet-metal lagging. Behind this lagging moisture could congregate, which was leading to increased corrosion. The bad accessibility of this part was a reason why the damage could only be discovered at a close inspection. Besides this it was assessed that the roof is leaky which meant that water could get to the tendons of the roof construction. The indoor pool had to be closed temporary in the spring, until the faults of the structure would be resolved, since the structural safety of the construction was not ensured anymore.

The primary objective by the assessment of the bearing capacity of the damaged structure had to be the investigation of the condition of the tendons and the sleeves, especially the screwings, because these were the critical members of the connections.

Fig. 4. The corroded joint of the tension rod.

## 4. Material tests

The steel of the tendons which was used for the tests was retained from the structure of the indoor pool.

The investigation of the bearing capacity could be divided into following four tests, made by the Lab (FMPA) of the BTU:

1. Tension test on specimen according to DIN 50125
2. Tension tests on notched bars with 1 000 und 10 000 load cycles
3. Tests of the bearing capacity of the sleeve splices
4. Tests of the bearing capacity of the screw nuts

All tests of the bearing capacity were carried out with a testing machine of the type MTS (peak load 500 kN, accuracy class 1). The tests 1, 3 and 4 were run force controlled with a test speed of v = 3150 N/s (calculated according to the parameters of the DIN 10002).

After the tension tests the stress-strain curves were calculated on the basis of the load-displacement diagrams. The values of the tensile strength as well as the ultimate strain and the contraction at failure were determined through calculations. The determination of the yield point $R_{p02}$ was made graphically based on the stress-strain curve with an appropriate big scale. The values of the yield strength $R_e$ can not be specified, because the material shows a discontinuous transition from the elastic to the plastic region.

In Table 1 it can be seen, that the measured yield strength of the material St 60/90 lies between 556 and 566 N/mm$^2$, what is under the nominal yield strength of this steel of 600 N/mm$^2$.

Tab. 1. The results of the tension tests.

| | Specimen Z-1 | Specimen Z-2 | Specimen Z-3 |
|---|---|---|---|
| $d_0$ [mm] | 19.9 | 19.9 | 19.9 |
| $d_u$ [mm] | 16.1 | 16.3 | 16.6 |
| $S_0$ [mm$^2$] | 311.0 | 311.0 | 311.0 |
| $S_u$ [mm$^2$] | 203.6 | 208.7 | 216.4 |
| $L_0$ [mm] | 99.5 | 99.5 | 99.5 |
| $L_u$ [mm] | 111 | 112 | 110 |
| $R_m$ [N/mm$^2$] | 1063 | 1064 | 1082 |
| $R_{p02}$ [N/mm$^2$] | 566 | 558 | 556 |
| A [%] | 11.6 | 12.6 | 10.6 |
| Z [%] | 34.5 | 32.9 | 30.4 |

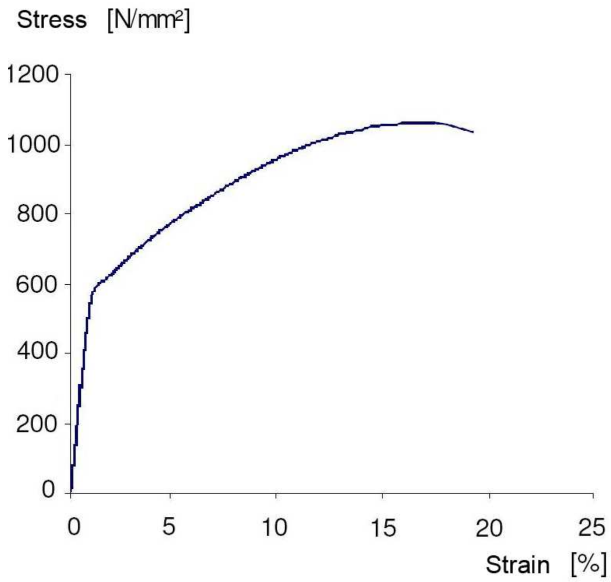


Fig. 5. The stress-strain curve of the specimen Z-1.

Within the tests on the notched bars round specimen were tested with a rising tensile load with 1 000 and 10 000 load cycles. This was made to investigate the behaviour of the tendons in case of a sectional weakening, caused by corrosion or potential notches. These round specimen are made from the material of the roof construction. At the fabrication the original diameter of the tendons was kept and a 2 mm deep revolving notch with an angel of 60° between the sides was milled. In the notch base is according to the geometry of the machine a radius of ca. 0.1 mm (Fig. 6). The tests were carried out with 1 000 and with 10 000 load cycles. The load was applied with a sinusoidal cycle with a frequency of 1 Hz. The test duration was according to that 17 minutes at 1 000 cycles and 170 minutes at 10 000 cycles. The summary of the results of the tests are shown in Table 2.

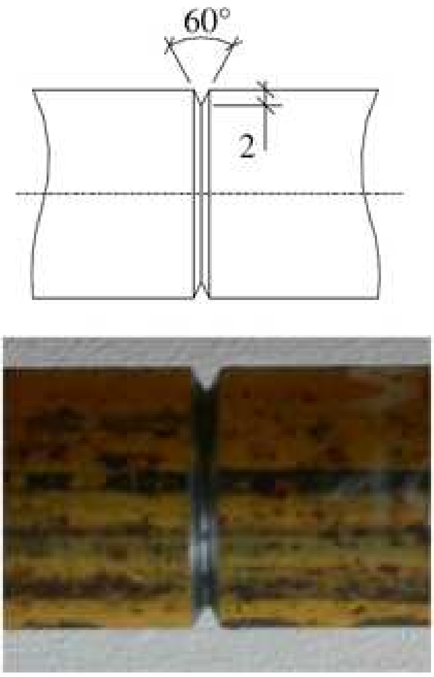


Fig. 6. The rod with the circumferential groove.

Tab. 2. Parameters and results of the tests.

| Notched bar | Load cycle N | Lowest load $F_{min}$ [kN] | Highest load $F_{max}$ [kN] | Observation |
|---|---|---|---|---|
| KSt1000-1 | 1 000 | 86 | 150 | Test succesful |
| KSt10000-2 | 10 000 | 86 | 150 | Test succesful |
| KSt1000-3 | 1 000 | 86 | 150 | Test succesful |
| KSt10000-4 | 10 000 | 86 | 150 | Test succesful |
| KSt.* | 1 000 | 86 | 200 | Test succesful |

For the bearing capacity tests of the sleeve splices, four sleeves from S235J2G3 were fabricated according to the plans at hand. The length of load transmission is 20 mm on both sides of the sleeves (Fig. 7). Table 3 shows the relevant loads and the causes of failure for the particular sleeve splices.

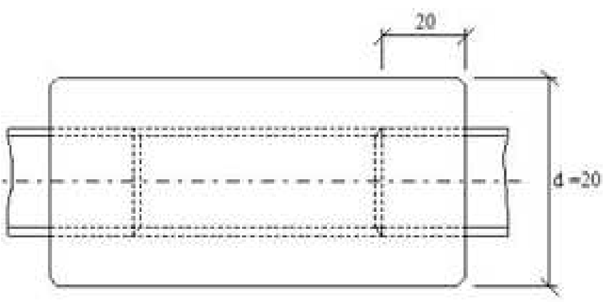


Fig. 7. The sleeve splice.

For the bearing capacity tests of the screw nuts, a device was build according to Figure 8 for a tightening test. The diameter of the hole is d = 30 mm, the nut was shortened to 28 mm. The peak load for the failure here was $F_{max}$ = 461 kN. The limit of elasticity lied at $F_e$ = 171 kN. The displacement contains the unavoidable deformation of the load transmission device. The cause of the failure was in the majority of cases the fracture of the tendon at the end of the thread.

Fig. 8. Experimental set-up for the load capacity test of the screw nut.

Tab. 3. Summary of the tests of the bearing capacity of the sleeve splices.

| Sleeve slice | Load at ekasticity limit $F_e$ [kN] | Maximum failure load $F_{max}$ [kN] | Failure reason |
|---|---|---|---|
| MS-1 | 190.4 | 390.2 | Failure of prestressed steel on thread run-out |
| MS-2 | 159.3 | 414.9 | Failure of socket screw thread |
| MS-3 | 161.7 | 390.4 | Failure of prestressed steel on thread run-out |
| MS-4 | 163.0 | 374.3 | Failure of prestressed steel on thread run-out |

After the analysis of the test results and the comparison of the load bearing capacity it was obvious that the roof construction had a sufficient bearing capacity, but the connection of the tendons in the columns did not.

## 5. Restoration

The requirement for the restoration concept was, that the structural safety of the building is not affected by the actions. After the assay of the building it was decided to restore the columns whose tension members are outside the facade. The penetration point of the tension members through the concrete column, the points of force transmission (inside) and the joints of the tension members had to be retrofitted.

The strengthening was carried out through the installation of additional parts. A new construction would bypass and unload the original connection. This way there was no critical intermediate state like in case of a replacement. (Fig. 9-11).

Fig. 9. The lower joint of the by-pass construction.

Fig. 10. The upper joint of the by-pass construction.

Fig. 11. The restored column line.

The area of the anchor plates on the concrete columns were laid open, in doing so the grouting was removed carefully. The cheeks on the concrete columns were cut off. The overhang of the existing threaded rods was cut off to ca. 1 cm over the counternut. The anchor plates were derusted and cleaned. 350 mm high U-profiles were laid pairwise on the created supporting areas and their bedding were sealed. The forces from the tendons were transferred by a 80 mm thick steel plate to the 350 mm high U-profiles. On each U-profile 3×2 stiffeners were applied because of the high local loads. The profiles bear directly on the concrete construction. (Fig. 10 and 12).

At both sides of the columns in each case a single tendon was placed between the U-profiles. They were guided directly beside the columns through the facade. The tendons are DYWIDAG-single-tendons without bracing, with quality class St1080/1230 and a diameter of 36 mm (DIBt 2006). They have to bear a maximal tension force of 835 kN. The tendons were to prestress with 50 % of the calculated service loading. The pretension was alternating (Fig. 13).

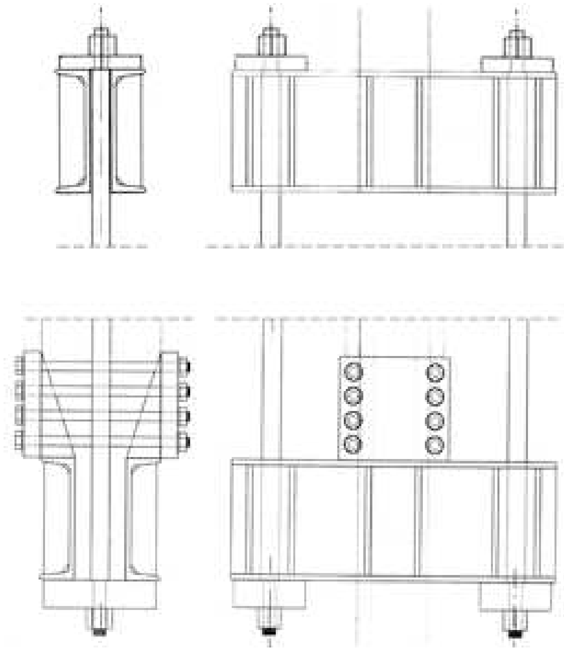

Fig. 12. The new connection.

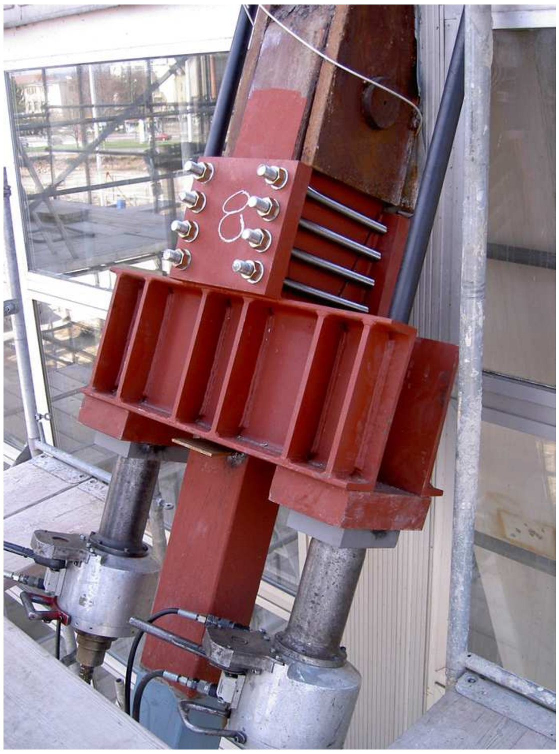

Fig. 13. The pretension of the tendons.

Using four bolted wedges a plane bearing surface was created on the chamfered part of the tension rods. The wedges were connected with 50 mm thick steel plates with eight M27 10.9 screws to the tension rod. Like at the inner construction a pair of 350 mm high U-profiles were placed under the bearing surfaces. Between these profiles the single tendons were placed. At the lower connection the U-profiles and the force transmitting plates were made identical. (Fig. 9 and 12).

## 6. Conclusion

With this for structural engineering unusual restoration technique, a fast and relative inexpensive retroffiting of the roof construction was carried out. By that measures the life cycle the indoor pool could be extended for at least 6 years. The next inspection will be performed in 2012. Moreover, the existing construction had to be modified only marginal, and the typical appearance of the building was preserved. This solution stands for sustainability. For more details see Boehme at all (2009).

In the restoration were involved:
Client: Energie und Wasser Potsdam GmbH
Report, calculation: Dr. Zauft Ingenieurgesellschaft für Bauwesen mbH, Potsdam
Structural design, construction: Ed. Züblin AG, Direktion Ost, Berlin
Proofing engineer: Prof. Hartmut Pasternak, Braunschweig

## References

Acker D., Heinze L. (1969). Montage des Seilhängedaches. *Bauplanung-Bautechnik,* Vol. 23, 534-536.

Boehme W., Pasternak H., Csesznák, A. (2009). Sanierung einer Schwimmhalle in Potsdam. *Bauingenieur*, Vol. 84, 95-100.

DIBt (2006). Allgemeine bauaufsichtliche Zulassung Z 13.1-19 DYWIDAG Spannverfahren mit Einzelspanngliedern.

Franke D., Heinze L. (1969). Schwimmhalle mit vorgespanntem Seilhängedach in Dresden. *Bauplanung-Bautechnik,* Vol. 23, 529-531.

Quade J. (1969). Konstruktion und Statik des Seilhängedaches. *Bauplanung-Bautechnik,* Vol. 23, 532-534.