ABSTRACT: In February 1945 the town Dresden was terribly destroyed by anglo-american bombs. The centre of the town became a big mountain of debris. The Church of Our Lady (Frauenkirche), built up from 1726 to 1743 by the building master George Baehr collapsed on 15th of February 1945 due to the failure of the south-east pillar. Therefore the church after the fall of the wall a group of engaged citizens of Dresden decided to found "The society for the support of the rebuilding of the Frauenkirche". Beginning with this time a lot of people from all regions in Germany and all over the world supported the idea of reconstruction. After an extensive research and planning the rebuilding of the Frauenkirche started in 1994. In 2003 the cupola was completed and in July 2004 the outer reconstruction was finished (Jaeger & Burkert 2001). The paper gives an insight into the principles and structural details of the reconstructed sandstone cupola.

1 DESCRIPTION OF THE BUILDING

1.1 History
On Ash Wednesday, February 15th, 1945 the Frauenkirche with its impressive cupola dominating the city-scape of Dresden over 200 years, collapsed. A 22,000 m³ rubble mountain and two high ruin blunts remained.

With a construction time of seventeen years, starting in 1726, the central-plan church with a floor plan of 41 x 41 m and an overall height of about 90 m was framed up by the master builder of the city council, George Baehr. The outside appearance is defined in particular by the enormous cupola as well as the four corner towers. The cupola starts at a height of almost 40 m and extends up to a height of 62 m (Fig. 1).

1.2 Load transfer in the cupola
The dead loads are the deciding loads for the building. Live loads, wind and snow are practically negligible relative to the dimensions of structural elements.

The total weight of the building-components above 37.78 m is 90 MN (~9000 t). Only about 8% of the total weight results from the lantern including its roof and the cross, i.e. the rest of the weight can be attributed to the cupola. In the opinion of the building master the load transfer from the main cupola should be approximately evenly divided among the piers, the walls of the staircase towers and the masonry walls between them. The system of abutments formed by the radial piers and the y-shaped shear walls going along from the piers have been denominated by George Baehr with the term 'Spierahmen'. From today's calculations and from the damages of the past is known that the planned load distribution does not exist in the necessary quantity in the original building (Jaeger et al. 1997). That's why in the course of the reconstruction of the Frauenkirche this load transfer was optimized, over the condition as it was in George Baehrs' Frauenkirche, through the use of an additional tie ring in the height of the main cornise.
The load transfer in cupolas themselves is represented schematically in Figure 2. Above the so called joint of rupture arise in circumferential direction compression and below them tensile forces. That principle load flow will be influenced by the small and large cupola dormers act as openings in the main cupola of the Frauenkirche, which the forces have to bypass.

1.3 The structure of the cupola

The main elements of the cupola are the exterior shell, the interior shell and the 24 diaphragms located between and connecting the two building-components. The rising spiral ramp, which is part of the visitor stairway up to the lantern platform at a height of approximately 65 m, passes through openings in the diaphragms. Both shells have each been penetrated by eight openings of the upper and the lower cupola windows. These window openings, which are necessary for natural light to enter into the church, detrimentally influence the otherwise perfect load transfer in the cupola (Fig. 3). George Baehr had intuitively recognized the problem with the load transfer in his cupola and had already planned the mounting of ring beams in the first design for his stone cupola.

Overall he located four anchors in the cupola, all consisting of single elements of about 3.20 m in length (Figs 5–6). They were installed at locations in plan where is no interruption of the exterior shell by openings directly under the faced layer from hewn stones. The connection consisted of wedges driven through the eyes forged at the end of the anchors. The anchor cross section varied from 40 mm × 40 mm to 50 mm × 90 mm (Jaeger et al. 1997, Henning 1995–1999).
1.4 Assessment of the old cupola structure

The stone cupola of the Frauenkirche is a masterfully executed feat of our ancestors with significant importance. The erection of the cupola, having been carried out prior to the development of the structural analysis as a discipline, shows the extraordinary efforts of the master builder to construct a long live, stable and unique structure with optimal functionality. The cupola construction was the crown of the main space of the church and ensured very good natural light entering the whole church.

Stress analyses (Rueth et al. 1938, Jaeger & Wenzel 1994) have confirmed the correctness of the mounting position of the wrought iron anchor rings. However, the construction system of the anchors, though at the time of construction at the technological forefront, was not able to absorb the existing thrust without stretching. Therefore cracks meridian direction developed in the cupola. Restoration measures were continually being executed during the roughly 200 year life span of the old Frauenkirche. Thus an overall stability was maintained despite localized damages of the supporting structure, particularly the column capitals and the connecting arches between the piers and shear walls (Spierahmen). As a result of the disastrous fire following the bombings, first one pier failed and then the cupola collapsed into the building and was dashed to pieces.

2 THE DEVELOPMENT OF THE CUPOLA CONSTRUCTION OF THE NEW FRAUENKIRCHE

2.1 Design objective and development of the evaluation criterions

To ensure a long-lasting cupola, during the analysis process special attention was paid to the functionality of the exterior shell construction as well as options regarding the impermeability. What was not at any time disputed was that to minimize cracking the steel inserted in the cupola would need to be post-tensioned.

The post-tensioning by steel bandages located in sheaths under the facing layer of the exterior shell of the cupola was chosen as the preferred solution during the preplanning and after coordination with the client, check proof engineer, the preservationist and the architect (Jaeger & Wenzel 1994).

Various theoretical examinations and calculations, which were verified by small and large-scale tests in the laboratory, in the climatic chamber and outdoors, were carried out to choose the suitable sandstone material as well as an optimal joint filling substance. These investigations were carried out by different specialists in coordination with and supervision by the Engineering Partnership Frauenkirche (Mueller 1999, Mueller et al. 2002, Burkert 2002, Haeupl et al. 2002a,b).

2.2 The structural design

A arithmetical comparison of different variants was carried out before the final design approval (Jaeger & Wenzel 1994), to determine the tendon profile as well as the desired stress-levels in the masonry after post-tensioning.

The result based on all examinations was the arrangement of six anchors. Three anchors each with the cross-section of $3 \text{ cm} \times 10 \text{ cm}$ and $3 \text{ cm} \times 15 \text{ cm}$ are mounted according to the ring tensile forces which are present. Two anchors with smaller cross-sections are mounted, one below the large cupola dormers and one above the small cupola dormers (Fig. 5). The three anchors with the large cross-sections were installed every two ashlar-courses in the zone between the large and small cupola dormers. The high-strength fine-grained structural steel S690QL1 was used in order to minimize the anchor cross-section.

The stressing was carried out by immediately applying the entire prestressing force on every anchor. Determination of the pre-stressing forces was carried out with consideration for the losses resulting from friction, creep, shrinkage and relaxation of the masonry; losses resulting from temperature changes as
well as slippage of the bolted connections of the steel bandages. The total losses are approximately 20% of the pre-stressing force. The pre-stressing forces were set at either 1200 KN or 1800 KN, depending on the specific condition. Tensile stresses in the masonry, resulting from the dead loads in tangential direction, were compensated, after the post-tensioning had been carried out, so that a stress-level of at least 0,2 N/mm² is ensured. The stress-level in the cupola never exceeds 2 N/mm² in any direction. The graphics of the radial stresses clearly show presence of tensile stresses as indicated by the white areas (Figure 7, left), which are almost eliminated after the post-tensioning was carried out (Figure 7, right). The white areas visible in the right figure, indicate zones where the stress is 0.

2.3 The masonry of the new cupola
2.3.1 The masonry bond
The masonry bond pattern of the cupola is analogous to the masonry of the façade. The erection of the masonry of the cupola in the vertical zone was carried out simultaneously with the main structure and while taking into consideration the mounting of the post-tensioning anchors, explained in the following paragraph. Therefore, ashlars with three standard formats of small-sized
stones with either 30 cm or 50 cm depth were mounted as stretcher or header courses in front of the back-up masonry. Adjustments to these standard formats were achieved on site by saws.

While erecting the masonry in the vertical zone of the cupola analogous to the construction of the façade, first the ashlers were dry laid on lead spacer before building the back-up masonry with mortar. After that, grouting mortar was in-filled into the bed and vertical joints between the ashlers. The ribs and the interior shell built simultaneously from small-sized stones of the back-up masonry.

This working technique, to use uniform stone formats for the back-up masonry, still could be continued at the beginning of the cupola inclination up to a height of approximately 47.50 m, but required more complicated preparations of the top and bottom surfaces of the ashlers to ensure that they can be laid with consistent joint thickness of 1 cm.

The increasing inclination also requires a greater variety of stone formats of the stones for the back-up masonry in order to maintain the joint thickness of approximately 1.5 cm in the so-called masonry guideline (Jaeger & Wenzel 1996).

Laying the ashlar courses in which anchors are located and beyond a certain building height, is only possible after the erection of the back-up masonry was completed or the anchors have been mounted. This required a precise preparation of the outer face of the back-up masonry in order to maintain a consistent joint thickness of 2 cm between back-up masonry and ashlar shell. An additional problem was the accurate form fitting of the ashlers, which can only be verified by means of regular geodesic measurements. The mounting of the anchors with their technological constraints as well as the construction of the small cupola dormers are additional reasons why a precise logistic, relative to the means and methods of constructing the cupola, was so essential.

2.3.2 Choice of the material

The choice of suitable cupola stones and mortars was a complex and extensive process, which could only be completed with the aid of specialists. A long time was needed to find a suitable sandstone material and the optimal mortar. Additionally the theoretical mortar development in the laboratory was supported by a weathering test, which was carried out at the Department of Structural Design, Faculty of Architecture of the Dresden University of Technology.

A part of the exterior shell of the main cupola was mocked-up and the stress state as it would be in the actual building, was simulated. This masonry specimen with the dimensions \( L \times B \times H = 3 \text{ m} \times 1 \text{ m} \times 1.8 \text{ m} \) was erected on a steel frame with the critical inclination set at \( 45^\circ \). Measuring devices were installed in and on this masonry body. The masonry body was enclosed in a climatic chamber. Within this climatic chamber temperature and moisture were modified from an initial fixed climate. Through this the Dresden climate was simulated for a 100 year time-span (Burkert 2002).

The results were continuously recorded and evaluated in the laboratory. A conclusion about the suitability of the tested stone-mortar-stone connection as then determined.

The experiments and their results, for which the financial support was generously provided by the Deutsche Bundesstiftung Umwelt (German Federal Foundation for Environment), formed the basis for the
erection of a permanent cupola construction. The gathered results are not limited in scope to the choice of the sandstone and the mortar. Additionally could be concluded, to execute the ashlar joints with roughened surfaces, to improve the bonding tensile strength of the surface shell. Practical indications were formulated for the execution of the masonry, the mounting and for the post-treatment of the mortar. Further measures were necessary to prevent water damp condensation at the inner surface of the exterior shell in case of lower surface temperature and higher damp content of warm air in the transition period of the year. The internal climate in the space between the both shells will be regulated in dependant on the current weather.

Therefore an optimization of the stone-mortar-stone connection for the cupola of the Frauenkirche was carried out within financially acceptable amount and with the application of the current knowledge in science and engineering (Mueller 1999, Mueller et al. 2002). So a high quality of the main cupola could be guaranteed on the basis of all gathered results and of a technically well executed construction of the dome. The prerequisite for longevity of the cupola is the regular maintenance and care of the exterior shell. The intactness of the joints has to be particularly checked in specified time intervals.

2.3.3 The complexity of the construction process

Prior to the mounting of the anchors, the small-sized units of the masonry backup had to be layed up to the upper edge of the sheath. Then the geodesic lineup was carried out for the starting-points of the bolts to mount the base metal strips with a length of 5 to 6 m as support for the anchor ring. After this remaining bolts were mounted. Then the sheet metal strips had been assembled on the bolts (Fig. 10). After all bolts and strips are attached, the sheet metal has to be exactly positioned in plan via geodesic lineup.

The areas between back-up masonry and base strap had to be filled and grouted to ensure the friction-fit connection to the masonry. The anchor had been mounted after a short setting time. This was carried out by four bolted connections and setting in the target position. The tubes (with outlet openings) for post-injections were mounted either before or assembling the anchors, depending on the specific conditions. Then the back-up masonry was completed up to the surface of the ashlar layer which is located in front of the sheath. Laying of the ashlers in front of the sheath then follows and grouting the lower vertical joints and the bed joints of the facing was carried out. In doing so, expansion strips had to be installed in the joints. This was required to be able to ensure the grouting of the ashlers without the grouting mortar entering into the sheath. Exceptions for all described procedures had to be considered in the area of the tensioning pockets. Figure 11 shows the state during construction and before stressing in the area around the tensioning pockets, the tubes for post-injection as well as cables for strain gauges which still have to be run.

2.4 The anchors of the new cupola

One complete anchor consists of eight parts, each up to a length of 10 m. In each case two pre-bent parts corresponding to the external radius of the cupola are connected through a butt strap connection and fitted with bolts after mounting (Fig. 12). So four quarter segments arose which were stressed at four points of the circumference. It was necessary to glue on the backside of the tensioning steel bandages a continuous sheet from polytetrafluoroethylene (PTFE) for the reduction of the friction during the stressing.

For optimum distribution of load from the stressing anchor to the cupola masonry and for precise

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Figure 10. Installation of the base metal strip as support for the anchor ring.

Figure 11. Tensioning pocket before injection with the different tubes for injection and exhaust.
positioning, a steel sheet was mounted under the anchor before its assembly.

The stressing of the anchors was carried out at the tensioning pockets which were not yet closed. In that time masonry over the anchors existed at a height of at least 1.50 m which has to have set and cured for at least 28 days. The groove-shaped recess for mounting the anchor ring was closed except of the tensioning pockets so that the sheath was formed by the dimensioned units and the masonry backup.

2.4.1 Pre-stressing of the anchors
The stressing technique has to be developed in such a manner as to ensure that the two anchor ends, which exist at each of the four stressing points of an anchor, can be clamped into position, be stressed and kept from slipping during the time of drilling and inserting the bolts and finally be released again (Figs 13–14).

Steel plates like flanges were already welded to the top and bottom of the anchor ends at the tensioning connection before delivery to the building site. On both sides of the connection, vertical stressing plates were pressed against these flanges. The pre-stressing heads with the elbow and the supports for the tensioning jacks are screwed on the right vertical stressing plate. Two auxiliary tendons at each stressing point, one above and one below the anchor, are used to introduce the stressing force into the anchor. Every tendon consists of four strands, each with a net cross-section of every 200 mm² and a tensile strength of 1770 N/mm².

The length of the temporary tendons was very short being only approximately 3.30 m. For this a precise length adjustment was required among the various strands before stressing. This was achieved using special screw adjusters at the stressing jack head. The anchorage wedges in the casings of the two outer anchors which get fixed into place under loading, are blocked by wedges that prevent re-opening. Unequal strand lengths would cause the shifting of the connections and the equipment during stressing which could culminate in the partial destruction. Spacer-wedges are inserted between the stressing connections to avoid stressing of the anchors during elongation of the strands under loads.

The bracing at the tensioning jacks for the middle anchorage point had the task to sustain the stressed anchorages and to allow the insertion of de-stressing sliders as well as the spacer-plates. If necessary, the de-stressing sliders would allow the complete de-stressing after stressing. The spacer-plates serve...
Figure 15. Stable drill mounting bracket ensures exact positioning of the drilled holes.

Figure 16. Exhaust tubes for injection in a collecting basin.

for the compensation of slippage. Slippage arose when decreasing the stressing force at the end of the stressing process at the middle anchorage point. Additional pre-stressing was necessary for inserting the spacer-plates. The pre-stressing jacks have elbows that lead away from the building thus leaving the line of the radius of the anchor. The arrangement of the two tensioning jacks per stressing point is thus possible (Fig. 15).

Bracings of the supports of the tensioning jacks and the hydraulic pumps with their manometers comprised the elements of the complete tensioning jack equipment for stressing.

The stressing force was applied with great precision at all anchor points, which was controlled at the first anchor ring by strain gauges. After the pre-stressing was finished, the steel connection plates have to be connected at the stressing points.

After pre-stressing, the remaining stressing pockets were predominantly closed prior to the injection of the special grouting mortar, both into the pockets and sheaths (Fig. 16).

3 EXHAUST TUBES FOR INJECTION IN A COLLECTING BASIN

3.1.1 The corrosion protection of the anchors of the new cupola

One of the construction principles for the anchors was that a depth of 30 cm of ashlar masonry was available as protection against the elements. However, this does not prevent the penetration of any moisture through the masonry, reaching the anchor. Damages to the stones will almost with certainty not occur; but the stone material is porous. Micro cracking in the joint range can occur. But on the other hand it is improbable that a crack will run the entire depth of the protective thickness where the anchor is located. However, corrosion danger cannot be completely excluded. Therefore, the task at hand was to produce a permanent and constant protection of the fine-grained steel. This could be ensured by the following construction details:

- construction of the sheath so that a 5 cm cover of injection material was possible on all sides
- using an injection material with a water cement ratio of 0.4 and an injection additive to ensure as low moisture penetration into the masonry around as possible
- galvanizing the base strap to improve the temporary corrosion protection during the time between the installation of the anchor and injection
- elutriation of the wall surfaces of the sheath by cement grout to prevent the fast dehydration of the injection material by hydrating the sandstone

Injection tubes and tubes for exhaust and overflow are placed in the masonry around. Beside the tubes that remain in the building, tubes for discharge and supply of the grouting mortar were mounted additionally.

The complete injection process follows working instruction with a detailed time schedule. After all 6 anchors had been imbedded in grouting mortar, the anchor’s channels were additionally post-grouted through specially tubes.

4 CONCLUDING REMARKS

The reconstruction of the Frauenkirche has been a continuous work in progress since 1994. The speed of construction has directly increased the further the project gets along. Designers and contractors have gathered experience throughout. What a few years ago was considered a problem, now has been resolved by technical solutions and realized in practice.

Since July 2004 the completion of the building shell of the Frauenkirche was carried out by attaching the lantern bonnet and the dismantling of the last outer scaffold.

Now the Frauenkirche is back in the silhouette of the Dresden city. The rebuilding followed the basic idea to
be as close as possible to the original structure applying the main principles and the plans of the building master. By the way the latest engineering knowledge was carefully introduced to fulfill the today's building safety requirements and to allow for a widely use for services, performances, concerts and other events. It was a decision of the citizens and the friends of Dresden to reconstruct that unique baroque church in the original form and structure and not of the science of preservation of architectural heritage. The Dresden Citizens would have back theirs midpoint of the city and its social life. The world should accept that. The result is convincing.

The ceremonial consecration of the church will then be carried out on 30th of October 2005.

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REFERENCES


SOURCES

Figure 1 is from the City Archive Dresden, followed up by Jaeger Ingenieure GmbH, Wichernstrasse 12, 01445, Radebeul, Germany
Figure 2 taken from Buettner & Hampe 1977
Figures 3–4, 8 made by IPRO Dresden, Architekten- und Ingenieuraktiengesellschaft, Schnorrstrasse 70, Postfach 200947, 01194 Dresden, Germany
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