Prestressing cracked masonry for restoration

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Prestressing Cracked Masonry for Restoration

Renforcement de maçonnerie fissurée par des armatures précontraintes

Sicherung von gerissenem Mauerwerk durch Vorspannen

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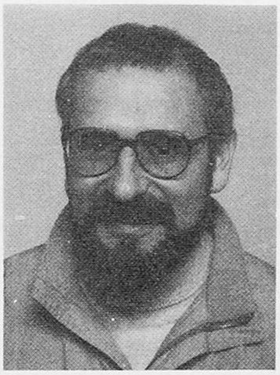


Jürgen Haller received a diploma as civil engineer at the University of Karlsruhe in 1966. 8 years of practice in structural engineering were followed by 5 years of scientific research on prestressing masonry at the University of Karlsruhe. Jürgen Haller now works as a consulting engineer with emphasis on the restoration of old buildings.

SUMMARY

This article describes the aims and the technology of prestressing cracked masonry for restoration. A report follows about research conducted on the problem of imposing concentrated loads upon masonry and about long term measurements of the forces in built-in tendons and reinforcement bars. As a result, proposals are made for construction and calculation.

RESUME

Ce rapport décrit le but et la technologie du renforcement de maçonnerie fissurée par des armatures précontraintes. Des recherches sont présentées concernant le problème de forces concentrées dans la maçonnerie. De plus des mesures de longue durée sont faites pour surveiller les forces des armatures précontraintes et non précontraintes dans la maçonnerie. L'auteur présente les résultats de ses recherches et fait des propositions pour la construction et le calcul.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt zunächst Ziel und Technik der Sanierung von gerissenem Mauerwerk durch Vorspannen. Es folgt ein Bericht über Untersuchungen zum Problem der Einleitung konzentrierter Lasten in das Mauerwerk, und über Langzeitmessungen der Kräfte an Spannstäben und nicht vorgespannten Bewehrungsstäben, die im Mauerwerk mit Verbund eingebaut wurden. Als Ergebnis der Untersuchungen werden Vorschläge für das Konstruieren und Berechnen angegeben.

1. THE OBJECTIVE OF RESTORATION BY PRESTRESSING

Damages in historical and modern masonry buldings become visible as cracks in walls, arches and vaults. In many cases they just affect the appearance and the use of the building. But as soon as the central stone of a vault falls down or the shaft of a tower separates into several parts that move away from each other the stability is at stake.

If walls are thicker than 45 cm, which is usually only the case in historic buildings, it is possible to repair the cracked masonry by prestressing. The method leaves no visible marks on the outside. The substance of the masonry is preserved this way. It is strengthened internally to absorb future loads without further cracking.

2.TECHNOLOGY AND EFFECTS OF PRESTRESSING MASONRY

2.1 Technology of prestressing

First a hole is drilled lengthwise through the masonry, normally using drills 60 to 80 mm in diameter. A continous prestressing bar is inserted into the drill-hole and anchored at both ends of the wall. A centre position in the hole is secured by distance pieces. Threaded rods can be used in handy, short pieces, an important advantage when working on a scaffold. They can be connected by couplers and be anchored at the end with a washer and steel plate. To transfer the prestressing force to the masonry an enlarged pressure area is needed. This is achieved by creating concrete cushions or steel anchor plates in pockets in the masonry that can be covered by plaster or by fair-faced masonry.

In order to pretension the wall the prestressing bar is anchored permanently at one end and tensioned at the other end by a hydraulic jack where it is then achored as well. To prevent the sensitive prestressing bar from corroding a cement-water paste is injected into the drillhole after pretensioning. The injection pressure ranging from 2.0 to 5.0 bar at the most, the cavities and cracks in the surrounding masonry are also filled. This way the wall receives a new mortar skeleton. More significant cracks and disturbances in the masonry bond, though, should be grouted before pretensioning.

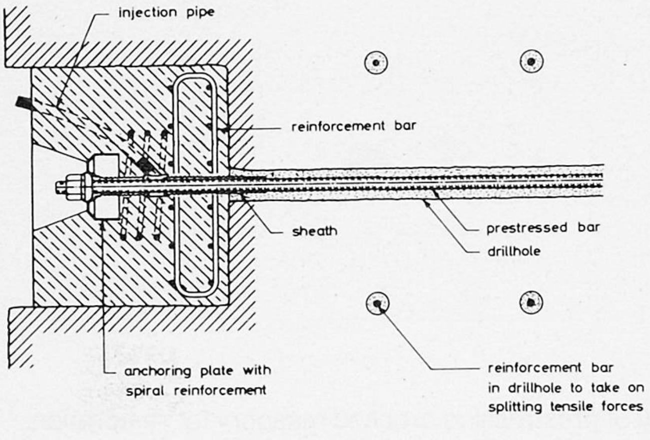


Fig.1 concrete cushion with anchorage

2.2 Effects of prestressing

Cracks in masonry indicate that transverse tensile forces are active. If prestressing bars are arranged at a right angle to the direction of the cracks pressure is applied at this angle. This means that as long as the pretensioning pressure forces prevail further tensile forces can be absorbed. The constructional securing of crack damages (Fig.2) is not the only field of application for prestressing. It is also possible to improve the flow of forces and the transfer of loads within masonry by pretensioning. This can activate sheet action or bowstring action within walls to allow for openings or larger intervals between single supports (Fig. 3, 4).

In masonry that shows vertical or oblique-angled cracks prestressing bars are inserted horizontally. This case of prestressing parallel to the horizontal joints is by far the most frequent.

Prestressing at a right angle to the horizontal joints is applied much more seldom. It may be sensible to arrange prestressing bars eccentricly to correct off-centre load transfer if horizontal joints show one-sided gaps. Additionally, vertical pretensioning can increase the stability of walls that have too little load to withstand pressure from wind or to resist earthquakes.

It is a special advantage that prestressing bars have small cross sections and nevertheless can impose great forces. The types of steel used for this purpose have great material strength. Due to the spread of pressure behind the anchor plates each prestressing bar has an extensive area of action. Because of this, expenditure for drilling work can be kept to a minimum by using bars with a larger cross section and arranged at greater intervals.

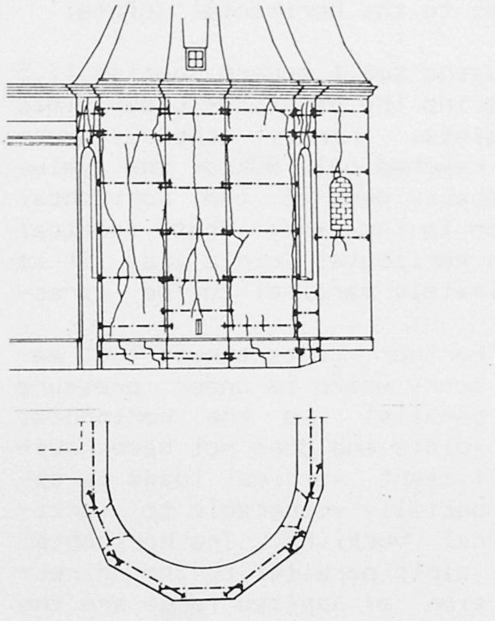


Fig.2 constructional securing of crack damages

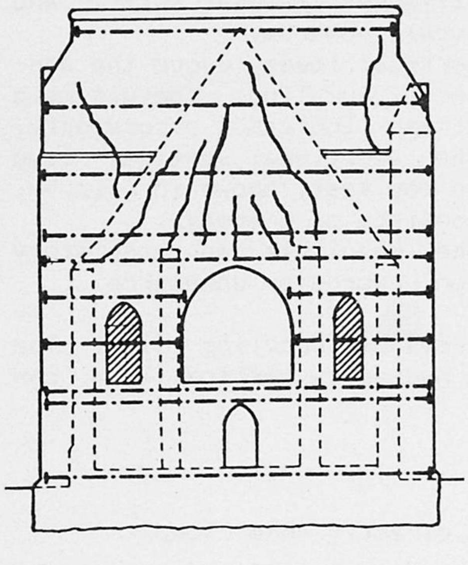


Fig.3 sheet action above an opening in a wall

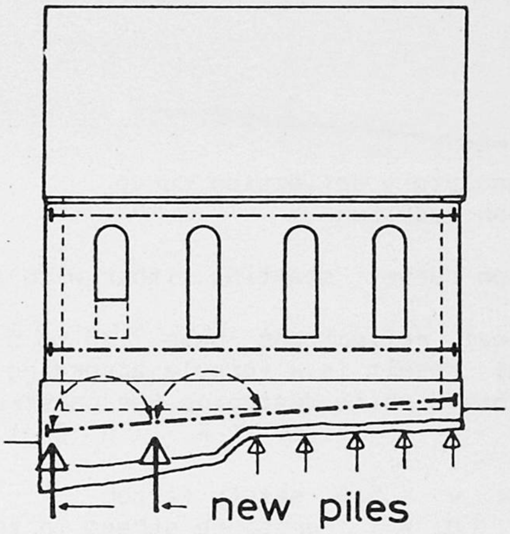


Fig.4 bowstring action

3. RESEARCH CONDUCTED ON PROBLEMS CONCERNING THE APPLICATION OF PRESTRESSING MASONRY

3.1 Imposing highly concentrated loads upon masonry

The size of prestressing forces is limited by the problems arising with imposing those concentrated loads upon masonry. Our knowledge of this problem is insufficient. The bearing capacity of masonry has been examined mainly for vertical load impact, that is at right angles to the horizontal joints. Prestressing, though, generally applies lateral pressure. This brought up the first question of interest for scientific research [1]: to clarify the behaviour of masonry under load applied parallel to the horizontal joints. To examine this, tests were conducted on samples of brickwork. The samples were made of solid brick masonry which was used very often in historic buildings.

3.1.1 The compressive strength of masonry parallel to the horizontal joints.

First of all the material strength was examined using small square samples 11,5 cm thick and 50 cm in lenght and height and comparing the behaviour under load parallel and perpendicular to the horizontal joints. Thereby, the ultimate stress achieved parallel to the horizontal joint reached only 80% of the value received for perpendicular load. This result probably owes to the horizontal joints that already split under low pressure and to the fact that vertical joints are generally filled with less mortar than horizontal joints are. It is significant in both cases that cracks run approximately parallel to the direction of the force applied.

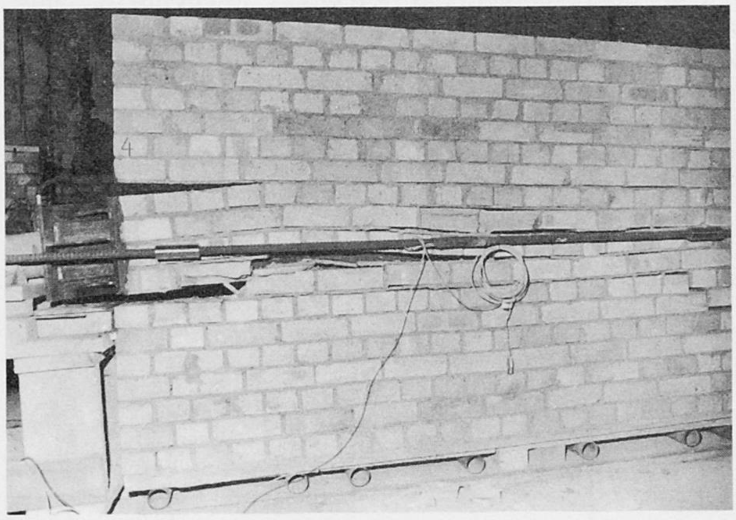


Fig.5 undulatory deflection curve upon buckling

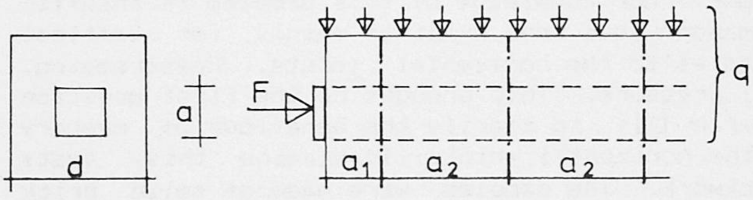
Further tests showed that masonry which is under pressure parallel to the horizontal joints and does not have sufficient vertical loads is especially vulnerable to vertical buckling. The horizontal joints parallel to the direction of applied force are the weak spots. They are torn apart fairly soon due to transverse tensile stress. The different layers split and buckle upwards.
Vertical loads reduce the danger of buckling. The ultimate stress increases accordingly. The additional vertical load in the test, shown in fig.5 , consists of masonry.
The result is an undulatory deflection curve, starting either with an undulation upwords or downwards.

Theoretical reflections have led to 5 possible cases of buckling [1] . The practical result is a formula according to which a necessary vertical load can be determined while designing the construction:

$$q = \nu \cdot 1{,}67 \cdot \sigma_{\parallel} \cdot F \; / \; (h \cdot E_{\parallel})$$

in which: ν = 5,0 safety factor
$\sigma_{\parallel}$ = pressure stress in those layers directly under load
F = prestressing force
h = layer height
$E_{\parallel}$ = elastic modulus of masonry parallel to the horizontal joints

If the vertical load is not sufficient either the prestressing force or the pressure stress in those layers which are directly under pressure must be reduced accordingly. Another solution is to arrange reinforcement bars in the masonry wall crown (Fig. 6). Reinforcement bars are intensioned and are inserted into drillholes in masonry along with spacers and then grouted with a cement-water paste. A formula has been derived to determine the space between the bars which has to be smaller than the critical buckling length.



$$a_1 = 0{,}246 \cdot \sqrt[4]{\frac{h^3 \cdot E \cdot a \cdot d}{q}}$$

$$a_2 = 2{,}4 \cdot a_1$$

Fig. 6 intervals between vertical reinforcement rods at the masonry wall crown

3.1.2 Partial surface pressures parallel to the horizontal joints

The anchoring elements of prestressing bars apply force only to partial areas. Behind this locally highly concentrated transfer of force the pressure stress spreads out until, after a certain length of introduction, the masonry is compressed in its entirety. The spread of pressure causes splitting tensile forces approximately transverse to the direction of the applied force.

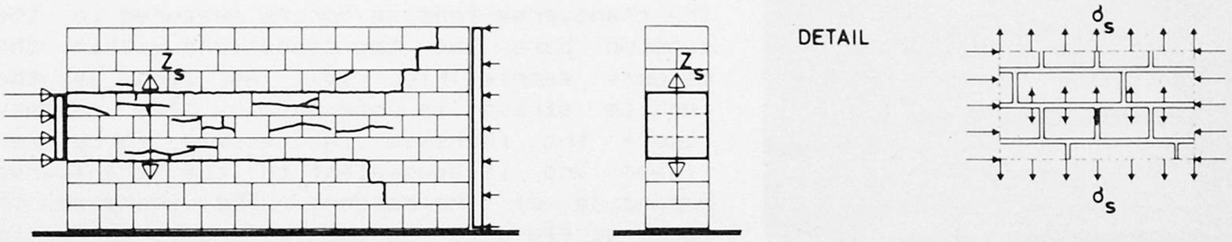
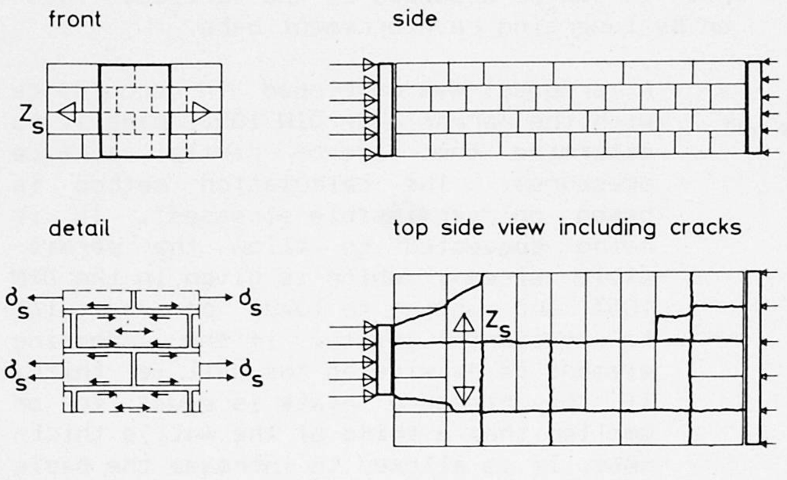


Fig. 7 splitting tensile forces perpendicular to the horizontal joints

Two cases must be distinguished when dealing with masonry, that is under load parallel to the horizontal joints: If the breadth of the pressure plate equals that of the masonry wall the splitting tensile forces act perpendicular to the horizontal joints and accelerate the process of splitting. The spread of pressure into peripheral zones is reduced until the layers directly under load remain to transfer the force alone. The masonry fails by showing horizontal cracks in the bricks (Fig.7).



δ_s = splitting tension

Z_s = splitting tensile force

Fig.8 splitting tensile forces parallel to the horizontal joints

If the pressure plate is smaller than the wall is thick then pressure spreads transverse to the plane of the wall. The splitting tensile forces act perpendicular to the vertical longitudinal joints, which are covered, according to the rules of masonry bond, by bricks in the layers above and below. The partial surface pressures reached in the moment when first longitudinal cracks occur were considerably higher than achieved when pressure only spread out in the plane of the wall. As expected, the masonry failed by cracking along the longitudinal joints. In some cases the cracks run through bricks towards the longitudinal joints (Fig.8).

During practical restoration work on masonry, reinforcement bars are arranged transverse to the wall plane behind the anchorage. This prevents the masonry from splitting and cracking and also helps stabilize the area of force introduction by grouting before the prestressing force is applied (Fig. 1 and 15).
This procedure was simulated with a few masonry samples by inserting 6 tension bars and cross bars. In comparison with the tests without tension bars the resulting partial surface pressures at the moments of first cracking and failure were increased by about 30% . The cracks were distributed more evenly than without tension bars (Fig. 9). In further tests the tension bars were arranged vertically. This way upward buckling of courses of brick should be ruled out as

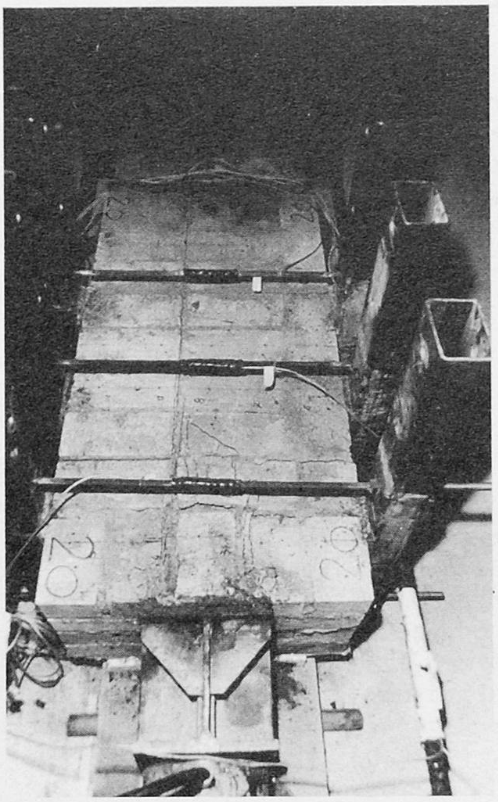


Fig. 9 transverse horizontal tension bars

a cause of failure. The beginning of cracking in the horizontal joints was not influenced substantially by the tension bars. The stage of failure, though, was reached much later than with comparable samples without tension bars. The ultimate pressure was about double the value of similar samples without tension bars.

The transverse tensile forces measured in the tension bars show the conditions within the masonry sample (Fig. 10). As long as the tensile stress is absorbed by the masonry itself the increase in tensile force is linear and is equivalent to the transverse expanding of the masonry. The increase in force accelerates as soon as cracks appear in the horizontal joints and later in the bricks as well. The internal tensile stress in the masonry is increasingly transfered to the outside tensile bars. The ultimate forces in the bars reach about the size of calculated splitting tensile forces.

For designing, an adequate safety margin toward failure of the masonry is needed. Therefore it is possible to use the value of the calculated splitting tensile force and prove that it can be absorbed by the vertical load or by inserting reinforcement bars.

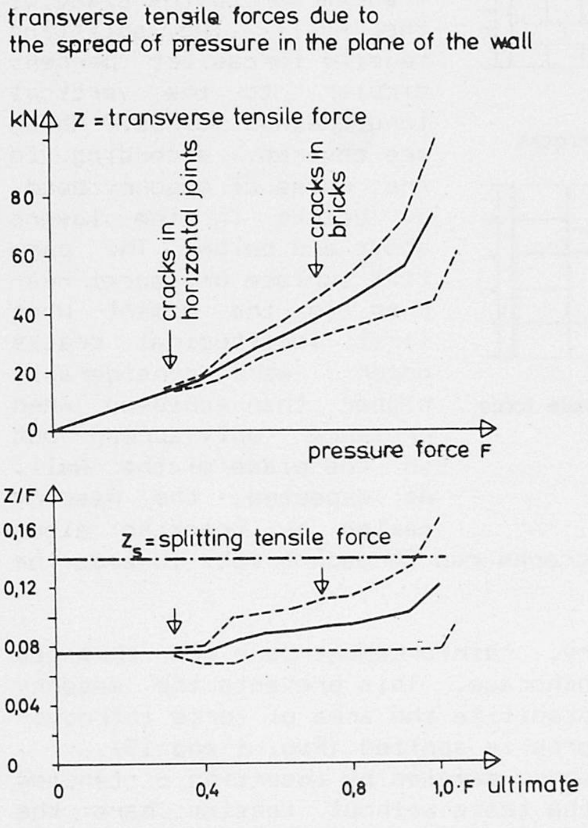


Fig. 10 vertical transverse tensile force

A proposal was developed in accordance with the German code DIN 1053, page 1, to determine the size of partial surface pressures. The calculation method is based on "permissible stresses". It is being suggested to allow the permissible stress, which is given in the DIN 1053 and applies to loads perpendicular to horizontal joints, if the anchoring element is as wide as the wall is thick. If the pressure plate is equal to or smaller than a third of the wall's thickness, it is allowed to increase the basic value 1,5 fold. Intermediate values can be determined linearly. Most important was the safety towards cracking which totals 1.5 . The greater the distance to the anchoring elements the less pressure stress there is in the masonry. If the danger of cracking is minimal, it is sufficient to determine prestressing forces that will create an average lateral pressure stress of between 0.1 and 0.2 N/mm^2. In all other cases it is necessary to carry out statical calculating of the construction elements to determine the size and position of prestressing forces that are needed to definitely avoid tensile stress in the masonry.

3.2 Measuring the prestressing forces of built-in prestressing bars.

Besides dealing with the problems already mentioned concerning the introduction of prestressing forces into masonry the durability of applied tensioning forces had to be clarified. Experience with prestressed concrete shows that concrete shrinks when it hardens, it dries out, and it creeps under pressure, it shortens, whereby part of the applied tensioning force is lost. This is the same with masonry. Deformations occurring after prestressing lead to changes in the prestressing force. The deformation behaviour of modern types of masonry is generally known. The already determined data, though, cannot be applied to masonry of historic buildings which includes significant disturbances of its bond. To gather experience I conducted measurements on several restoration sites over a number of years. Thereby measurements were carried out during construction and afterwards on the built-in prestressing bars with the help of strain gauges.

3.2.1 The influence of the wall temperature

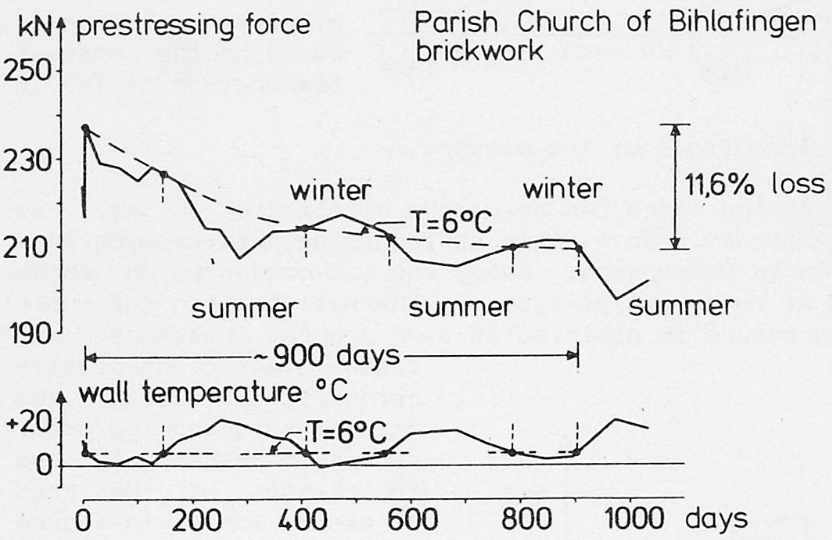


Fig.11 course of prestressing forces due to the wall temperature

Besides a falling tendency at the beginning the measurement curves show seasonal changes that are opposite to the course of the wall temperature shown below (Fig. 11). With increasing temperatures the force within the prestressing bar declines, whereas towards winter, when temperatures fall, the tensioning force grows, caused by the different coefficients of expansion α_T of steel and masonry. These coefficients could be determined from the measuring data.

With $\alpha_T = 11.5 \times 10^{-6}$ of steel, the following values were determined:

for brick masonry $\alpha_T = 6,3 \cdot 10^{-6}$

and for sandstone masonry $\alpha_T = 7,7 \cdot 10^{-6}$

The long-term tendency of the curves becomes more obvious if points of the same wall temperature are connected in the curves shown by the dashed line in fig.11. This procedure eliminates another cause of seasonal changes, though, which shows only after the influence of temperature has been compensated by way of calculation for every single measuring spot as shown in fig. 12.

3.2.2 The influence of moisture in the wall

The curve, in which temperature changes have been compensated for, still shows seasonal swinging. The amplitude is smaller and the high and low points are displaced by about three months compared to the curve of wall temperature. Because the creep deformations occur mainly during the time of high summer temperatures some relation to the moisture content of the walls can be assumed. In summer the masonry dries out, creap is accelerated, in other words: if the creep of masonry happens continuously then the curve of creep is overlaid by an annual swinging caused by swelling and shrinking of masonry. The annual changes due to temperature and moisture are insignificant for the determination of prestressing forces, because they do not lead to permanent losses.

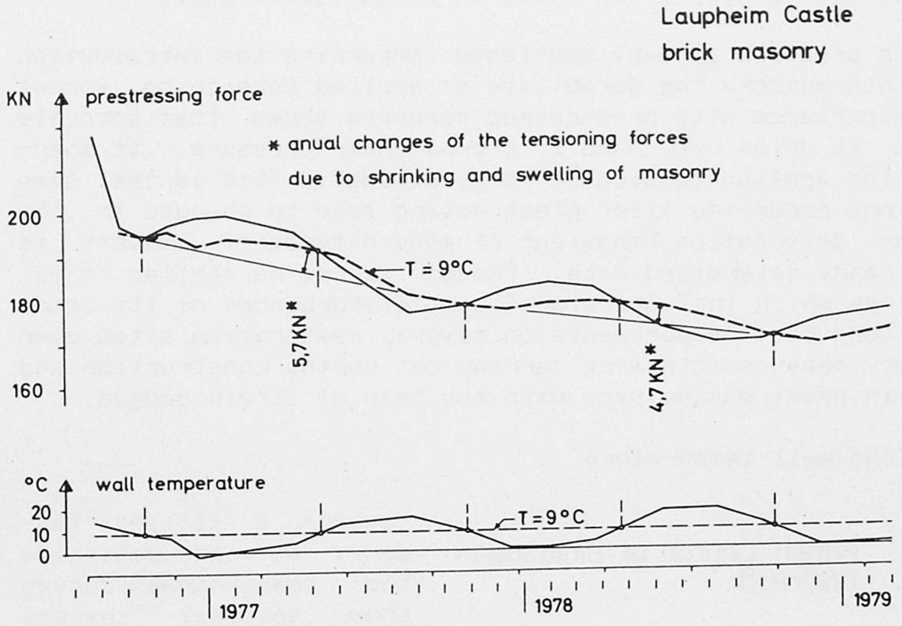


Fig.12
the calculated course of prestressing force based on the constant temperature of T=9° C

3.2.3 Elastic and plastic deformations of the masonry

Permanent losses of prestressing force can be caused by elastic as well as plastic deformations of the masonry. An example could be the prestressing of a wall in the Collegiate Church in Herrenberg. Measuring was conducted on three prestressing bars, arranged at intervals of 1,3 m in the wall between the choir and the vestry. The applied pressure is distributed over a great distance and causes elastic and plastic deformations along the axis of neighbouring prestressing bars that lead to changes of the prestressing force. In figure 13 the bottom diagram shows 3 attempts to prestress the lower bar whereas the top diagram shows the changes in pretensioning force in the already stressed upper bar. The loss at the beginning is greater than during further burdening and relieving procedures. It consists of elastic deformations as well as of a portion of plastic shortening which effects the bar. The complete loss on the first day totalled 5.4% . These initial losses in pretensioning stress can be avoided by applying excess pressure to begin with or after the losses have occurred. It is not possible, though, to do the latter, if the drillhole has already been grouted, because of the bond between masonry and the bar.

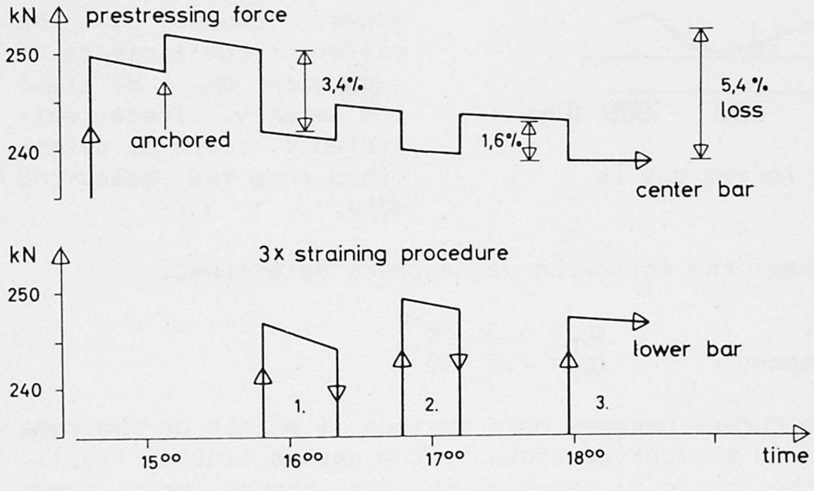


Fig.13 correlation between the bars during prestressing

The course of active tensioning force stabilizes after grouting. The plastic deformations of the masonry happen more slowly. There are only few measuring curves, though, that match the path of the creep curve to the same extent as the one in figure 12 showing the greatest at the beginning and smaller losses afterwards. The course of the curve is usually disturbed by effects resulting from the foundation, the rest of the building or from restoration work. Drilling in the masonry, work with a pneumatic hammer during excavating, settlement movements and underpinning cause spontaneous plastic deformations in the masonry that cannot be regarded as creep in the actual sense. The cause is not pressure stress,which leads to creep , but vibrations,that make the masonry give way. Figure 14 shows the most striking example measured.

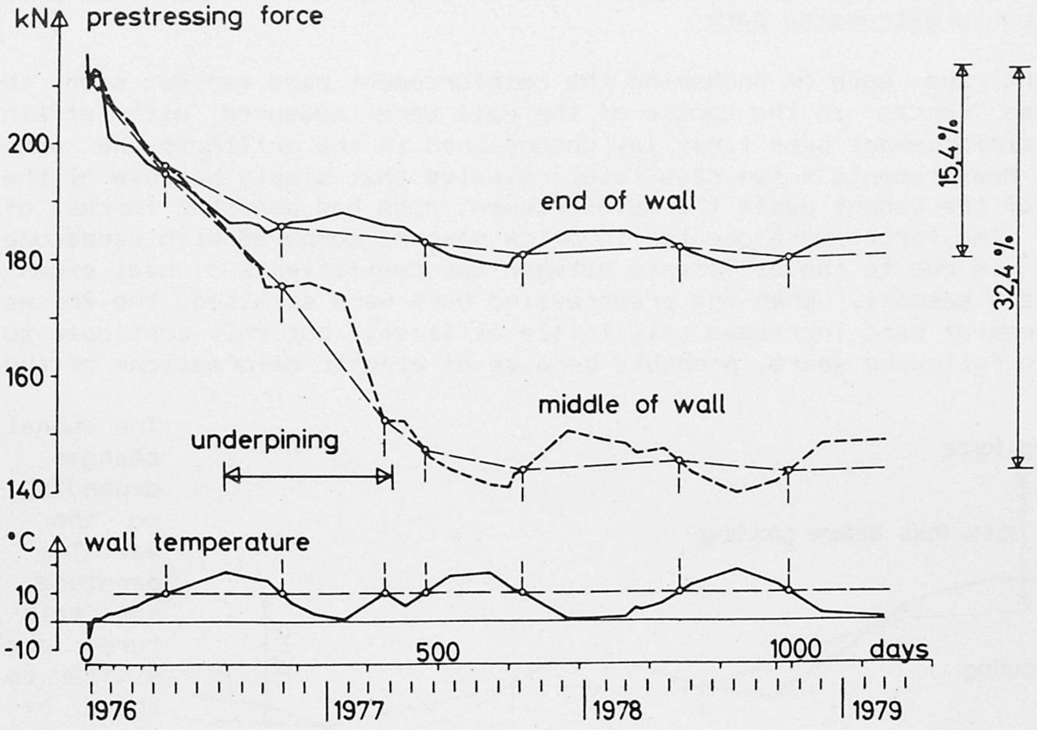


Fig.14 measuring curves showing great losses of prestressing force due to underpinning and excavating work

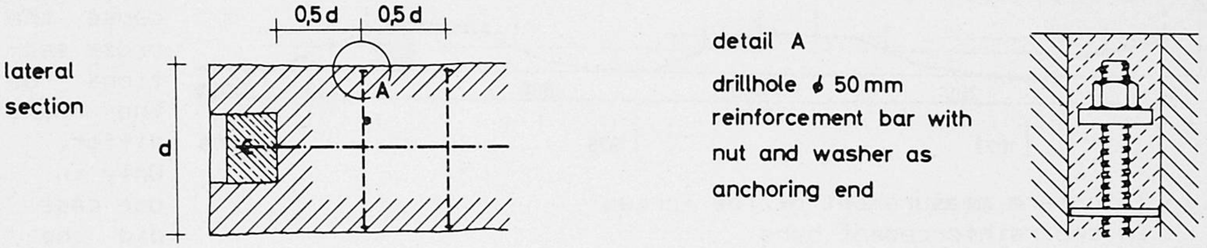


Fig.15 reinforcement rods in the zone of force introduction through prestressing bars

The size of losses of prestressing force usually remained under 15% . Because creep deformations happen over a long period of time, examinations conducted by Hage [2] were used to establish a prognosis on the final amount of creep. According to his results about 80-85% of all creep deformations occurred within 2 1/2 and 3 1/2 years. For dimensioning, coefficients of creep φ_∞ were derived from the measuring results. With their help the expected losses of tensioning stress ΔZ_K can be determined.

$$\Delta Z_K = \varphi_\infty \cdot \sigma_{M,V} \cdot A_V \cdot E_V / E_{\parallel}$$

$\sigma_{M,V}$ = pressure stress in masonry due to prestressing
A_V = cross-sectional area of the prestressing bar
E_V = elastic modulus of the prestressing bar
$E_{\parallel}$ = elastic modulus of masonry parallel to the horizontal joint

As long as no other measuring results are available, the following can be assumed: φ_∞ = 1,3 till 1,6 and $E_{\parallel}$ = 1000 N/mm^2.

3.3 Measuring the forces of reinforcement bars in the zones of force introduction through prestressing bars

The arrangement and mode of anchoring the reinforcement bars can be seen in figure 15. The forces in the centre of the wall were measured with strain gauges. The reinforcement bars first lay untensioned in the drillhole and were then grouted. Measurements a few days later revealed that simply because of the setting heat of the cement paste the reinforcement rods had absorbed forces of up to 5.6 kN . The forces were greater in brick masonry compared with sandstone masonry. This is due to the difference between the coefficients of heat expansion of steel and masonry. When the prestressing bars were strained, the forces in the reinforcement bars increased only little at first, but they continued to grow during the following years, probably because of plactic deformations of the masonry.

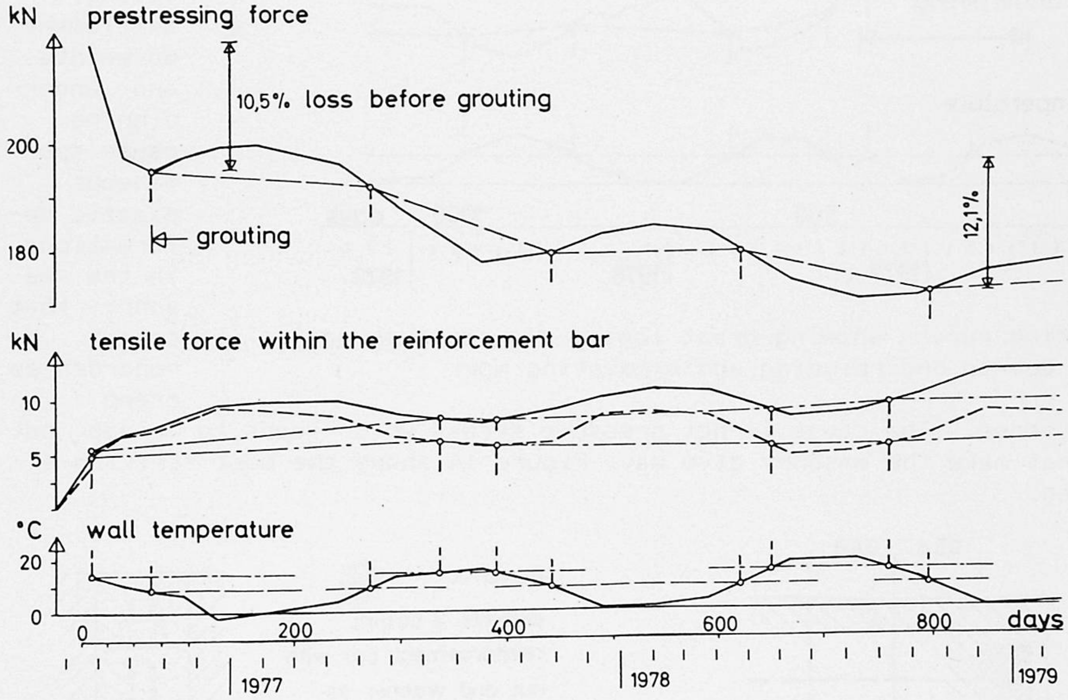


Fig.16 long-term measurement of the forces in two reinforcement bars

The annual changes depending on the wall temperature and moisture are similar to those of the prestressing bars exept that they are smaller in absolute value because the cross sections of the bars differ. Only in one case did the size of forces exceed the splitting tensile force calculated for the bar. Because of incertainties concerning the distribution of tensile stress within the masonry, it is suggested to dimension the reinforcement bars for 1.5 fold of the proportionate splitting force. For reasons concerning the securing of cracks the permissible stress for steel should be restricted to 120 N/mm^2 = 1200 kp/cm^2 according to the DIN 1053, irrespective of the steel quality.

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