W. Steiner, P.P. Rossi, P. Devin

Flatjack measurements in the lining of the Hauenstein-Tunnel as a design base for the new Wisenberg-Tunnel

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ismes spa □ viale G. Cesare, 29 - 24100 Bergamo (Italy) □ telex 301249 Ismes I □ tel. (035) 358111 □ fax (035) 211191

Flatjack measurements in the lining of the Hauenstein-Tunnel as a design base for the new Wisenberg-Tunnel.

Walter Steiner,
Ingenieurgemeinschaft Wisenberg-Tunnel, Bern, Switzerland

Pier Paolo Rossi,
ISMES S.p.A., Bergamo, Italy

Pierre Devin,
ISMES S.p.A., Bergamo, Italy

ABSTRACT: The new double-track railway Wisenberg-Tunnel through the Jura mountains with a length of 12.6 km is planned in Switzerland. Tunnels crossing the Jura Mountains have to cope with swelling pressures which are particularily severe in formations with anhydritic shale. Past experience with tunnels in such rock was not always satisfactory. The existing Hauenstein-Base-Tunnel, roughly parallel to the Wisenberg-Tunnel, built 1912-1916, with major reconstruction work performed 1919-1923 and 1980-1986, provides an excellent longterm in-situ fullscale test on lining behavior and performance. A series of 35 flatjack measurements was performed in several sections of the Hauenstein-Tunnel. These results combined with the past performance of the tunnel give substantial insight in the range of swelling pressures and their development with time. The measurements and their interpretation are discussed and put in context with other experience from tunnels in swelling rock.

1 INTRODUCTION

In the future the railway transportation system in Switzerland will be improved. For passenger traffic the system "Rail 2000" shall be created. This is a system based on a series of hubs linked at least by a train every hour. Travel time between adjacent hubs has to be somewhat less than one hour. On parts of the railway system speed and capacity of existing lines no longer suffice. In addition northern and southern Europe shall be linked by a new base tunnel through the Alps which will serve as a link for freight and passenger traffic.

These traffic flows require the creation of a second pair of tracks between Basel and Olten (Fig. 1).

The line will cross the main chain of the Jura Mountains with sedimentary rocks of Jurassic and Triassic age through the 12.6 km long Wisenberg-Tunnel. (Fig. 2).

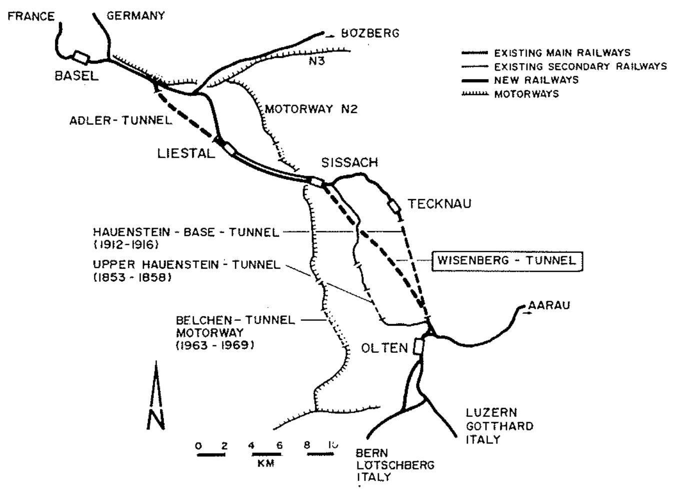

Fig. 1. Location map of existing Hauenstein-Tunnels and planned Wisenberg-Tunnel

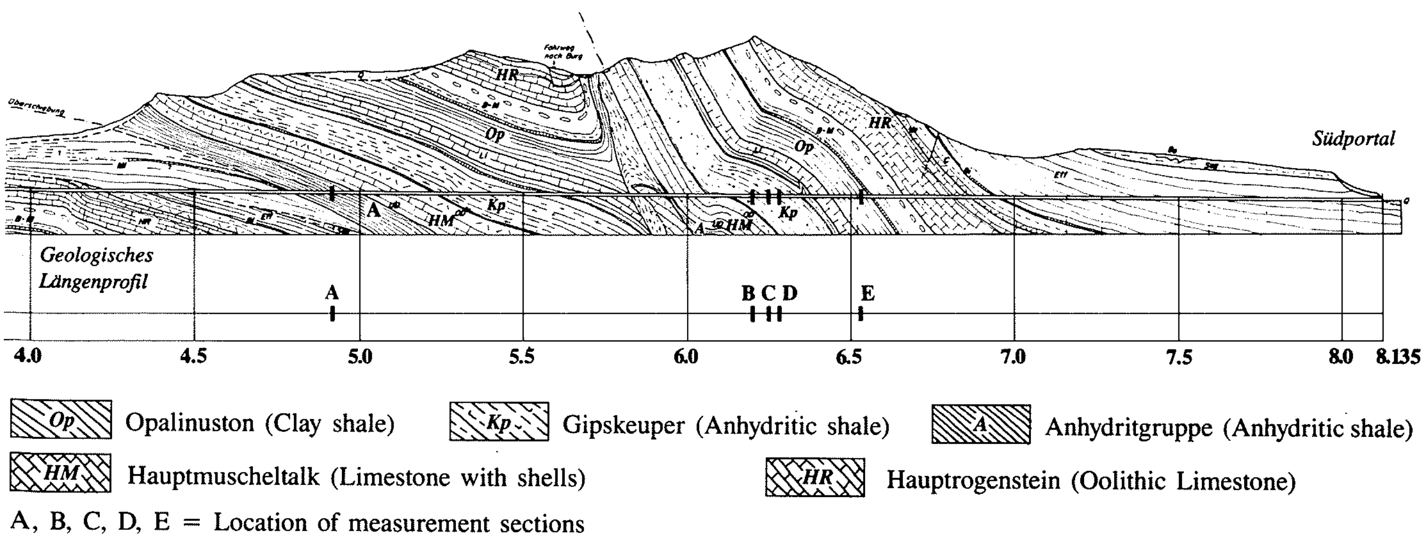

Fig. 2. Geological longitudinal section through the southern half of Hauenstein-Base-Tunnel.

The rocks are limestones, shales, sandy shales and anhydritic shales, i.e. anhydrites mixed with shales.

A large portion of the tunnel will cross rocks with swelling characteristics, particularly the anhydritic shales are well known for their extreme swelling behavior. The experience from the Belchen-Tunnel (Grob, 1972; Einstein, 1979) is well known, as is the experience from the two Hauenstein-Tunnels (Upper and Base-Tunnel). Measurements are scarce.

The only existing measurements of swelling pressures exist from the Belchen-Tunnel with contact pressure cells, most of which, however, had failed after only a few years operation. Swelling tests on samples give results which cannot be easily extrapolated to the full-scale tunnel.

The consultation of historic documents revealed that the Hauenstein-Base-Tunnel with its invert in granite stones provides an excellent in-situ long-term test on swelling pressure in anhydritic shales and other swelling rocks.

2 HISTORY OF THE HAUENSTEIN-TUNNEL

The railway from Basel to Olten was built as double-track line during the years 1853-1858. It included the 2.5 km long Upper-Hauenstein-Tunnel and also ramps on both sides with grades up to 26% (1 in 38).

The increase in traffic with the opening of the transalpine lines led to capacity problems. Thus a base tunnel was planned during the beginning of the twentieth century and built from 1912 to 1916. The new alignment leaves the old one at Sissach and enters the 8.135 km long Hauenstein-Base-Tunnel south of Tecknau to join Olten.

The tunnel has an asymmetric grade, climbs first for 2 km with 2% (1 in 500) and then falls 6.1 km with 7.8% (1 in 128) towards Olten. The tunnel was built from both portals (Wiesmann, 1917). Water from the aquifer in the limestone formation did not allow a falling heading, thus the tunnel was holed through asymmetrically. The tunnel was normally lined in the abutments with concrete. In zones with agressive water, the abutments consisted initially of limestone blocks from quarries in the area.

The crown was either lined with cement-stones or limestone blocks. The initial invert was made of concrete (see Fig. 3), only 624 m of the tunnel included an invert after holing through.

The swelling was such that during the years 1919 to 1923 important reconstruction work had to be undertaken. In some sections the invert had to be replaced and additional invert had to be placed (Tab. 1).

Table 1. Hauenstein-Base-Tunnel, Tunnel length with invert (after Zünd and Bischoff, 1986)

Invert	Built m	Remaining m	Rebuilt m	Total
Built during Tunnel Driving (1912-1916)	624			624 m (7.7%)
First Reconstruction (1919-23)				
Invert from 1912-1916		215	409	2294 m
New Invert	1670			(28.2%)
Second Reconstruction (1980-86)				
Invert from 1912-1916	—	64	151	
Invert from 1919-1923	—	2079	—	5368 m
New Invert 1980-1986	3074			(66.0%)

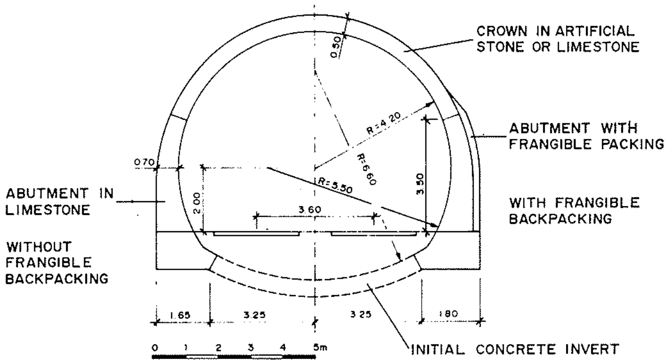

Fig. 3. Initial Tunnel cross-section with invert arch, dating from 1912-1916, in Anhydritic shale (km. 6.2)

Although the engineers of the first reconstruction work strongly recommended the placement of even further invert, the economic situation of that time did not permit any further work and the reconstruction was halted once the strongest swelling sections were reconstructed (Etterlin, 1986).

However, with time the trackbed in the sections with shaly types of rocks and without invert became instable. The rock deteriorated underneath the trackbed to mud due to the effect of dynamic loads and water. A second reconstruction was necessary from 1980 to 1986.

Additional inverts were placed and some of the inverts from the period 1912-1916 were replaced. Finally two thirds of the tunnel had an invert.

A section in the Gipskeuper (Anhydritic shale) exists where an original concrete invert from 1912-1916 had survived until approximately 1960 when heave was observed (Fig. 3). This section received a new invert in 1984 (Fig. 4).

The inverts from the first reconstruction period (Fig. 5) had performed satisfactorily, none of them had to be repaired during this second reconstruction. This invert made of granite blocks of 0.5 m thickness and an interior radius of 5 m has a substantial bearing capacity, since the unconfined compressive strength of granite usually exceeds 150 MPa.

In addition measurements of heave in the invert exist from the construction period (Fig. 6). In the anhydritic shales heave of the invert up to one meter was observed during a relatively short period of time.

Thus the idea grew that the Hauenstein-Base-Tunnel with its quite complex history may serve as in-situ long-term test section for the estimation of the swelling pressures.

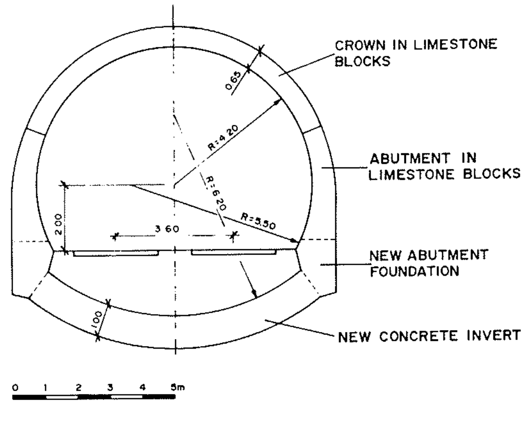

Fig. 4. Reconstructed concrete invert in Anhydritic shale during 1980-1986, Tunnel km 6.2 (after Rothpletz, Lienhard & Cie, 1986)

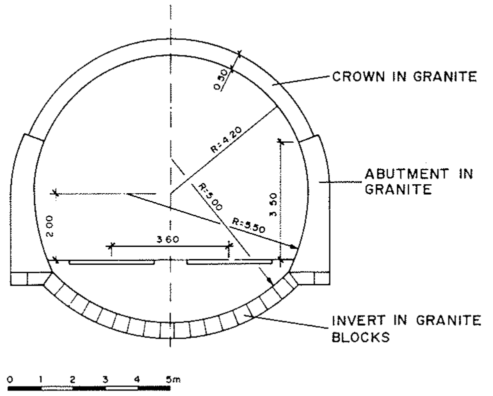

Fig. 5. Tunnel cross-section with complete reconstrution of liner and invert designed by Schlosser, 1919-1923 in anhydritic shale (km 6.23-6.32)

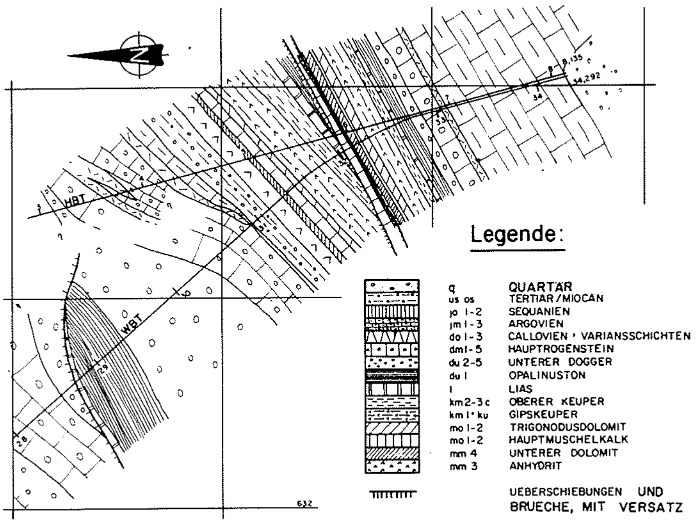

Fig. 7. Geologic horizontal section through southern half of Hauenstein-Tunnel (HBT) and Wisenberg-Tunnel (WBT)

3 STRESS MEASUREMENTS IN THE LINER

As mentioned above, the sections reconstructed with an invert arch from granitic cutstone performed satisfactorily. However the stress level within the tunnel liner was not known. With flatjack measurements in the liner of the Hauenstein-Base-Tunnel, the stresses can be measured and basing on the dimensions of the liners the swelling pressures can be estimated.

The highest swelling pressures are expected in shales that contain large percentages of anhydrite. In the case of the existing Hauenstein-Base-Tunnel and the planned Wisenberg-Tunnel these rocks will be encountered in the folded section of the Jura Mountains (Faltenjura) with overburden reaching 500 m. In this zone the two tunnels approach each other. A horizontal geologic section through the southern part of the two tunnels is shown in Fig. 7.

The measurement sections were located in the following geologic formations:

— Gipskeuper } Anhydritic snale
— Anhydritgruppe } i.e. a mixture of shale and anhydrite
— Opalinuston = Clay shale

For the measurements the operational requirements of the railway had to be taken in consideration. Only one track could be shut down for a limited period, only during the night and during a slack period of freight traffic. During the day both tracks had to be fully operational without speed restrictions.

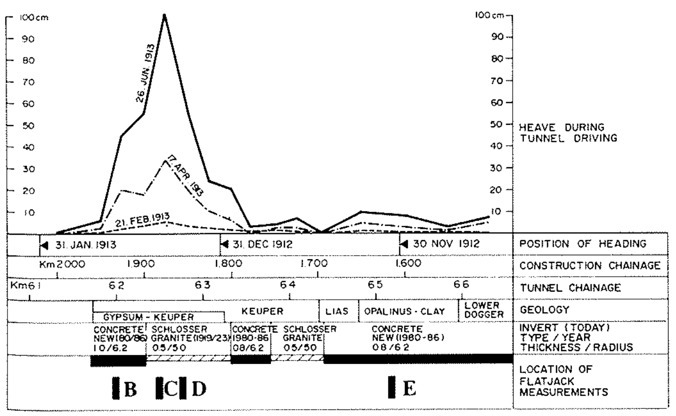

Fig. 6. Hauenstein-Tunnel: heave during tunnel construction, geology, invert type and measurement sections with flatjack (adapted from Wiesmann, 1917 and others)

This requirement together with limited space between tunnel walls and the tracks did not permit the execution of flatjack measurements in the invert arch proper.

Thus measurements could only be performed in the abutments and the crown, there they were clustered in order to obtain a better picture of the stress in a zone.

The determination of the state of stress by means of flatjacks is based on the stress release in a structure caused by a plane slot normal to the surface of the wall. If the test is carried out in a masonry structure, the slot is generally made in a mortar layer.

The stress release produces a partial closing of the slot which is measured by means of convergence measurements of couples of reference points situated in a simmetric position with regards to the slot. A thin flatjack is then cemented inside the slot and the pressure is gradually increased to cancel the previously measured convergency. In this condition, the pressure p inside the jack is proportional to the pre-existing state of stress in a dierection normal to the plane of the slot. The value obtained must be corrected by a coefficient which depends on the ratio between the flatjack surface and the slot surface and on the rigidity of the welded boundary of the jack. Fig. 8 shows the scheme of the principal phases of the measurement technique.

The value of the state of stress σ [MPa] in the testing point is given by:

$$\sigma = p \cdot Km \cdot Ka$$

where:

p = oil pressure [MPa];

Km = jack constant which must be determined by means of laboratory calibration [-];

Ka = A_J/A_S (ratio between the surface of the jack and the surface of the slot) [-].

Convergence measurements are carried out by means of a mechanical removable estensometer and a series of measurement bases having a length of 200 mm and realized by gluing invar steel plates of 5 mm in diameter on the surface of the wall.

Flatjacks can have dimensions of 120x120x8 mm up 600x300x12 mm according to different factors. In the case described in this paper the 400x200x12 mm size was chosen for most cases; the 150x150x8 mm size was utilised in particularly hard concrete to reduce slotting time.

The calibration of the flatjack technique has been carried out in ISMES, during the last eight years, using large size (at least 1.5x1.5x0.5 m) brick and stone masonry samples. The tests allowed the determination of the constant Km for different states of stress and flatjack sizes. More detailed information can be found in the literature (Barla and Rossi, 1983; Rossi, 1987).

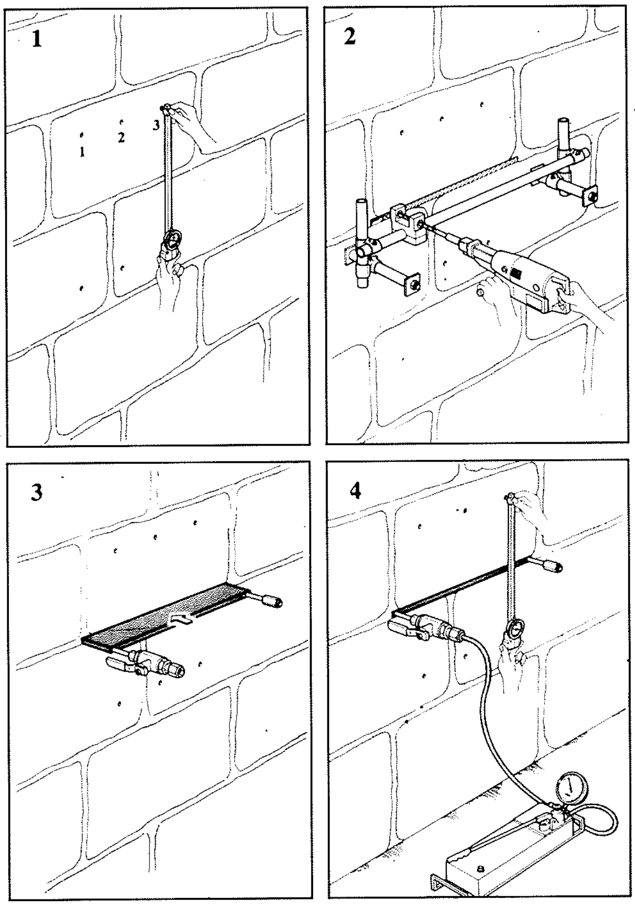

Fig. 8. Scheme of flatjack test

4 RESULTS AND INTERPRETATION

A total of 35 flatjack measurements in 5 sections was performed; 31 with the rectangular flatjack with a width 400 mm and a depth of 200 mm, and 4 with a square flatjack with 150 mm side length. The measurements were distributed over the height of abutment and into the crown.

The measured stresses in the liner varied from no stress to a maximum of 50 MPa in the Gipskeuper and the Anhydritgruppe (Anhydritic shales). In the Opalinuston (Clay shale) the measured stress in the liner ranged from 0.9 to 3.0 MPa.

The results of the stress measurements in the eastern side of the tunnel at tunnel chainage 6240 and 6270 are graphically presented in Fig. 9. The stresses vary considerably. This is due to the fact that the abutment liner had to be repaired and reconstructed several times. Thus a force redistribution must have taken place; the strong and stiff granite took the load when poorer zones, possibly made from limestone, failed, and had to be replaced by concrete. The concrete did not always take back the load which was transferred from the zone. Interestingly the scatter of stresses is much smaller in the crown, also the crown required much less repairs.

We attribute this to the following peculiarities of this zone: the zone initially included a compressible backpacking of the abutments (Fig. 3, right side). This backpacking was compressed. This in turn caused bending of the abutment which led to high edge pressures along the mortar joints, ultimately resulting in contact of the adjacent rock blocks followed by spalling of the liner. In some zones the surface of the liner had to be cut and replaced by a layer of reinforced shotcrete. For the interpretation of the stress measurements these effects must be considered. Other sections showed less scatter.

For the interpretation a simple model was used which is based on the following assumptions:
— no bending stresses exist in the lining;
— the thickness of the liner (blocks) corresponds to the drawings;
— the swelling pressure acts over the width of the invert.

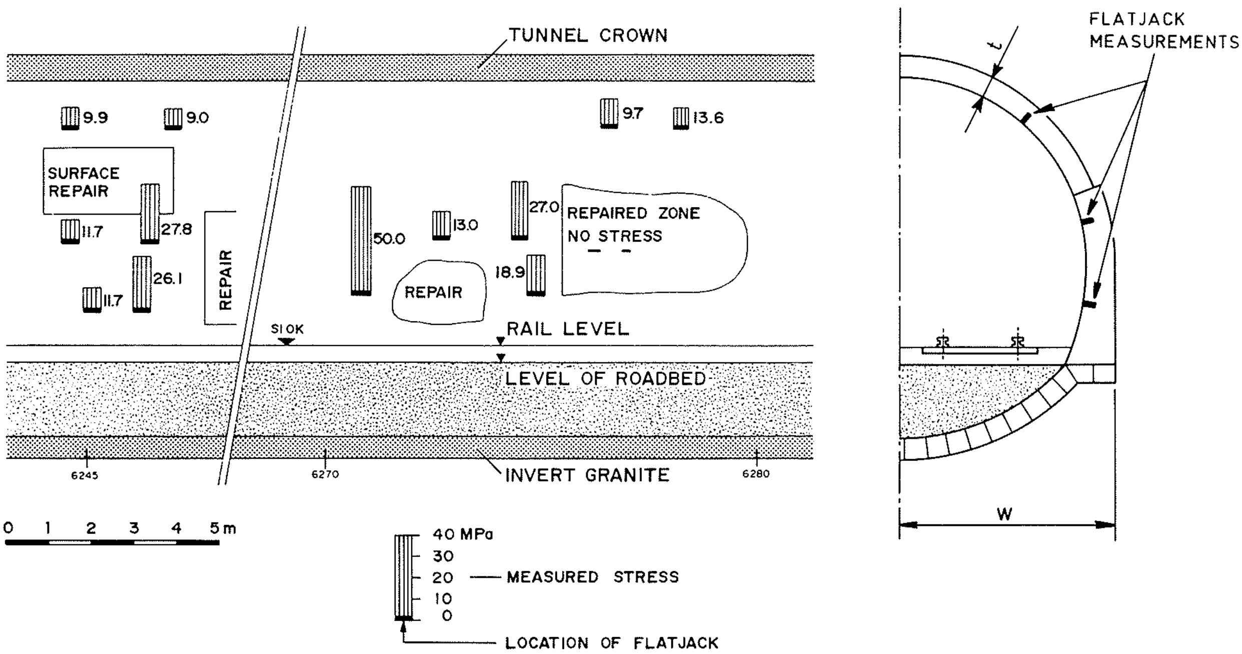

Fig. 9. Flatjack measurements in the eastern tunnel wall of the Haueustein-Base-Tunnel, km 6.24 and 6.27, location and measured stresses in the liner

The assumption of no or little bending stresses is supported by the fact that the liner consists of blocks with mortar between them that equalizes uneven stresses. The assumption was confirmed by the comparison of the computed thrust forces in the same section but different levels. The computed thrust forces did not vary in the same section for otherwise similar conditions. The assumption of the theoretical thickness of the liner particularly for block liner is conservative, as the contact area usually may be smaller.

The following computational assumptions were made (Fig. 9, right side).

$$T = p \cdot t$$

$$ps = \frac{T}{W} = \frac{p \cdot t}{W}$$

with:
p = measured liner stress;
t = thickness of liner;
T = liner thrust;
W = halfwidth of invert (in our case approximately 4.9 m);
ps = swelling pressure.

The computed liner thrusts and backcalculated swelling pressures are listed as mean values and standard deviations. (Tab. 2).

In the tunnel sections with Gipskeuper (Anhydritic shale) the liner thrust may exceed 10 MN/m = 1000 tonnes/meter. The scatter when considering all measurements is large and to a large extent due to the stress redistribution within the liner. By separating similar liner conditions and empirically adjusting localized stress conditions the scatter (standard deviation) could be reduced.

The backfigured mean swelling pressures in anhydritic shale for the long term behavior are approximately 1.7 MPa. In the section with a new concrete invert, the backfigured mean swelling stresses are 1.21 MPa and also the scatter expressed as a standard deviation is less. This difference may be attributed on one hand to the fact, that swelling pressure in the second case is not yet been fully redeveloped, on the other hand also on the better quality of the backfilling in the section with new invert.

It is interesting to note that in the section where 1984 a new invert was placed, the initial concrete invert and the tightly placed liner had performed satisfactorily until approximately 1960.

In contrast in the other sections where the invert and parts of the liner had to be reconstructed already 1919-23, a compressible backpacking was initially placed in the abutments. Also further repairs proved necessary during the second reconstruction (1980-86) in the abutments of the liner.

By comparing these two adjacent sections with similar geology but different lining techniques one must conclude that the compressible backpacking caused more problems, rather than solving them.

Also substantial heave up to one meter was observed during the tunnel driving in the zone with anhydritic shale (Fig. 6). The backcalculated stresses in Anhydritic shales are similar to those reported by Grob (1972-75) and Wichter (1989) for Anhydritic shales in shallower tunnels.

In the Opalinuston section substantially smaller liner thrust and swelling pressures are estimated: they are nearly one order of magnitude smaller.

These estimated swelling pressures are of similar size as those reported by Grob (1972) for the Belchen-Tunnel.

Tab 2. Compiled and interpreted results of flatjack measurements

Geology Liner conditions	Thrust in Liner MN/m		Backcalculated Swelling Pressure MPa	
	Mean	Standard-deviation	Mean	Standard-deviation
Gipskeuper (Anhydritic shale) All measurements considered	7.86	6.23	1.62	1.31
Gipskeuper Sections with invert and liner in granite blocks (Schlosser type liner from 1919-1923) without adjustment (Fig. 5)	10.83	6.46	2.27	1.34
Gipskeuper with empirical adjustments	7.87	2.08	1.66	0.43
Gipskeuper with new concrete invert, placed 1984 (Fig. 4)	5.93	0.80	1.21	0.16
Anhydritgruppe (Anhydritic shale) with Schlosser invert and original Limestone liner	8.26	2.90	1.69	0.69
Opalinuston (Clay Shale)	1.06	0.47	0.22	0.10

5 CONCLUSION AND SELECTION OF DESIGN SWELLING PARAMETERS

The flatjack measurements were the only way to estimate the level of stress in the liner of the Hauenstein-Base-Tunnel. The results give indications of the long-term swelling pressures and their development in time.

The design parameters selected for the Wisenberg-Tunnel are listed in Tab. 3.

Table 3. Design swelling pressures for Anhydritic shales

Design parameter	Swelling pressure MPa
Mean value (working stress level)	1.7
Structural capacity for drill and smooth blasting excavtion	3.0*
Structural capacity for excavation by full-face TBM	2.5*

*Including the load factor

From the measurements in the section with a new concrete invert placed 1984 one can conclude that at least 80% of the swelling pressure in anhydritic shale have developed over four years after placement. However, part of the difference in swelling pressures and measured liner thrusts might be attributed to other factors rathers than time.

Possibly the different construction and lining technique used in this section (km 6.2), where a strong liner was tightly placed against the rock, may have had an influence. Therefore one might conclude that the final swelling pressure has been reached to a larger extent four years after replacement of the invert.

From the experience in section km 6.23 to km 6.32 one has to conclude that a compressible pack in the abutments created problems with uneven loads, bending in the liner and necessitated more repairs and reconstruction than the tightly placed liner in section km. 6.2 in the Anhydritic shale of the Gipskeuper and in the similar Anhydritic shale of the Anhydritgruppe (km 4.9). The compressible packing in the abutments (km 6.23 to 6.32) obviously caused more problems rather than solving them.

The selected design pressures lie in the same range as the in-situ and laboratory measurements in the Belchen-Tunnel (Grob, 1972) located in the same formation but with a substantially smaller overburden. They are also comparable to the values reported by Wichter (1989) from shallow tunnels in southern Germany.

For the Opalinuston (Clay shale) the in situ design swell pressures are reported in Tab. 4.

Table 4. Design swelling pressures for Clay shale.

Design parameter	Swelling pressure MPa
Mean value (working stress level)	0.25
Structural capacity	0.6

These values are comparable to the in situ measurements in the Belchen-Tunnel (Grob, 1972). They are substantially below the swelling pressures measured in the laboratory in oedometer test, where the swelling pressures may reach 2 MPa.

The Wisenberg-Tunnel shall be excavated by a shielded full-face TBM with a diameter of approximately 12 m.

A liner in precast concrete segments will be assembled in the tail of the shield. At some distance from the tunnel face an interior cast-in-place liner is foreseen. For the Opalinuston (Clay shale) it appears at present that already the precast segments made from regular concrete with a thickness of 0.3 m can withstand the swelling pressures. One important prerequisite is a proper backfilling procedure of the tail-void.

The substantially higher swelling pressure in the Anhydritic shale will require the use of high-strength special concrete which is at the same time also resistant to chemical attack. At present these problems are being studied.

6 ACKNOWLEDGMENTS

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- Locher & Cie AG, Civil Engineers and Contractors, Zürich;

supplemented by
- smh Tunnelbau AG, Consulting Engineers, Rapperswil.

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