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In-situ swelling pressures in sulphate bearing rocks: Findings from field observations

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ABSTRACT: Swelling in sulphate bearing rock causes damage to structures, but particularly to tunnels with large heave or large swelling pressures. Swelling pressures determined in laboratory tests are substantially larger than in-situ swelling pressures that are still large. Swelling of sulphates is a process that involves flow of water and dissolution of anhydrite. Once solutions exceed the saturation limit, gypsum precipitates and crystallizes in the rock mass. This leads to heave and swelling pressures on structures but also on pressures within the rock mass around the underground opening. Discontinuities will close and the transport by dissolution and precipitation of gypsum will be reduced, until, with a sufficiently high pressure the swelling process ceases. The average swelling pressures are substantially lower than local crystallization pressures. Lining stresses have been measured with flat jacks or in other cases with measurement of strains in the concrete. Measurements with contacts stress cells generally indicate larger stresses.

1 INTRODUCTION

1.1 *Basic issue of swelling in sulfate rocks*

Swelling of sulfates in rock has caused many problems in tunnels (Steiner & Metzger, 1988; Wichter, 1991; Steiner, 1993; Alonso et al., 2013) although substantial research and studies have been carried out, many open questions and many contradictions and differences exist in understanding the process of sulfate swelling in-situ.

Laboratory tests on small scale samples indicate extremely high swelling pressures that appear not realistic. The transformation of sulphates and thus swelling passes over the fluid phase and is important in understanding the process, together with the rock structure. Anhydrite and other sulphates dissolve and when the solution is oversaturated gypsum precipitates. The formation of fissures around an underground opening due to excavation is a key factor.

With sufficient resistance of the tunnel liner, mainly in the invert, the rock mass around the tunnel is compressed, the fissures as flow path will be closed and the process of dissolution and precipitation will ultimately with sufficient counter stress cease.

This work is a compilation of different pieces of puzzles into a coherent picture of the main factors. The reasons for the differences between laboratory and field swelling pressures and behavior will be discussed.

1.2 *Discrepancy between laboratory swelling tests and in-situ swelling pressures*

Laboratory swelling tests are carried out in a swelling apparatus (Madsen, 1989) similar to the apparatus used for argillaceous rock (ISRM, 1989). The swelling pressures determined with laboratory tests reach often 6 MPa or more. In-situ pressures are substantially lower albeit still large as 1 MPa corresponds to 40m of rock overburden or 50m of soil.

2 EXPERIENCE FROM FIELD OBSERVATIONS AND LABORATORY TESTS

The authors have been involved in several cases, individually or together. The main features of the rail and road tunnels and the key issues are described for the Hauenstein Base tunnel (Steiner et al, 1989) and the Gotschna tunnel. Other case histories complete the picture.

2.1 *The Hauenstein base tunnel in Switzerland*

From 1987 to 1992 the Wisenberg tunnel through the Jura Mountains between Basel and Olten was planned that should double the existing Hauenstein base tunnel, built 1912–1916. The problems with swelling of argillaceous and sulphates bearing rock in this tunnel are well known (Steiner et al. 1993). To obtain design parameters for the new Wisenberg tunnel, investigations were carried out in the existing Hauenstein base tunnel.

Comprehensive mineralogical investigations and swell tests were carried out on seven specimens taken from four samples (Nüesch et al. 1995) that indicate a large heterogeneity of the samples within a single boring. The free heave (Table 1) tests indicate that more anhydrite was transformed into gypsum than for the swell test (Table 2), however not all anhydrite was transformed into gypsum. As these samples that underwent free swelling still have anhydrite left that may be transferred to gypsum, swelling may continue. In the swell pressure tests less anhydrite, from 0 to 25%, was transformed in gypsum.

Comparing these results with results from other tunnels (Lilla and Gotschna) one notes that the initial mineralogy from other sites appear similar, also swelling pressures are in the same range. One has to assume the swelling behavior in the laboratory is comparable.

The Hauenstein base tunnel has a lining made from masonry of limestone. The tunnel had to undergo a first repair from 1919 to 1923, when granite blocks were used, mostly for placing invert arches. A second major renovation proved necessary from 1980 to 1986 when invert arches cast from concrete were used. The Hauenstein base tunnel can be considered a large scale in-situ test to evaluate the long-term swelling pressure in the tunnel. Therefore, in five cross sections with swelling rock, tangential stresses in the lining were measured with flat jacks (Steiner et al. 1989) and thrust forces estimated and swelling pressures back figured (Table 3). Other results were found, as

Table 1. Mineralogical content in specimens from the Hauenstein base tunnel (Nüesch et al. 1995)

Sample/ Specimen	Orientation (Dip)	Clay	Sulphate	Anhydrite	Gypsum	Transformation into Gypsum	Swell Heave
	Degree	%	%	%	%	%	%
32/1	60	7	87	29	58	67	11.8
33/0	0	13	78	17	61	78	30.6
33/2	0	12	77	27	50	65	18.9
33/3	0	14	75	20	55	73	23.8
34/2	0	11	72	41	31	44	12.5
34/3	90	9	71	56	15	21	17.3
34/4	40	8	72	49	23	32	17.1
before test		5–20		30–75	5–20		

Table 2. Mineralogical tests on specimens that underwent swell pressure tests (from Nüesch et al., 1995)

Sample	Carbonate	Clay	Quartz + Accessories	Sulphate	Anhydrit	Gyp-sum	Transformation into Gypsum	Swell pressure
No.	%	%	%	%	%	%	%	MPa
32/3	3	5	8	84	63	21	25	4.7
34/2	4	15	16	65	57	8	12	1.7
34/3	4	18	15	63	56	7	11	4.5
34/4	4	11	16	69	62	7	10	3.3

Table 3. In-situ swelling pressures in the zone with gypsum formations in the Hauenstein base tunnel (after Steiner et al. 1989)

Geologic conditions Liner types in crown and invert	Mean thrust back calculated from flat jack in liner	Swelling pressure back calculated acting on invert (characteristic)
	(MN/m)	MPa
Gipskeuper (Gypsum keuper) with original replaced during first repair (1919–23) with granite blocks in invert and top arch	8 ± 2.0	1.66 ± 0.43
Gypsum keuper with original top heading in limestone masonry from 1912–1916 and with new concrete invert arch in 1984	6 ± 0.8	1.21 ± 0.16
Anhydrite group (Anhydritgruppe) with invert arch from 1919–1923 in granite blocks and crown in limestone masonry	8.3 ± 2.9	1.69 ± 0.89

in one section during the initial construction from 1912–1916 compressible backpacking had been used, which did not perform satisfactorily, and the liner had to be replaced 1919–1923 in crown and invert with a liner made from granite blocks that has performed satisfactorily over decades. These average in-situ pressures are only a fraction of the swelling pressure in the laboratory.

2.2 *Wagenburg tunnel, Stuttgart, Germany*

The Wagenburg tunnel, a road tunnel with a parallel safety tunnel in the City of Stuttgart, built from 1941 and opened in 1957 (Müller, 1978; Henke et al, 1979) has been well instrumented and has undergone major repairs (Paul & Wichter, 1996). Radial stresses measured with contact stress cells range from 0.5 to 6 MPa. Tangential stresses vary from 1 to 16 MPa, thus swelling pressures back figured from thrust loads are only a fraction of the radial contact stresses.

2.3 *Weinsberg tunnel near Heilbronn, Germany*

The 891 m long Weinsberg tunnel built from 1859 to 1862 near Heilbronn crosses a 90 m high hill of the Gipskeuper formation. Near both portals the tunnel crosses over 200m, leached Gipskeuper, i.e. weathered clay and in the center about 500m of non-leached claystone with sulphates (anhydrite and gypsum). The swelling process led to heave of the invert and deformation of the lining (Dauwe et al. 2007). After construction, during the 19th century, large heave was observed. One has to assume that the material of the heave below the tracks was shaved-off to keep tracks on level. From 1920 to 1953 heave became less important, however lateral movements occurred and lateral (horizontal) convergence of 217 mm was observed. During the years 1955/56 in sections with more than 100 mm convergence the masonry was replaced by sidewalls of 0.9 m concrete with 0.4 m backfill to the rock and a 0.5 m thick invert arch with a radius of about 10 m. Adjacent sections were repaired in 1974 with a circular lining of reinforced concrete. Around 2002 that line had to be electrified for a transit system requiring the enlargement of the tunnel. For the design, stresses in the lining were measured with flat jacks. The tangential stresses in the invert were around 16 MPa and in the crown 24 MPa, and only a few MPa in the side walls. Back calculated swelling stresses in vertical direction are 1 MPa and in horizontal (lateral) direction 1.5 MPa.

2.4 *Freudenstein tunnel, German railroads, high-speed line Stuttgart – Mannheim*

The construction of the Freudenstein tunnel on the high-speed railway line in the 1990s was preceded by the construction of a trial tunnel, scaled 1:2 with 15 sections of different resisting and compressible linings (Fecker,1996) with measurement of radial stresses, stresses in the lining in invert and crown. The contact stresses in the invert were 1 to 2 MPa. The measured tangential stresses in the lining indicate that the average radial stresses are lower. Laboratory swelling tests (Kirschke, 1996) reached 6 MPa, also after unloading the samples they increased to the same level rapidly again. In the rail tunnel a compressible layer was placed in the invert, that should yield once the swelling pressure exceeds 1 MPa. Measurements of the contact stresses (Kirschke, 2010) indicate that the swelling stresses increased nearly linearly during the first two years to a pressure of 0.6 MPa. After five years the contact stresses reached 0.7 MPa and have remained constant for eight years. The compressible stratum was thus not activated.

2.5 *Lilla tunnel near Tarragona, Spain*

The Lilla tunnel (Alonso et al. 2013) on the Spanish high-speed railway line from Barcelona to Madrid, near Tarragona, suffered substantial heave of the flat invert after opening the tunnel. Extensive investigations were carried out including laboratory and field tests. The laboratory swelling tests indicated swelling pressure up to 6 MPa. After these studies, a strongly reinforced invert was placed, and the crown was reinforced with an additional 0.3 m thick layer of reinforced concrete. Instrumentation includes radial contact stress cells, six each section in the invert and strain gauges on the reinforcement in the concrete. The radial stress measurements with the contact cells are rather erratic. In one section, five of the six cells indicate about 1.5 to 2.3 MPa and the sixth cell 5.3 MPa. In a second section 5 cells indicate 0.5 MPa and one cell 5.64 MPa and in the third section from zero to 0.26 MPa and a peak of 6.7 MPa. The measurements of strains on the reinforcement are given as steel stresses; to obtain the concrete stresses the steel stresses have to be divided by the ratio of steel to concrete modulus. Radial stresses back calculated by the authors indicate a range from 0.3 to 0.9 MPa.

3 OBSERVATIONS IN THE GOTSCHNATUNNEL

3.1 *Location and geologic setting*

The Gotschna tunnel is a 4.207 km long dual lane road tunnel that forms the bypass road of the resort Klosters, Canton Grison, in the Swiss Alps. Road construction started in 1995 and tunnel blasting initiated October 3, 1997, the tunnel was holed through on December 10, 2001 and opened to service in December 2005. The longitudinal section (Figure 1) shows complex geological structure that lead to different possibilities for swelling potential around the excavation profile (Figure 2).

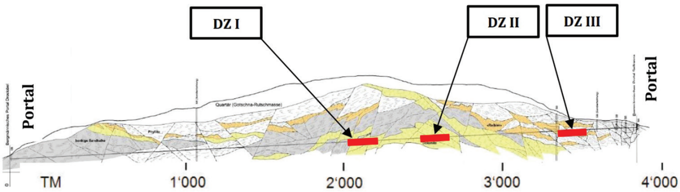

Figure 1. Geologic conditions along Gotschnatunnel (Krähenbühl, 2014) with swelling zone 1 (DZ I), zone 2 (DZ II) with anchors in the invert and zone 3 (DZ3) with invert.

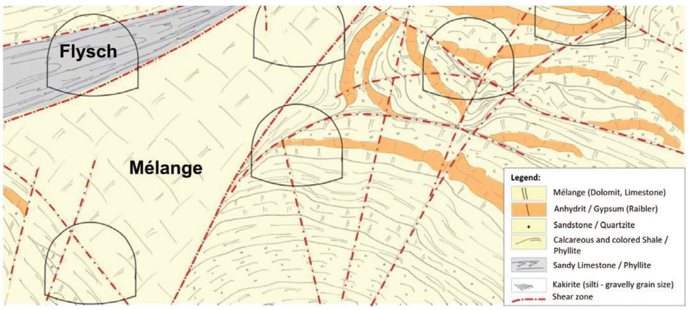

Figure 2. The complex geological structure at Gotschna tunnel showing different geotechnical situations for different tunnel sections (Krähenbühl, 2014)

3.2 *Laboratory test*

To investigate the swelling potential in the critical zones, swelling tests had been executed in the laboratory of the Institute for Geotechnics (IGT) of ETH in Zurich. Swelling pressure in drilling core samples with maxima of 2 to 4MPa where measured. Rock strength (IGT, 2015) was determined with triaxial compression tests (Table 5).

3.3 *Observed heave*

In three sections of the Gotschna tunnel heave occurred during construction, at least between excavation and completion of the inner lining and placement of the roadway. In deformation zone DZ I a flat invert arch, weakened by channels for tunnel utilities was placed (Figure 3, left). The rate of heave at the begin of measurements in 2005 was about 15 mm/a and the support pressure of the invert was estimated as 200 kPa, corresponding to the weight of concrete below the roadway and the structural capacity of the invert, limited by cable ducts. The residual bearing capacity after cracking of the invert can be estimated between 40 and 80 kPa.

In deformation zone DZII initially the horizontal floor of the tunnel heaved with a rate of about 20mm/a and once more of the invert was excavated, a stratum of newly formed gypsum was found in the invert (Gall et al. 2014). As the date of opening approached, a rapid solution had to be sought and a slab anchored with tie-backs (Figure 3, right) was placed. The tiebacks with 1500 kN yield capacity and a limiting load of 1274 kN were placed in 1.5 by 3.5 m grid, resulting in an average support pressure of 325 kPa.

In swelling zone DZ III an invert slab (Figure 3, center) designed for a load of 500 kPa over the width of the tunnel with an emergency bay. Only small heave of a few millimeters was observed in this area. This heave observed is interpreted as the displacement to activate the reaction of the crown arch.

The rates of heave of the inverts, the applied support pressure and the overburden for the three zones with deformation (DZ I, DZ II, DZ III) are summarized in Table 4 and presented in Figure 4. With a resisting stress of 0.5 MPa the heave could be stopped, with smaller resistance heave continued. In zone DZ II a newly formed stratum of gypsum had been found in the invert prior to installing the tiebacks. Local pressure peaks cannot be excluded.

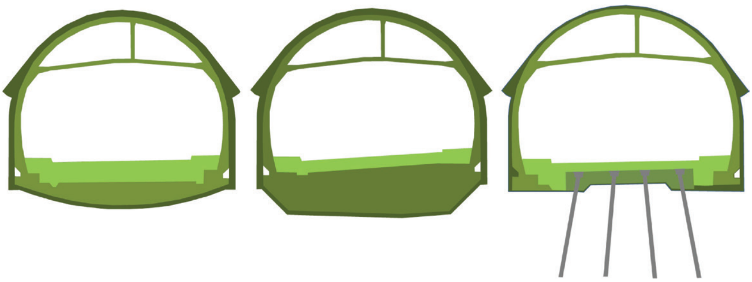

Figure 3. Cross sections in sections with swelling heave: left: Zone DZ I; center: DZ III with invert slab, right: DZ II with tiebacks

Table 4. Observed rates of heave in the deformation zones of Gotschna tunnel.

Deformation zone with swelling phenomena		Overburden	Rate of heave of invert	Invert support pressure	Invert type
	chainage (m)	(m)	(mm/a)	(kPa)	
DZ 1	2030–2170	380	rupture	200 to 40–80	Flat invert arch
			12–15		Flat invert arch sheared
DZ 2	2514–2549	380	20	20	Prior to the placement of tiebacks
			4	325	With tiebacks in invert
DZ 3	3300–3380	200	0.5–1	500	Invert slab

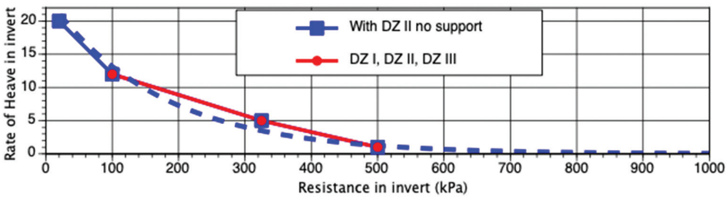

Figure 4. Rate of deformation vs. support pressure of invert for Gotschna tunnel with exponential correlation (—) through four points and linear correlation (—) through 3 points.

3.4 *Measurement of stresses in the side walls of the liner in zone DZ III*

In section DZ3 during a cleaning interval of two nights in August 2014, 12 measurements with flat jacks were carried out. These measurements indicate an irregular distribution of swelling pressures in the invert, varying from 0 to 0.48 MPa.

3.5 *Swelling heave as consequence of a clogged drainage system*

During 2017 in inverts uphill from DZ III, local swelling phenomena were noticed in an area where previously no heave had been observed. The heaves estimated on about 5–8 cm where located in two zones at km 3.400–3.420 and 3.610–3.620 by driving through and removed during service works without any measurement at the end of 2017. The swelling heave occurred in sections without invert arch, the second one in a section directly adjacent to a section with invert arch. In the adjacent tunnel sector with invert arch an extraordinary drainage pipe was situated in the tunnel profile, allocated outside of the invert arch, practically in the axis of the tunnel over a length of 100 m. The inspection of the tunnel revealed that this special drainage system became completely clogged at about km 3.560 and led to the situation that the drainage system was converted into an irrigation system upstream and at least in the vicinity of clogging point. The drainage system was cleaned in the area and became functional again. With flat jacks the stresses loaded in the lining from the swelling processes were monitored in the tunnel-sector between km 3.560 and 3.610. The back estimated average swelling stresses per measurement were mostly around zero and reached a maximum near the clogging point of the order of 0.4 MPa.

The observation that swelling occurred after clogging the drainage confirms that gypsum swelling is related to transport of sulphates by water. The water velocity has to be sufficiently slow such that sufficient sulphates can be dissolved in the seepage and the concentration of sulphates exceeds the saturation limit such that gypsum falls-out and crystallizes and swelling occurs. This confirms the dissolution-precipitation process as described by James (1992) and also observed in tunnel Markusberg (Steiner et al, 2015).

3.6 *Excavation damaged zone (EDZ) or loosening zone around Gotschna tunnel*

Analyses have been carried out to estimate the size of the zone with excavation damage, as proposed by Steiner et al. (2010, 2011), Kaiser et al. (2010), and Amann et al. (2010). For Gotschna triaxial tests had been carried out on samples from two deformation zones: DZI and DZII.

For DZ I and DZ II the overburden of the tunnel is 380 m or the vertical stress σ_v =10.3 MPa for DZ III the overburden is 200m and the stress σ_v =5.4 MPa.

Table 5. Rock strength parameters for estimating EDZ (Excavation damaged zone) after IGT (2015)

Rock type	Friction angle	Cohesion	UCS	N_σ	Over 200m	burden 380m
	σ' (°)	c'	$\sigma_c = 2c$ ¶ (N_Φ)		EDZ (m)	EDZ (m)
Kakirit	27	8 kPa	26.11 kPa	2.66		
Bündnerschiefer massive	45	4 MPa	19.3 MPa	5.82	1 to 5	2 to 8
Bündnerschiefer sheared	28	1 MPa	3.3 MPa	2.76		
Dolomite & Gypsum	45	2 MPa	9.6 MPa	5.82	2 to 8	4 to 12

Table 6. Mineralogy for two samples at Gotschna compared to the range for Hauenstein (HBT) tunnel

Sample	Quartz %	Dolomite %	Magnesite %	Anhydrite %	Gypsum %	Clay %
HBT before tests	5–20	0–20	5–25	30–75	5–20	5–20
Part 1	6.8	18.4	18.0	36.9	4.2	7.0
Part 5	9.3	7.8	16.8	42.3	7.9	7.3

The stress level depends on the horizontal stress ratio that is estimated K > 1.0 to 2.0. The stress levels estimated are SL =1.0 to 2.0. With the recommendations by Kaiser (2016) a thickness 5 to 10 m was estimated for the dolomite/gypsum formation (Table 5). The formation of gypsum crystals has been observed by mineralogical tests (Krähenbühl, 2013) on samples from Gotschna tunnel. The formation of new gypsum crytals has also been observed in an adit of the Belchen tunnel by Amann et al. (2013).

3.7 *Mineralogy of the rock*

At Gotschna tunnel mineralogy by weight was determined on two specimens (IGT, 2015) compared to the mineralogy of the Hauenstein Base tunnel (Table 6), and found to be comparable.

4 CONCLUSIONS

The evaluation of several case histories and the proper experience indicate that the processes that lead to in-situ swelling pressures are not properly represented by the laboratory swelling pressure tests. The differences in swelling pressure from laboratory tests up to 6MPa an more and field measurements, in case of the Gotschna tunnel less than one MPa are caused by different conditions and effects as listed in Table 7. In-situ swelling stresses of 1 to 2 MPa have been postulated by Steiner et al. (1989), Steiner (1993, 2007) and Kovari and Vogelhuber (2014).

In practice there appear to be no reliable design criteria in practice for swelling rocks in sulphates. The present research has shown that in-situ the swelling pressures acting on a structure (tunnel lining) are on the average only a fraction of the laboratory swelling pressures. The uncertainty regarding this topic is documented by several cases that failed or underwent several stages of repair and suffered structural damage. The interaction of ground with the structure has to be considered. To improve this unsatisfactory situation further systematic analysis of case histories is necessary and has to be documented.

Table 7. Comparison of conditions for laboratory tests with in-situ conditions for the Gotschna tunnel

Laboratory swell test	Swelling in tunnel
Swelling process – no transportation of gypsum	Swelling process – solution, transport of gypsum, oversaturation, precipitation and crystallization. Flow rate and rock type govern the dissolution and transport of sulphate (James, 1992)
Sample compact and mostly undisturbed	Fractured rock formed by stress changes around tunnel bottom in contact with water and on-site traffic
Sample selected from drilling core with high swelling potential	Unselected rock –rock with various content of anhydrite leads to reduced dissolution potential of anhydrite
Optimum swelling conditions by saturation and alimentation with nearly saturated water	The swelling pressure in the zone around a tunnel does not act only on the lining, there is a reaction into the fissured zone (EDZ). Fissures will close, flow reduced until swelling ceases.
Full swelling pressure can be measured as saturated sulphate water is alimented continuously	3-dimensional situation. As farther the swelling process from excavation the more distinctive is the effect and the losses swelling pressure between source and lining.
Direction of swelling pressure is constrained by apparatus.	Maximum of swelling pressure only in selected positions around the tunnel excavation, where anhydrite layer occurs tangential to excavation profile.
Swelling measurement starting from begin of process	Long history starting by excavation and begin of measurement only by realization of the inner lining.

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