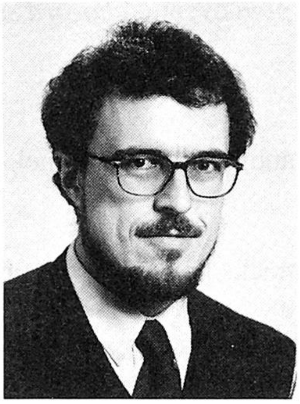
Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 78 (1998)

Artikel: Cut-and-cover tunnel a karst environment
Autor: Vogel, Thomas / Kalak, Joseph
DOI: https://doi.org/10.5169/seals-59051

Cut-and-cover Tunnel in a Karst Environment

Thomas VOGEL
Professor
IBK, ETH Zurich
Zurich CH



Thomas Vogel, born in 1955, graduated from the Swiss Federal Institute of Technology ETH Zurich in 1980. He gained eleven years of experience in bridge and structural design in consulting firms in Chur and Zurich. In 1992 he joined ETH as a professor of Structural Engineering.

Joseph KALAK
Civil Engineer
Aegerter & Bosshardt AG
Basle CH

Joseph Kalak, born in 1944, graduated from the Swiss Federal Institute of Technology EPF Lausanne in 1970. He has been involved in solving special structural engineering problems and is at present a senior engineer at a well known consulting firm in Basle.

Summary

A tunnel of the Swiss railway network traverses in its cut-and-cover section over a Triassic limestone stratum which exhibits the karst phenomena that causes dolines and subsidences. The tunnel is to bridge the dolines as a stiff tube and deform with the long term subsidences. The paper describes the hazard scenarios that were considered, the derived design concepts, the chosen solution, the calculations made to ensure structural capacity and serviceability and how these considerations influenced the final design.

1 The project

The Adler Tunnel is a paramount part of the Swiss railway project "Railway 2000" to provide travel times under 60 minutes between the major towns of Switzerland. The two-track tunnel constitutes a more direct route than the existing railway line that leads from Basle southbound to the central parts of Switzerland and through the Alps. The tunnel consists of a drilled core section of 4.3 km in length and cut-and-cover sections at each end. A general overview of the project with contributions covering design and construction is given in ref. [1].

The western cut-and-cover section which is 750 m in length traverses a gravel bed that was deposited by the Rhine river following the last glacial period. Beneath the gravel lies a Triassic limestone stratum, known as Muschelkalk (shell limestone), which exhibits the karst phenomena. This causes dolines (sink holes) and extensive subsidences to occur. Within the last 40 years about 20 new dolines with diameters between 5 and 10 m have arisen.

According to estimations summarised in a geotechnical report dolines up to 22 m in diameter and 5 m in depth can form within a few hours and subsidences with a diameter of 100 m can develop with a settlement speed of 10 mm per year.

2 Conceptual considerations

According to the Swiss Actions Code [2] the intended use of a structure and its performance requirements are defined by the client and the designer in the *utilisation plan*. On this base the design engineer identifies and evaluates critical situations both during construction and throughout the projected service life. In the *security plan* eventual *hazard scenarios* are listed with the measures specified to ensure safety.

2.1 The utilisation plan

The Swiss Federal Railways as client and operating authority of the tunnel specified the following service requirements:

- a planned service life of at least 150 years
- partial replacements requiring lengthy closure of tracks not before the first 100 years
- speed limits of 160 km/h at present, 200 km/h in the future
- sophisticated restrictions of availability for maintenance taking into account time of day (day or night), number of tracks (single track or total closure) and intervals (daily, weekly, monthly, yearly closure)
- acceptance of local moisture, no acceptance of dripping water that could form ice
- minimum radius of curved deformation due to subsidences: 5000 m
- maximum subsidence: 250 mm
- keeping within the indicative values of deflections for railway bridges of [2] for the local spanning of dolines.

The slowly occurring subsidences can be taken into account by designing the cross-section with the necessary additional space which allows a realignment of the tracks. Suddenly occurring dolines, however, should be bridged by the tunnel acting as a stiff tube.

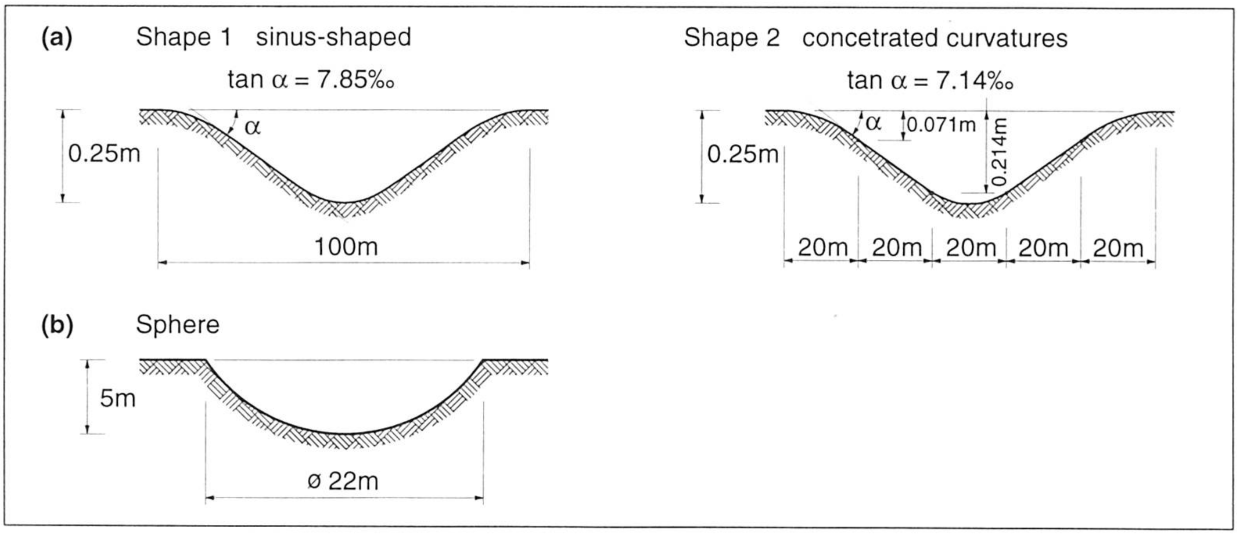


Fig. 1 Standard subsidences shape 1 and 2 (a) and standard doline (b)

2.2 Hazard scenarios and safety plan

In addition to the usual loads on a cut-and-cover railway tunnel the following situations were taken into account:

1. A slowly developing extensive subsidence with an average speed of 10 mm per year and a maximum of 200 mm per year. These are caused by the collapse of caverns in great depth as a consequence of a dissolving of the limestone or a changed stress state.
2. A superficial collapse of a karst cavern due to the same reasons resulting in a funnel that

reaches the surface or the tunnel level.

In both cases a reliable detection and subsequent refill are not possible within reasonable economical limits. The above cases were therefore defined numerically in order to incorporate them as hazard scenarios into the structural analysis. Figure 1 shows the two types of standardised subsidences (sinus shaped and with concentrated curvature) and the standardised spherical doline. The doline may occur at or eccentric to the tunnel axis within hours. Thus a very high accuracy in the design process is not justified as these assumptions and their numerical values rely mainly on sound judgement and experience.

3 Structural analysis

3.1 The cross-section

To bridge local dolines at any position leads to a monolithic tube over the whole endangered length. Any joints or hinges would reduce the bridging effect. The adjacent drilled section and the large overburden of up to 5 meters led to a circular cross-section with an external diameter of about 12.5 m. The crown vault has a depth of 400 mm and the invert vault a depth of 600 mm. An additional vault of 280 mm depth is located over the invert and forms the service channel. The remaining invert area is filled with gravel (fig. 2(a)).

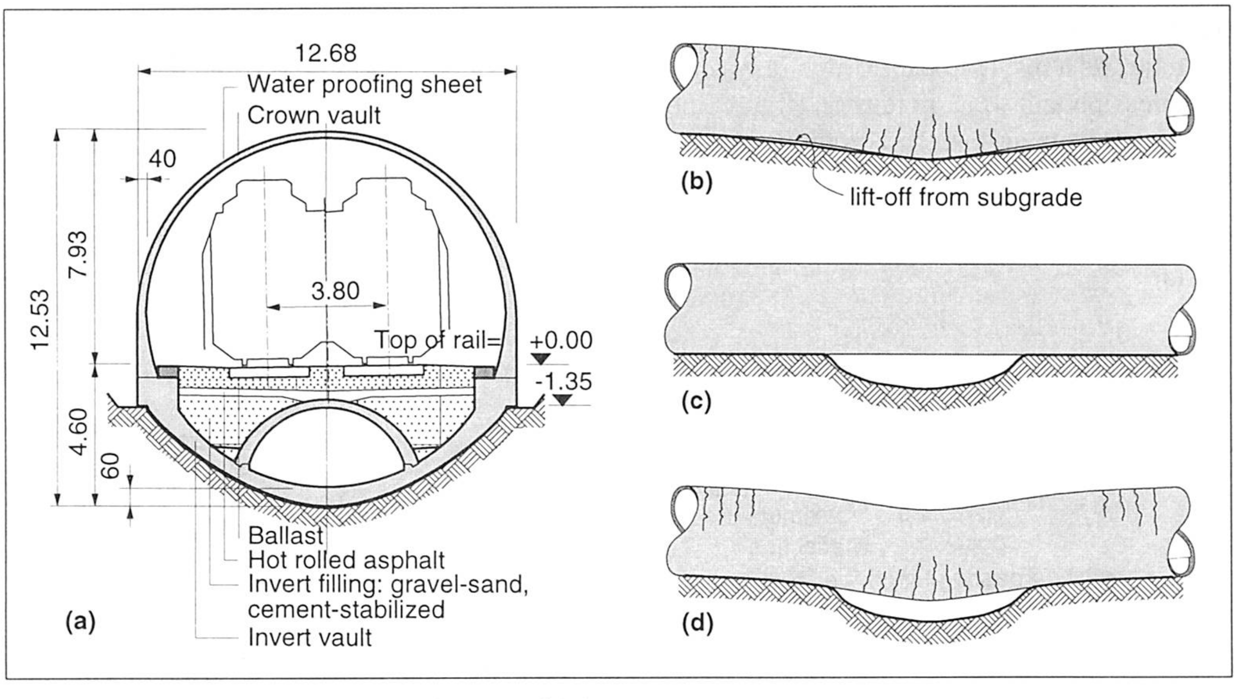


Fig. 2 Typical cross-section of the tunnel (a), fundamental cases of structural performance: extensive long term subsidence (b), doline (c) and subsidence with doline (d)

3.2 Structural performance and the statical model

The model used in the structural analysis is an elastically supported beam with self-weight, imposed and traffic loads. Since the tube is unable to span a subsidence of 100 m, it is forced to deform with the settlement. If the tube was fixed to the ground, the shape of the subsidence would equal the deflection curve of the tube and the second derivation would equal the curvature. The elastic support allows a certain balance of deflections due to the bending stiffness of the tube. As the load is limited a local lift of the tunnel from the ground has to be taken into account. Nevertheless the situation can be treated as a serviceability problem; the deformations result in cracks and at most in the plastic hinges, but failure does not occur (fig. 2(b)).

If a doline develops, however, the tube is to span the funnel and act as a tunnel bridge. That means a safety problem arises and the ultimate limit state has to be considered. A doline eccentric to the tunnel axis leads to asymmetric earth pressure which can be taken up by a tube of relatively large thickness by transversal bending (fig. 2(c)).

It is very likely that a collapse of a deep cavern and the subsequent subsidence will lead to the failure of a cavern at a minor depth causing a doline to form simultaneously. The relevant and most plausible case to be considered is the superposition of a standard subsidence and a standard doline.

The important factor at the ultimate limit state is that the available ductility is reduced at the relevant cross-section as it is already drawn upon to deform with the subsidence. Therefore the behaviour of the tube spanning the doline is less ductile or more brittle (fig. 2(d)).

3.3 Implementation of the statical system

The preceding description shows that linear-elastic models are unsuitable to the task. Local problems such as transversal bending due to asymmetric earth pressure and concentrated reactions at the edge of the doline, however, can be separated from the global bearing function of the tube. As a result a beam structure was modelled, supported by springs that are stiff in compression and weak in tension (figure 3(a)). The spring stiffness in compression can be calculated from an assumed modulus of subgrade reaction *k* and the affiliated reaction area (figure 3(b)).

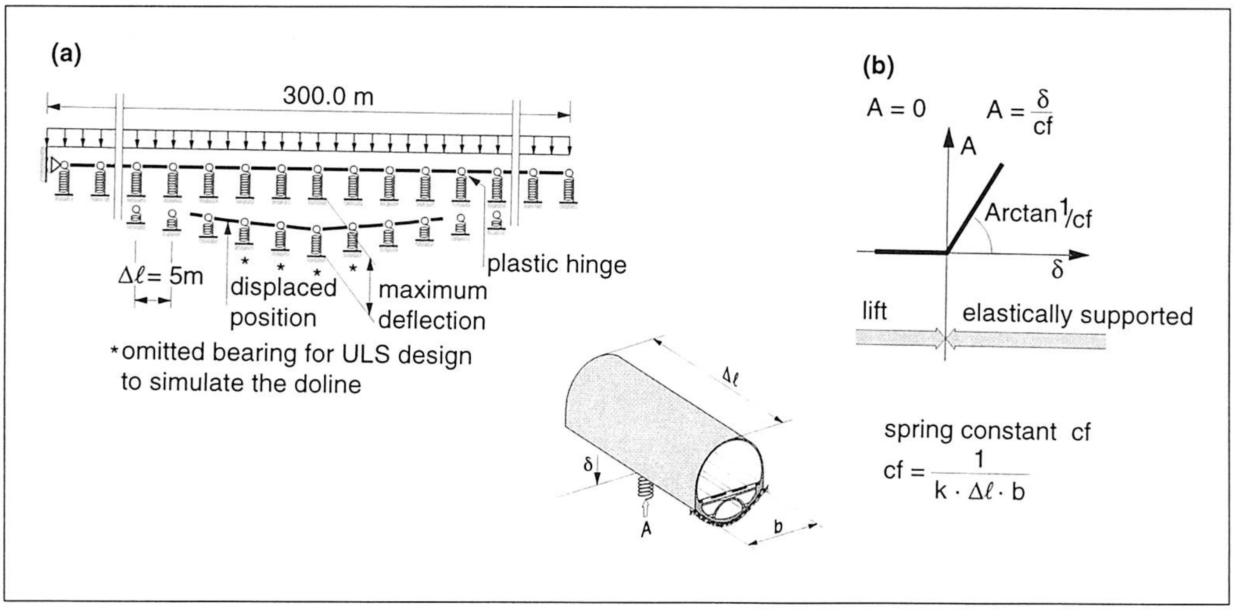


Fig. 3 Modelling as a series of bars (a) and characteristics of springs (b)

A rough approximation shows that the cross-section is under-reinforced even with two layers of Ø 30 mm @ 150 mm in the upper and the lower vaults respectively. Thus for pure bending and limited steel strains the concrete does not reach its ultimate compressive strain although the moment-curvature diagram, calculated with the material properties given in [3] and a failure steel strain of 10‰, shows a typically ductile behaviour (figure 4(a)).

4 Numerical calculations

The calculations were made with the program STATIK-N [4], a postprocessor to STATIK-2, a program for frame structures widely used in Switzerland. To meet the requirements of STATIK-N the following simplifications were made:

- The elastic domain of the moment-curvature diagram up to the yielding moment M_y (containing the uncracked and the cracked stage) is linearized and the gained bending stiffness attributed to the cross-section. By this simplification creep is neglected i.e. the difference between the long-time and short-time value of Young's modulus is not taken into consideration.
- The plastic domain of the moment-curvature diagram between the yielding moment M_y and the bending resistance M_R is concentrated at the nodes as yielding with hardening (fig. 4 (a)).

For the serviceability limit state evidence is required on the crack width resulting from the calculated curvatures . This is done using equation (1), where w_m denotes the average crack width, ε_{sb} and ε_{sc} the average strains calculated from the bending moment and the affiliated curvatures as well as the shrinkage (assumed to be 0.2‰) respectively. s_{rm} denotes the average crack distance (assumed to be the spacing of the lateral reinforcement @=150 mm). The factor 0.9 accounts for the tension stiffening, i. e. the participation of the concrete between the cracks.

$$w_m = 0.9(\varepsilon_{sb} + \varepsilon_{sc})\ s_{rm} \tag{1}$$

4.1 Executed calculation runs

The calculation procedure went as follows:

- Serviceability limit state check with both forms of the standard subsidence; with loads, but without load factors. The settlement was increased step by step up to the maximum value of 250 mm. The formation of plastic hinges and of local gaps between tunnel and subgrade could be observed.
- Ultimate strength limit state check with a doline; introduced by omitting 4 springs and loads increased by applying the relevant load factors. Subsequently the formation of the standard subsidence was imposed and increased up to the same maximum value as above. To meet the design level the moments derived from the moment-curvature diagram were reduced by dividing by the resistance factor $\gamma_R = 1.2$.

The first calculation run was executed with two moduli of subgrade reaction ($k = 10^4$ kN/m^3 and $k = 10^5$ kN/m^3) to evaluate their influence. The amount of reinforcement corresponded to two layers of Ø 30 mm @ 150 mm in both the crown and invert vaults and Ø 18 mm @ 150 mm in the lateral regions. For the second calculation run all rebars were reduced to Ø 18 mm and k fixed at 5 x 10^4 kN/m^3. This run served to establish an intervention point for the deflection that leads to average crack widths of 0.4 mm. A crack width of 0.4 mm was regarded as the threshold value for corrosion to begin. The third calculation run was identical to the second but with a main reinforcement of Ø 26 mm.

5 Results

In the serviceability limit state calculation the modulus of the subgrade reaction governs the appearance of plastic hinges. The stiffer the subgrade the sooner the hinges appear. The length of the plastic zones depends mainly on the presumed shape of the subsidence and amounts to between 10 and 20 m. Adequate flexural strength to span the standard doline alone is guaranteed in any case. When the standard doline and standard subsidence are applied combined the plastic hinges appear at an earlier stage. A reduced bending capacity M_R/γ_R is reached when $k = 10^5$ kN/m^3 at a maximum settlement of about 190 mm. As expected the intervention point is only minorly dependent on the reinforcement content. Nominal crack widths of 0.4 mm occurred with rebars of Ø 18 mm at a settlement of about 120 mm, with Ø 26 mm at 130 to 140 mm.

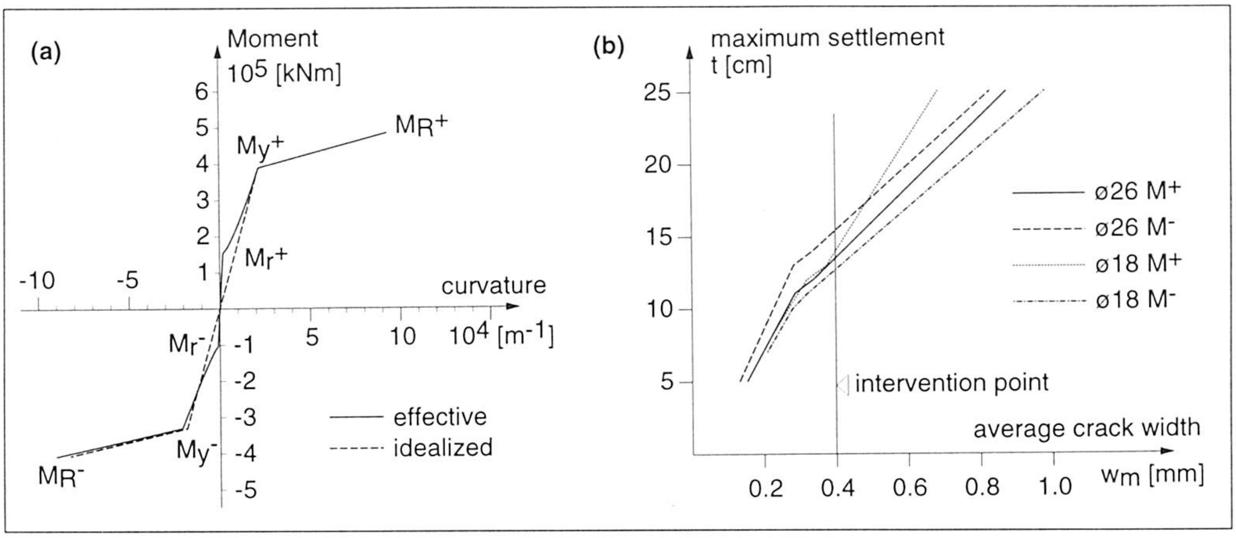


Fig. 4 Moment-curvature diagram for rebars Ø 26 mm (a) and settlement - crack width diagram for standard subsidence shape 1 (b)

Figure 4 (b) shows the relationship between crack width and maximum settlement for different reinforcement contents and both positive and negative bending. The crack widths were calculated at the cracking moments both before and after the yielding moment was reached and for the maximum deflection of 250 mm. All results were arrived at quite simply as described above. The figure shows that the yielding point is not a marked event as the deflections which govern the process are forced. This explains why the amount of reinforcement alone does not greatly influence the crack width.

5.1 Influence on the final design

The amount of reinforcement was governed by the ultimate strength limit state design with the load case of doline and subsidence combined at the same location. The Swiss Federal Railways specified an adequate safety even at a late intervention point concerning cracks and deflections. Reinforcement bars of Ø 26 mm @ 150 mm were finally chosen for both the crown and invert vaults. The ductile behaviour of the tunnel tube and its rotational capacity is herewith guaranteed.

6 References

1 Grether, M. (editor, 1996). *Adlertunnel*, Schweizer Ingenieur und Architekt, Heft 18/1996, pp. 337-370.

2 Swiss Society of Engineers and Architects (1989). *Actions on structures*, Standard SIA 160, 1989 Edition, SIA Zurich, 113 pp.

3 Swiss Society of Engineers and Architects (1993). *Betonbauten*, Norm SIA 162, Ausgabe 1989/Teilrevision 1993, SIA Zurich, 86 pp.

4 Anderheggen, E., Despot, Z. (1994). *STATIK-N – Ein Computerprogramm für die nichtlineare Berechnung von Stabtragwerken*, Schweizer Ingenieur und Architekt, Heft 39/1994, pp. 773-777.