2. Elements of the NATM

2.1 Shotcrete

2.1.1 Components and composition

Requirements

In order to improve workers' protection, minimize environmental pollution (water and ground) and reduce the amount of eluates (alkalis, calcium hydroxides), shotcrete mixes may only be applied in general if they at least equivalent to conventional structural concrete mixes for support elements with respect to their physiological properties and their leaching behavior.

The following requirements, among others, have to be met by the shotcrete:

- Low water permeability,

- no use of alkali-containing additives,

- a minimum strength of the green shotcrete, termed early strength (see Chapter 2.1.3).

The required early strength for the shotcrete can be achieved by either:

- The use of so-called spray bonding agents (SBM) or spray cements, which allow to dispense with setting activators, or

- the use of alkali-free accelerating admixtures in powder or fluid form.

In special cases, e. g. with a high water discharge, spray bonding agents and alkali-free accelerating admixtures may also be applied in combination (ÖBV, 1998).

Bonding agents

According to DIN 18551, the following bonding agents may be used:

- Standard cements according to DIN 1164 (Parts 1 and 100, 1990),

- spray bonding agents or spray cements certified by the supervisory authorities.

If spray bonding agents or spray cements have not been certified by the supervisory authorities, the suitability of the bonding agent for the production of shotcrete must be proved before construction by a testing certificate from an approved institute for materials testing. With respect to the leachability the amount of eluate must not be greater for this bonding agent than for standard cements. Proof of this must be provided by a testing certificate from a public health institute.

On the basis of their rate of reaction, one distinguishes between two types of spray bonding agents (ÖBV, 1998):

- Spray bonding agent SBM-T:
  With a maximum processing time of less than one minute, this type of bonding agent can only be used for the production of shotcrete with dry aggregates (water content $w \leq 0.2$ M.-% and according to the manufacturer's specification, respectively).

- Spray bonding agent SBM-FT:
  With an admissible processing time of several minutes, this type of bonding agent can also be used for the production of shotcrete with wet aggregates (water content w generally 2 M.-% to 4 M.-%).

Admixtures

With respect to the improvement of the shotcrete properties such as workability, stickiness, formation of dust, rebound, strength and tightness of the shotcrete fabric as well as reduction of the heat production, adding hydraulically active admixtures is useful (ÖBV, 1998).

Fly ash is a proven admixture, but the use of other admixtures is also possible (e. g. silica dust, smelting sand, hydraulic lime). The total amount of added ground material and admixtures must not exceed 35 % of the bonding agent (ÖBV, 1998).

Aggregates

For shotcrete, concrete aggregates as specified in DIN 4226 (parts 1 and 2, 1983) must be used (DIN 18551, 1992). The maximum grain size must be selected between 4 and 16 mm (ÖBV, 1998).

Additives

Until a few years ago, alkali-containing accelerating admixtures were used as additives for shotcrete. This way it was possible to achieve a favorable development of early strength (see Chapter 2.1.3). These additives are strongly caustic and due to reasons of environmental protection they are not used anymore. Furthermore, they have a negative effect on the leaching behavior of the shotcrete. This has lead e. g. to drainages being clogged by encrustations and in some cases also to contaminations in the groundwater caused by the eluates. In addition, the shotcrete became porous and permeable to water with the leaching. This results in decreasing strength with progressing age.

It is therefore state-of-the-art today to use spray bonding agents or spray cements without accelerating admixtures or with alkali-free accelerating admixtures, added as powder or in fluid form and certified by the supervising authorities.

The suitability of the planned shotcrete recipe including the used additive must be proved before construction by laboratory testing of the setting behavior, the early strength and the strength development. Laboratory tests yield reference values, but they cannot capture all influences from the construction site and therefore cannot replace suitability testing on site (ÖBV, 1998). Furthermore, it has to be proven that the additives do not have a negative impact on the reinforcement and the remaining steel mounting parts.

Composition

According to ÖBV (1998), the mixes for dry-mix and wet-mix shotcrete are subdivided into:

- Dry mix (TM),
- moist mix, storable (FM-L),
- moist mix for immediate application (FM-S),
- wet mix (NM).

These mixes are referred to as supply mixes. They differ in composition from the sprayed concrete due to the rebound occurring during spraying. The rebound is the share of the shotcrete mix which does not adhere to the surface of application during spraying and which must be disposed of.

2.1.2 Spraying methods

Dry-mix method

For the dry-mix method, TM, FM-L and FM-S mixes can be used. The mix is conveyed intermittently to the spray nozzle via compressed air using a piston or rotary engine (thin stream transport). At the nozzle, it is wetted with water and sprayed onto the surface of application at a speed of 20 m/s to 30 m/s.

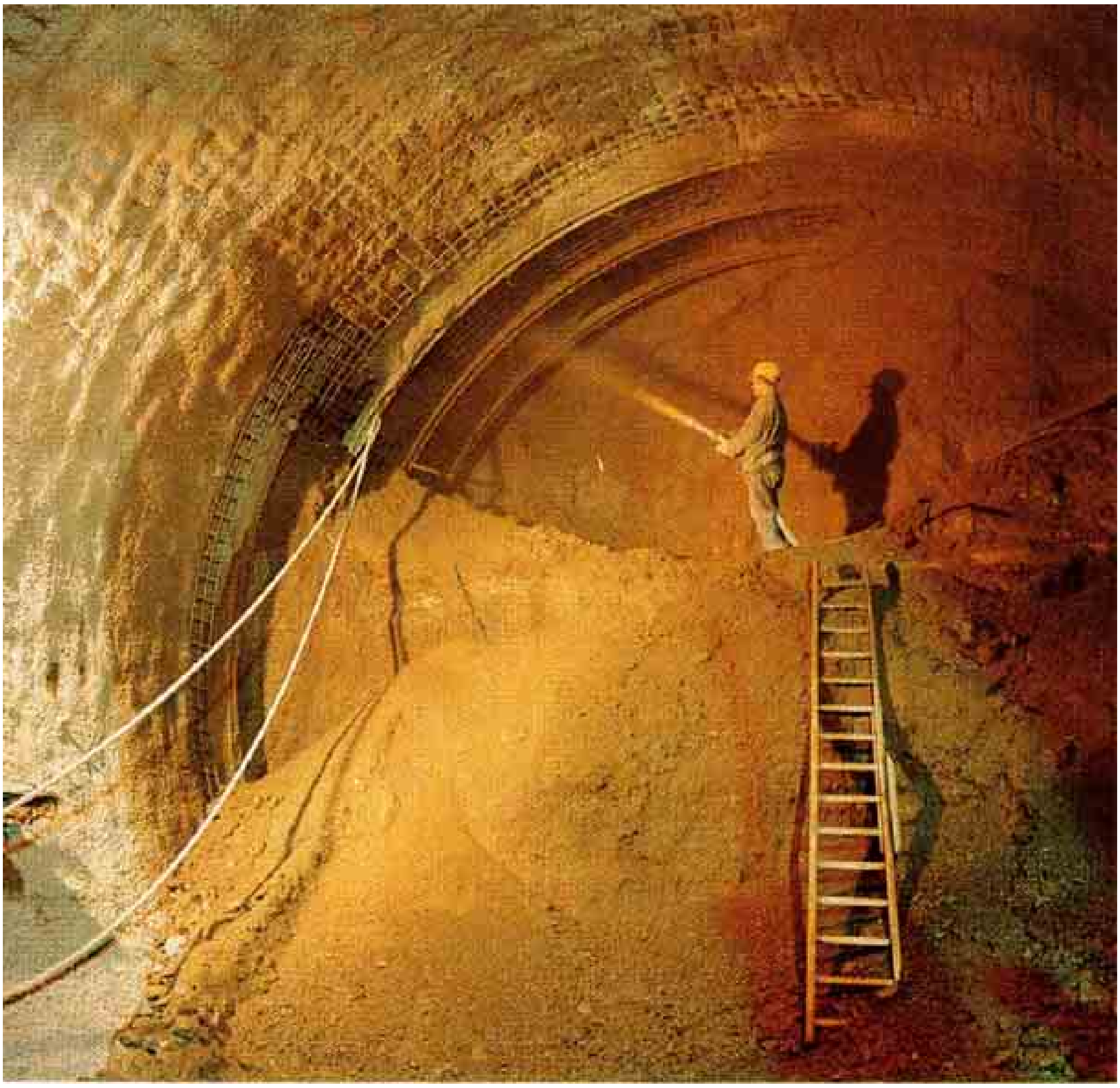


Fig. 2.1: Placing of the shotcrete by a manually guided spray nozzle (DB, 1985)

Fig. 2.2: Placing of the shotcrete using a spray vehicle with a remote-controlled spray arm (Limburg Tunnel, new railway line Cologne – Rhine/Main)

Liquid accelerating admixture is added continuously to the supply water using dosing pumps. Accelerated admixture in powder form is added immediately before the mix transport using proportioners.

Nowadays, the shotcrete is only rarely applied to the tunnel surface by a manually guided spray nozzle (Fig. 2.1). It is standard practice to apply the shotcrete using a spray vehicle with a remote-controlled spray arm (Fig. 2.2). Due to the high velocity of the shotcrete during placing, high rebound portions arise from the dry-mix method. A further problem is the resulting heavy formation of dust. The dry-mix method is therefore only permitted for special cases by the government safety organizations today.

An alternative transportation technique for dry-mix shotcrete was developed by the Rombold und Gfröhrer Co. Here, the dry-mix shotcrete is subjected to compressed air in a pressure tank (silo) and conveyed via dust-encapsulated dosing screws continuously and dust-free with the air stream to the spray nozzle. As in the conventional dry-mix method, the water is added only just before the nozzle (Balbach and Ernsperger, 1986). As a bonding agent, spray cement with a swift development of strength (fast cement) and a high final strength is used (see Chapters 2.1.3, 2.1.4 and 4.1.4). The addition of an accelerating admixture is therefore not necessary.

Advantages of the dry-mix method are the workability in small amounts and the transportability over long distances.

Wet-mix method

With the wet-mix method, the wet mix (NM) is conveyed by the spraying machine to the spray nozzle either by compressed air (thin stream transport) or hydraulically using piston pumps (thick stream transport).

Like dry-mix shotcrete, wet-mix shotcrete is generally applied using a spray vehicle with a remote-controlled spray arm (Fig. 2.2). Manually guiding the spray nozzle is problematic because of the high weight of the wet-mix shotcrete.

Less rebound, less formation of dust and a higher spraying performance are advantages of the wet-mix method over the dry-mix method.

Processing and application

Before the shotcrete is applied, loose rock must be removed from the excavation surface. The surfaces of application must be carefully cleaned with compressed air in order to achieve the best possible adhesion of the shotcrete. This particularly applies if the shotcrete lining is constructed in layers or if longer interruptions occur during the application of the shotcrete.

An immediate sealing of the exposed rock surfaces with shotcrete of at least 3 cm thickness is intended to provide early support to the ground close to the excavation surface in order to largely avoid loosening and the resulting decrease in rock strength.

The shotcrete must be applied in such a way that a homogeneous, dense shotcrete with a closed, even surface is achieved.

Thick shotcrete linings are applied in two or more layers in order to avoid separation from the excavation surface. The shotcrete must be applied in such a way that spraying rebound and adherent spray dust into the shotcrete is avoided by all means. Rebound and dust must be removed before the next shotcrete layer is applied, and the shotcrete lining must always be constructed from the bottom to the top.

The distance between the nozzle and the surface of application must be adapted to the delivery rate and the speed of application. It ranges between 0.5 and 2.0 m, depending on the air flow. The nozzle should be oriented at right angles to the surface of application, if possible. Exceeding or falling below the recommended nozzle distance as well as an inclined orientation of the nozzle relative to the surface of application generally lead to a reduced quality of the shotcrete and an increased amount of rebound. In case of steel insertions such as steel arches, steel girders, lagging plates, pipes, etc., spray shadows cannot be totally avoided, but they can be considerably reduced by proper nozzle control (ÖBV, 1998).

Special care has to be taken when the connection is made to the existing shotcrete lining in the crown invert, the bench invert and the permanent invert. Existing rebound must be removed first. The reinforcement and the support arches should be completely wrapped up in shotcrete. It is important that the visible surfaces are constructed in a convex shape only, if possible, in order to achieve an arching effect.

2.1.3 <u>Early strength</u>

Shotcrete up to an age of 24 hours is termed green shotcrete.

With respect to the requirements on strength development, green shotcrete is distinguished into the three early strength classes J_1, J_2 and J_3 determined on the basis of years of experience (Fig. 2.3).

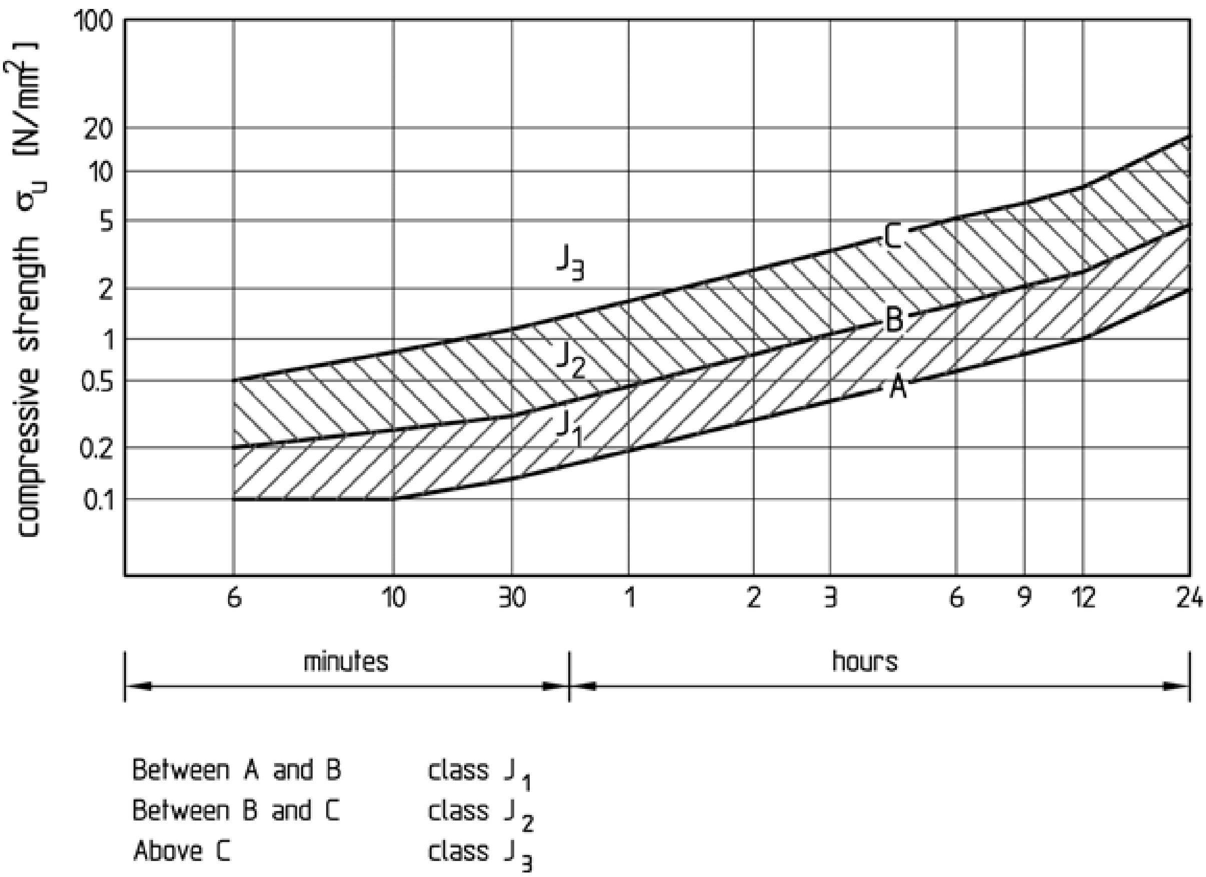


Fig. 2.3: Early strength classes of green shotcrete (ÖBV, 1998)

Class J_1 shotcrete is suitable for the application of thin layers on dry surfaces without specific statical requirements. It has the advantage of little dust development and rebound.

If statical requirements exist with respect to the green shot-crete, e. g. for the exterior lining of a traffic tunnel, class J_2 shotcrete is generally used. Class J_3 shotcrete is required if rapidly developing loads from water pressure and/or rock pressure are to be expected.

It is known from experience that dry-mix shotcretes allow to achieve the highest early strength values. Fig. 2.4 shows the comparison of the strength development of two wet-mix shotcretes, one with accelerating admixture containing alkali (1) and one with alkali-free accelerating admixture (2), and one dry-mix shotcrete with alkali-free spray bonding agent (3). The latter type reaches class J_3. Using accelerating admixtures, either containing alkali or alkali-free, class J_2 is reached. Here, the shotcrete with alkali-free accelerating admixture shows greater early strength.

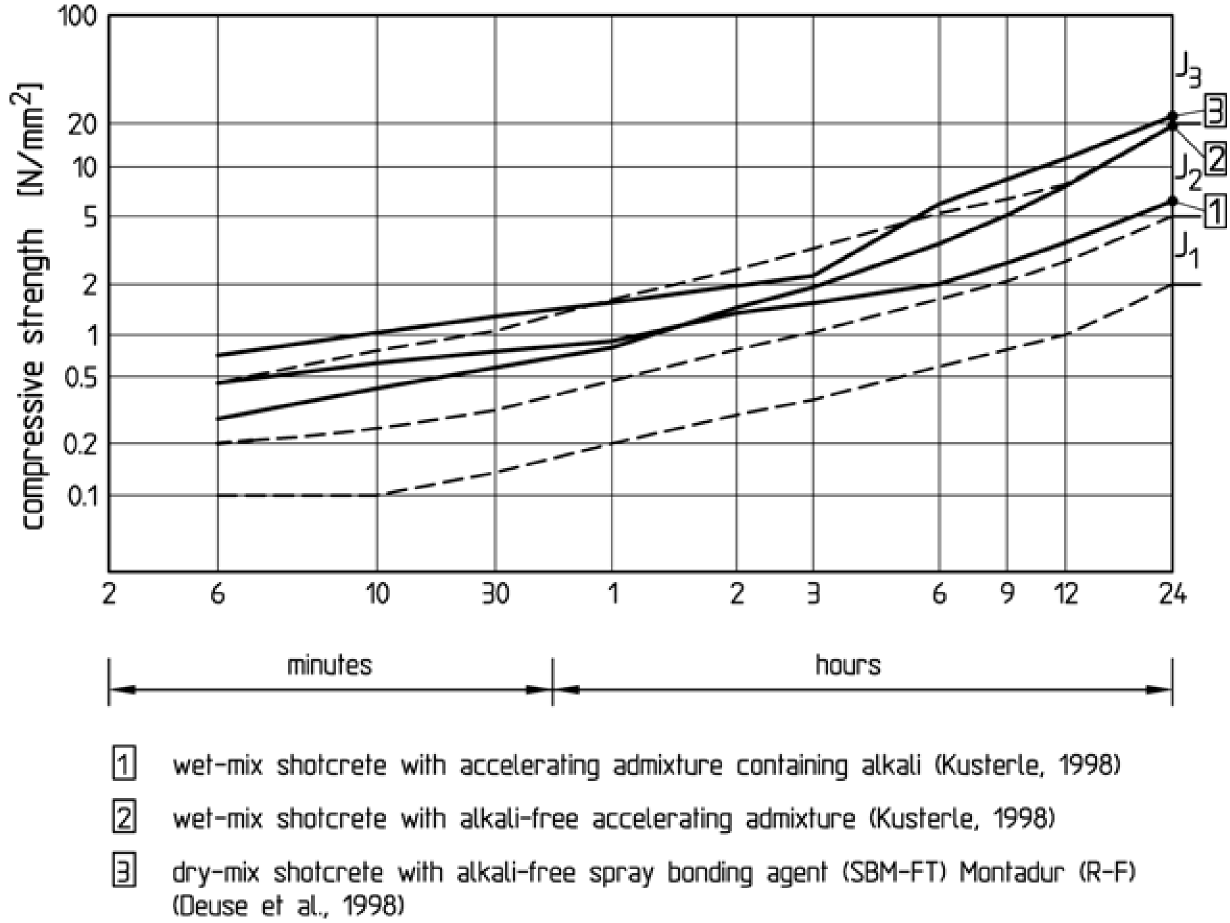


Fig. 2.4: Comparison of the strength development of shotcretes with accelerating admixtures, either containing alkali or alkali-free, and alkali-free spray bonding agent

Because of the disadvantages of the dry-mix method compared to the wet-mix method, which are discussed in Chapter 2.1.2, wet-mix shotcrete is often preferred over dry-mix shotcrete also when high demands are made on the early strength, notwithstanding that a higher early strength can be achieved with dry-mix shotcrete.

An example for this is the shotcrete for the Schulwald Tunnel of the new railway line (NBS) Cologne – Rhine/Main of German Rail. Due to the predicted poor geological conditions, dry-mix shotcrete with spray cement as bonding agent was selected at first. After comprehensive preliminary testing, an early strength corresponding to class J_3 was achieved with the recipe given in Fig. 2.5. Because of the high formation of dust during the application and the insufficient placement performance, it was later decided to change to wet-mix shotcrete with a liquid, alkali-free accelerator (BE U22). With this shotcrete, the recipe and early strength development of which are given in Fig. 2.6, a class J_2 early strength was achieved which proved sufficient. A placement performance of up to 25 m^3/h was obtained with this wet-mix shotcrete, which clearly exceeds the 14 m^3/h achieved with the dry-mix method. Further, rebound values of less than 10 % were reached (Brötz et al., 2000).

Recipe:

450 kg/m^3	spray cement CEM I 32,5 R-SE Chronolith ST	707 kg/m^3	Rhine sand 0/2
		218 kg/m^3	crushed limestone 1/5
		848 kg/m^3	Rhine gravel 2/8

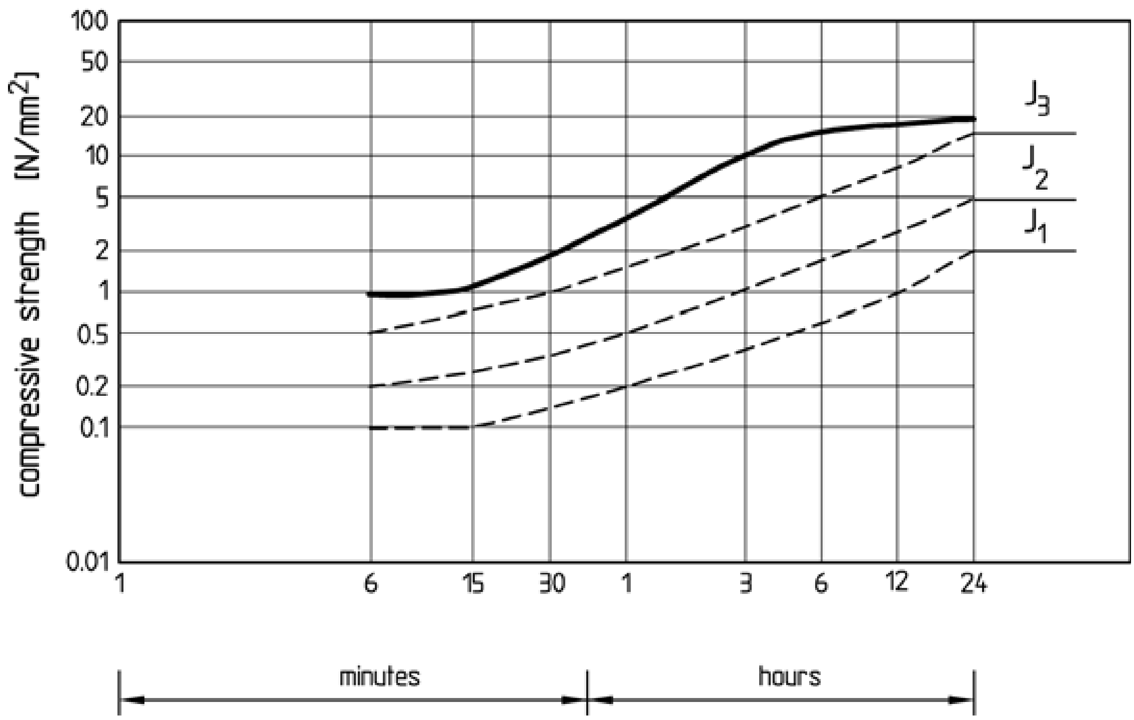


Fig. 2.5: Recipe and early strength development of the dry-mix shotcrete for the Schulwald Tunnel of the new railway line Cologne – Rhine/Main (Brötz et al., 2000)

Recipe:

400 kg/m^3	CEM I 42,5 R Dyckerhoff Amöneburg	864 kg/m^3	Rhine gravel 2/8
50 kg/m^3	Safament	180 kg/m^3	water
798 kg/m^3	Rhine sand 0/2	1.0 % of cement weight	liquefying agent
		5.7 % of cement weight	alkali-free accelerator (BE U22)

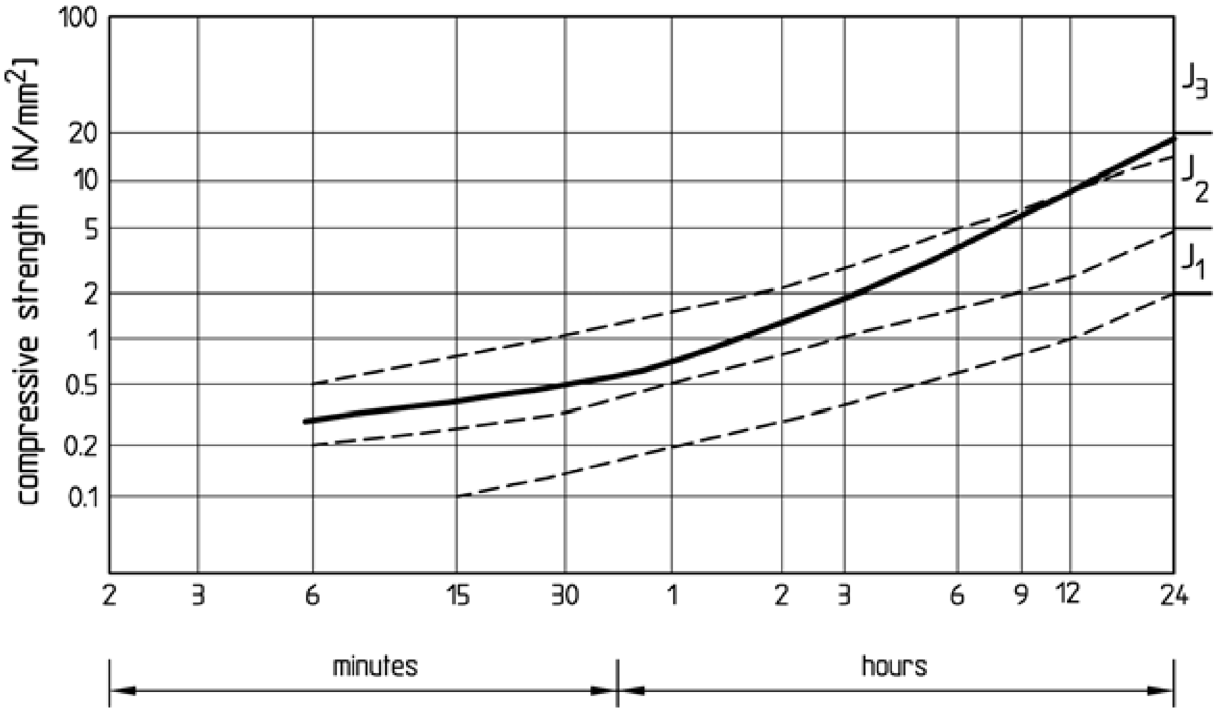


Fig. 2.6: Recipe and early strength development of the wet-mix shotcrete for the Schulwald Tunnel of the new rail-way line Cologne – Rhine/Main (Brötz et al., 2000)

2.1.4 Final strength

Besides its rapid strength development, shotcrete with alkali-free accelerating admixtures or spray bonding agents also possesses a high final strength. While accelerators containing alkali impede the hydration of the cement, this is not the case if alkali-free accelerating admixtures or spray bonding agents are used. A final strength of 30 to 40 N/mm^2 can be obtained in practice (Brötz et al., 2000; Bauer, 2000; NATS, 1998). With shotcrete with accelera-tors containing alkali, even a final strength of 25 N/mm^2 combined with a high early strength is difficult to achieve (NATS, 1998).

2.1.5 Deformability

The deformation behavior of green shotcrete according to Hesser (2000) is essentially characterized by

- hardening with time and a creep ability decreasing with time,

- overproportional, non-linear creep with increasing load.

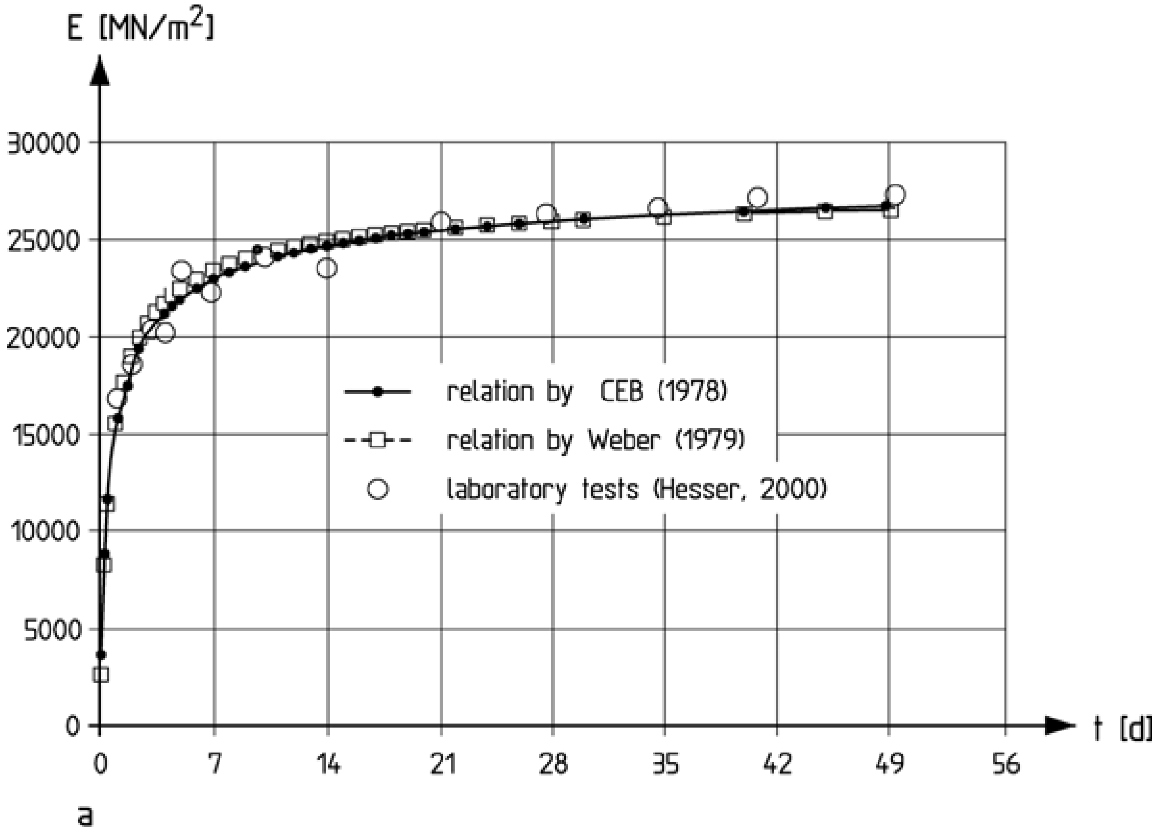
The Young's moduli determined by Hesser (2000) in laboratory tests on dry-mix shotcrete test specimens of different ages show good agreement with the relations by Weber (1979) and by the Comité Euro-International du Béton (CEB, 1978) (Fig. 2.7a).

Fig. 2.7b shows the development of Young's modulus in the first 24 hours. According to this, Young's modulus of the shotcrete amounts to approx. 15000 MN/m^2 after 24 hours, with creep and shrinkage not being taken into account.

Investigations by Manns et al. (1987) showed that the creep and shrinkage deformations of wet-mix shotcrete are larger than those of dry-mix shotcrete. In comparison to standard concrete, the creep and shrinkage deformations of shotcrete are generally clearly larger.

In finite element stability analyses for tunnels, the development of Young's modulus of the shotcrete with time as well as creep and shrinkage are generally not taken into account. An equivalent modulus is instead assigned to the shotcrete to account for hardening during application of the load as well as creep and shrinkage.

The interpretation of the displacements and stresses measured at different tunneling projects by means of back analyses has shown that a modulus of E = 15000 MN/m^2 can be used as equivalent modulus of the shotcrete, corresponding to the 24-hour-value after CEB and Weber (see Fig. 2.7b). Specially in cases, where the shotcrete is already loaded at a young age due to short round lengths and early closing of the invert, values of 2000 to 7500 MN/m^2 for the equivalent modulus have proven to be more realistic (see Chapters 6.1 and 7.1).



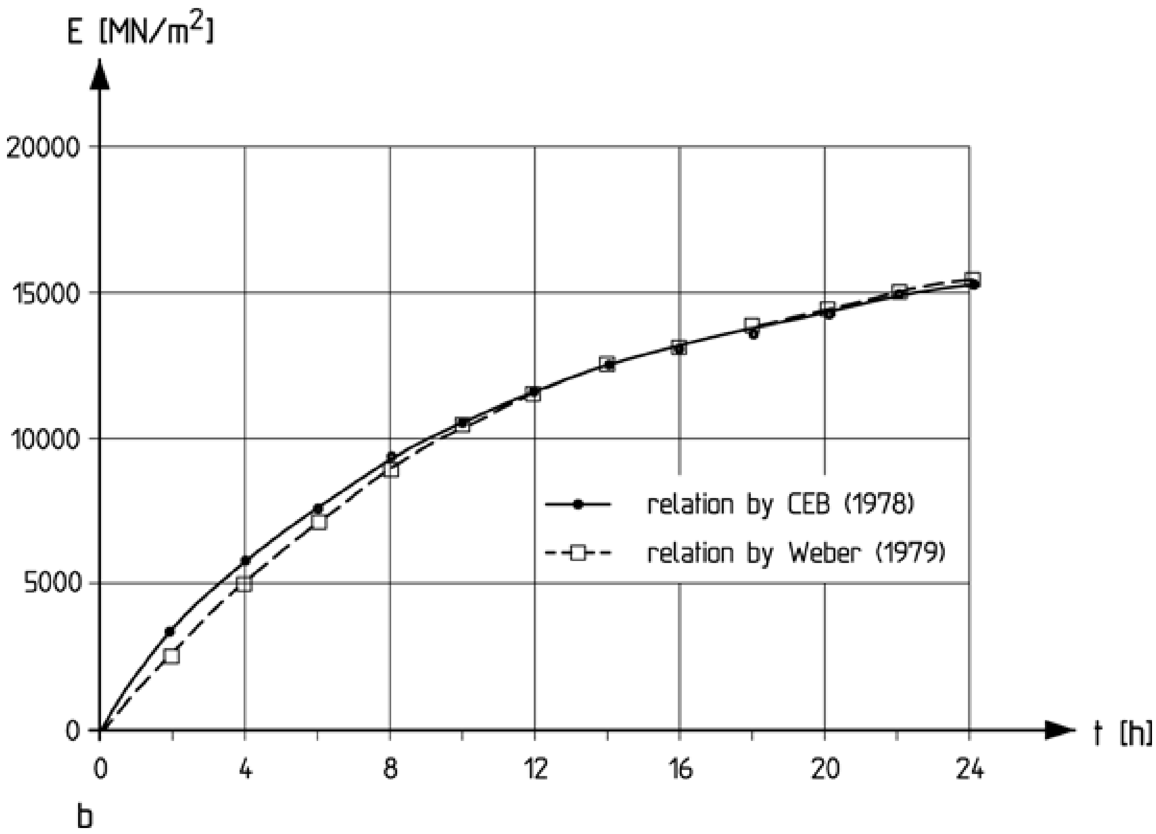


Fig. 2.7: Development of Young's modulus for shotcrete: a) In the first 49 days; b) in the first 24 hours

The development of the deformability of shotcrete with time including shrinkage and creep and its representation in numerical analyses are still a subject of further development.

2.1.6 Rebound

As mentioned above, the amount of rebound is higher for dry-mix shotcrete than for wet-mix shotcrete. Using dry-mix shotcrete with spray bonding agent, however, allows to reduce the rebound considerably as compared to conventional dry-mix shotcrete (Fig. 2.8).

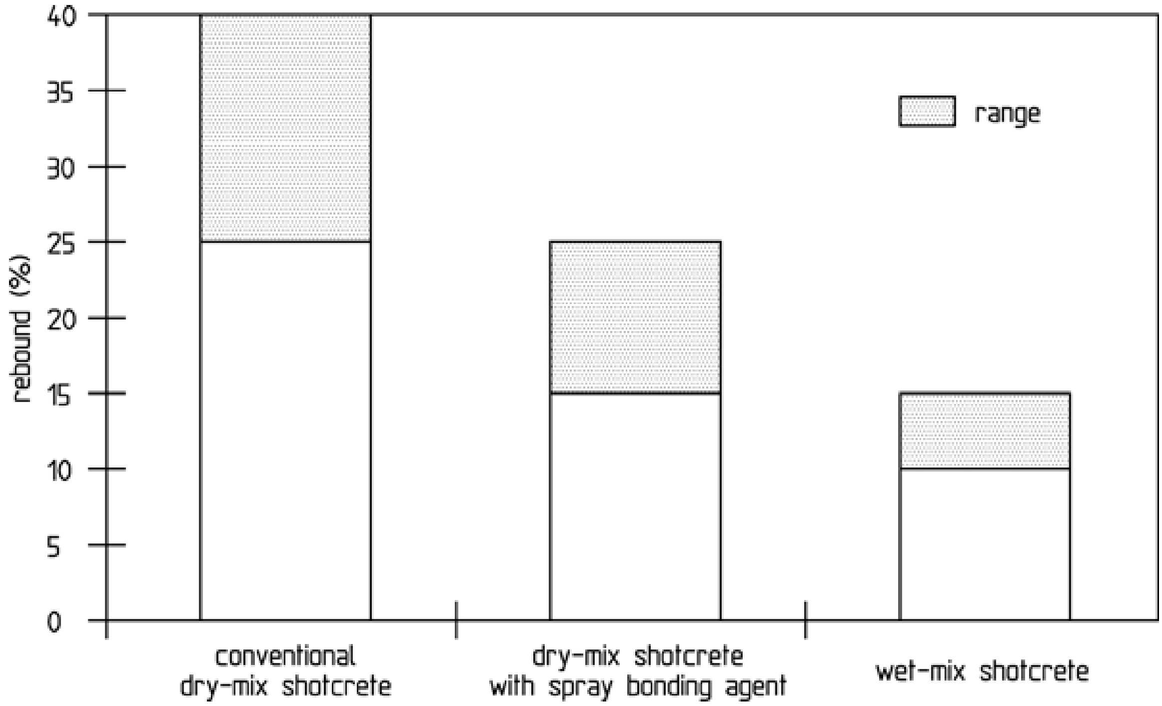


Fig. 2.8: Comparison of rebound between conventional dry-mix shotcrete, dry-mix shotcrete with spray bonding agent and wet-mix shotcrete (NATS, 1998)

The rebound further depends, among other things, on the water cement ratio, the aggregates and the cement type (Maidl, 1992).

2.2 Steel sets

2.2.1 Basic types

Steel sets are made with different profiles. Examples are shown in Fig. 2.9. One distinguishes between plain girders and lattice girders.

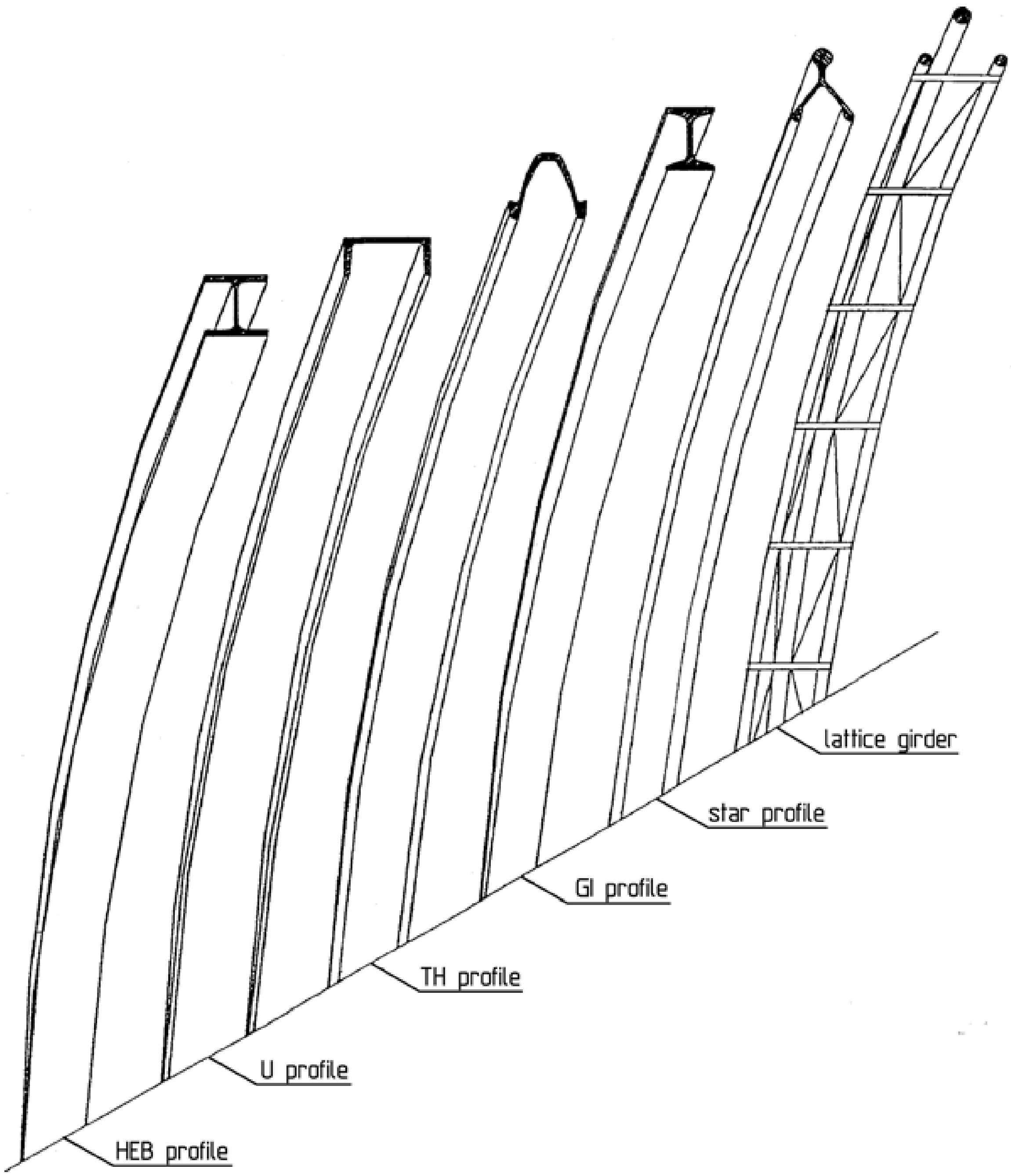
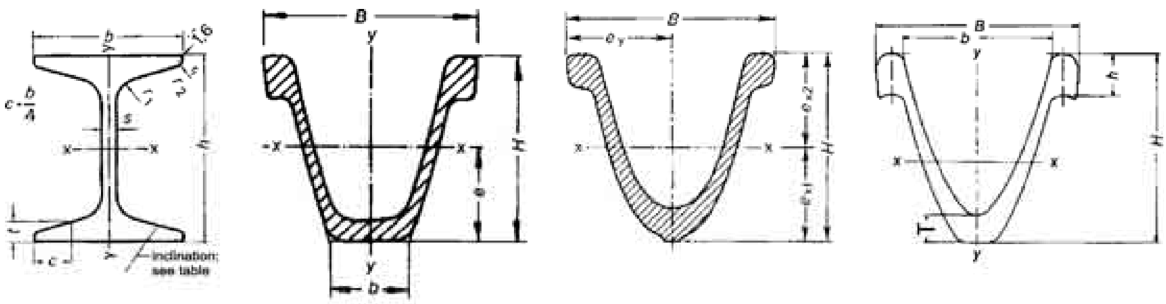


Fig. 2.9: Profiles for steel sets, survey (Heintzmann Ironworks Bochum, Germany)

Among the plain girders are e. g. GI profiles (mining I profiles), TH profiles, bell profiles and standard profiles. Their dimensions, weights, geometrical moments of inertia, section moduli and characteristic parameters are given in Fig. 2.10. Further plain girders are star profiles, the specifications of which are shown in Fig. 2.11.



GI profile TH profile bell profile standard profile

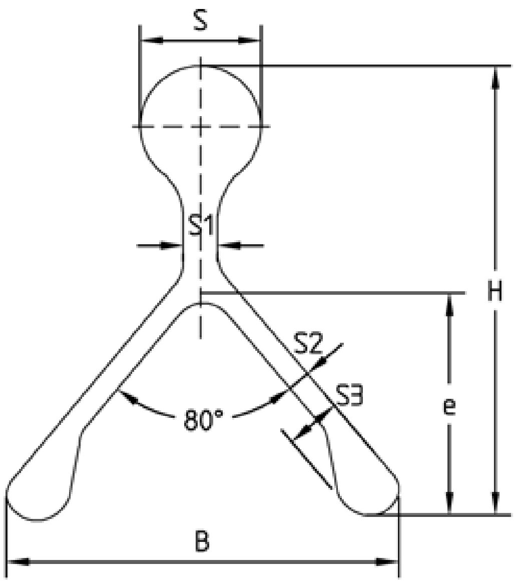
GI profiles	h mm	b mm	s mm	t mm	r_1 mm	r_2 mm	inclination %	cross-section A cm^2	weight kg/m	characteristic parameters x−x J_x cm^4	W_x cm^3	i_x cm	y−y J_y cm^4	W_y cm^3	i_y cm
GI 100	100	80	9	12,5	13	4		26,4	20,7	403	80,7	3,91	80,5	20,1	1,75
GI 110	110	84	10	14	14	5		31,1	24,5	570	103	4,28	103	24,5	1,82
GI 120	120	92	11	15,5	15	6	33	37,6	29,5	816	136	4,66	150	32,6	2,00
GI 130	130	100	12	17	16	7		44,6	35,0	1130	175	5,05	211	42,3	2,18
GI 140	140	110		19	17	8		53,0	41,6	1586	227	5,47	315	57,3	2,44

TH profiles		kg/m / type	13 / 48	16 / 48	21 / 58	25 / 58	29 / 58	36 / 58	44 / 58
height	H	mm	85	89	108	118	124	138	147,8
width	B	mm	98	98	124	135	151	171	172
width	b	mm	36	36	35	38	44	51	50
area	A	cm^2	16	20	27	32	37	46	56
weight	G	kg/m	13	16	21	25	29	36	44
geometrical moment of inertia	Jx	cm^4	137	176	341	484	616	972	1265
	Jy	cm^4	150	196	398	560	775	1264	1564
section modulus	wx	cm^3	32	40	61	80	94	137	174
	wy	cm^3	31	40	64	83	103	148	182
distance of neutral fiber	e	mm	41,9	44,35	52,4	57,5	58,2	66,8	72,3

bell profiles kg/m	profile dimensions H mm	B mm	A cm^2	G kg/m	e_{x1} cm	e_{x2} cm	characteristic parameters J_x cm^4	W_{x1} cm^3	W_{x2} cm^3	e_y cm	J_y cm^4	W_y cm^3
26	123,2	141,0	32,9	25,8	6,15	6,17	510,4	83,0	82,7	7,05	530,1	75,2
28	125,00	141,0	35,5	27,8	6,24	6,26	555,7	89,1	88,7	7,05	572,2	81,2
30	126,80	141,0	38,0	29,8	6,33	6,35	601,3	95,0	94,7	7,05	614,2	87,1
32	137,25	147,0	40,6	31,8	6,84	6,89	767,8	112,3	111,5	7,35	721,9	98,2
34	139,00	147,0	43,1	33,9	6,93	6,97	823,8	119,0	118,1	7,35	768,2	104,5
36	140,75	147,0	45,7	35,9	7,01	7,06	880,5	125,5	124,7	7,35	814,6	110,8
42	157,00	179,0	53,4	41,9	7,82	7,88	1331,9	170,3	169,0	8,95	1390,6	155,4

standard profiles	H mm	h mm	B mm	b mm	T mm	A cm^2	G kg/m	J_x cm^4	J_y cm^4	W_x cm^3	W_y cm^3
EP 80/28	129	31	139	100	19,5	35,6	28	626	587	97	85
EP 80/34	142	31	153	113	21	43,3	34	880	870,5	126	113,8
EP 80/40	155	33,6	166	166	23	51	40	1246	1205	160,8	145,2

Fig. 2.10: Specifications of GI-profiles, TH profiles, bell profiles and standard profiles (Maidl, 1984)



S mm	S1 mm	S2 mm	S3 mm	e mm	H mm	B mm	kg/m	J_x cm^4	W_x cm^3	J_y cm^4	W_y cm^3
28	8.5	5	13	53	106	92	14	207	40	86	19
32	9	6	15	60	120	104	18	346	60	143	28

Fig. 2.11: Specifications of star profiles (Heintzmann Iron-works Bochum, Germany)

Fig. 2.12 to 2.14 show examples of butt joints of HEB profiles, TH channel profiles and star profiles.

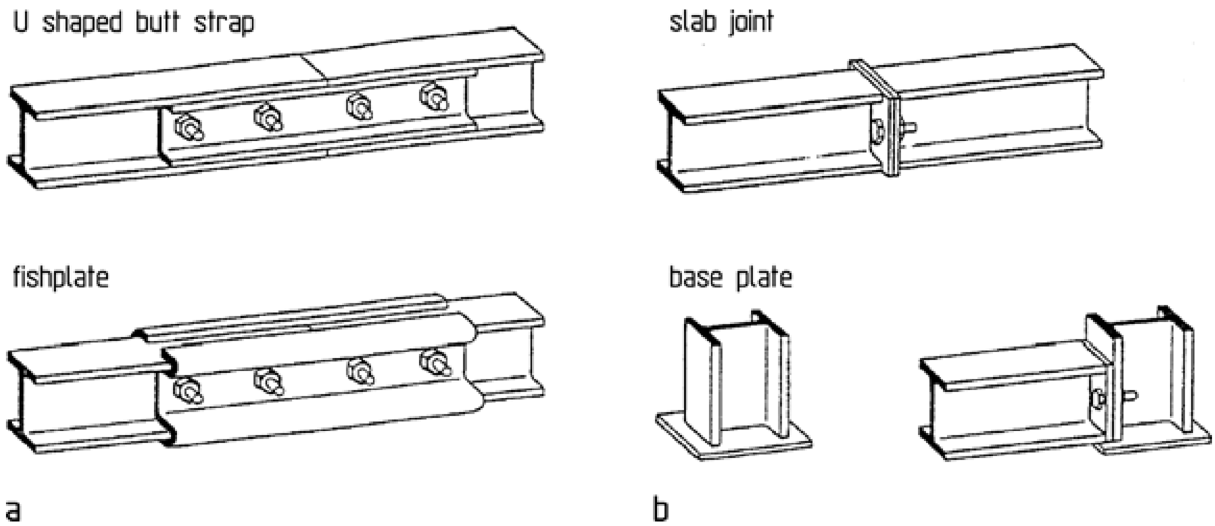


Fig. 2.12: HEB profile joints: a) Butt strap joints; b) slab joints

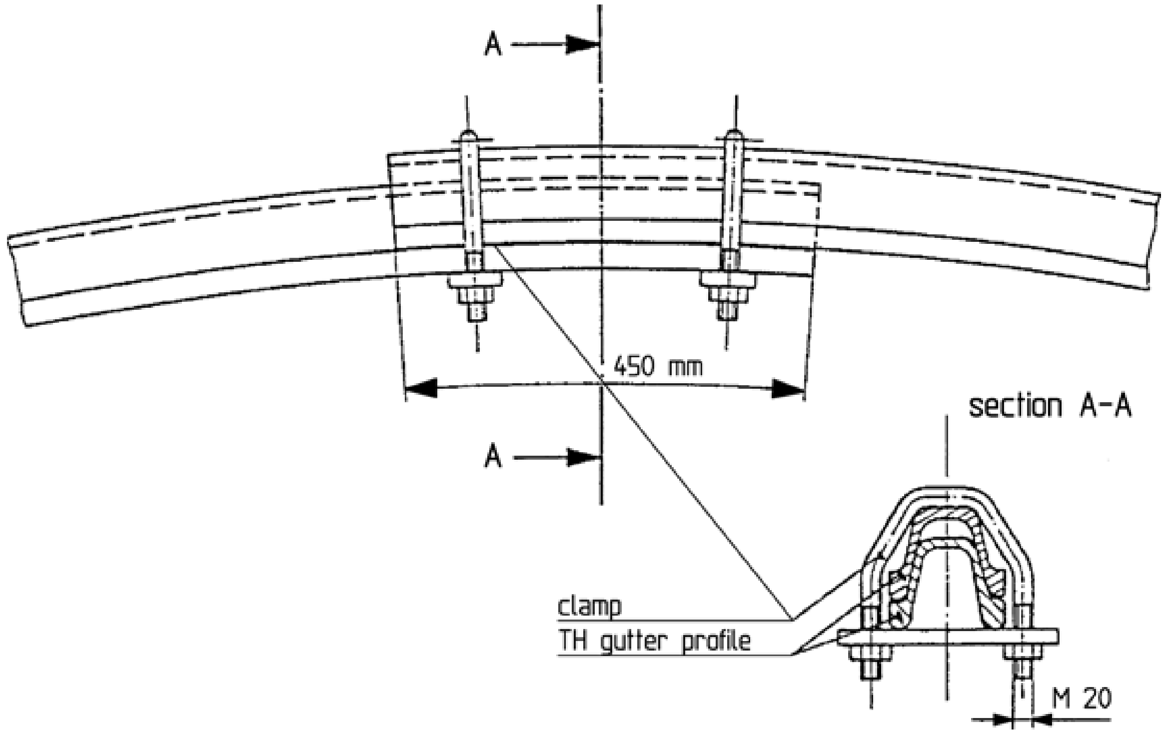


Fig. 2.13: Flexible joint for TH gutter profiles

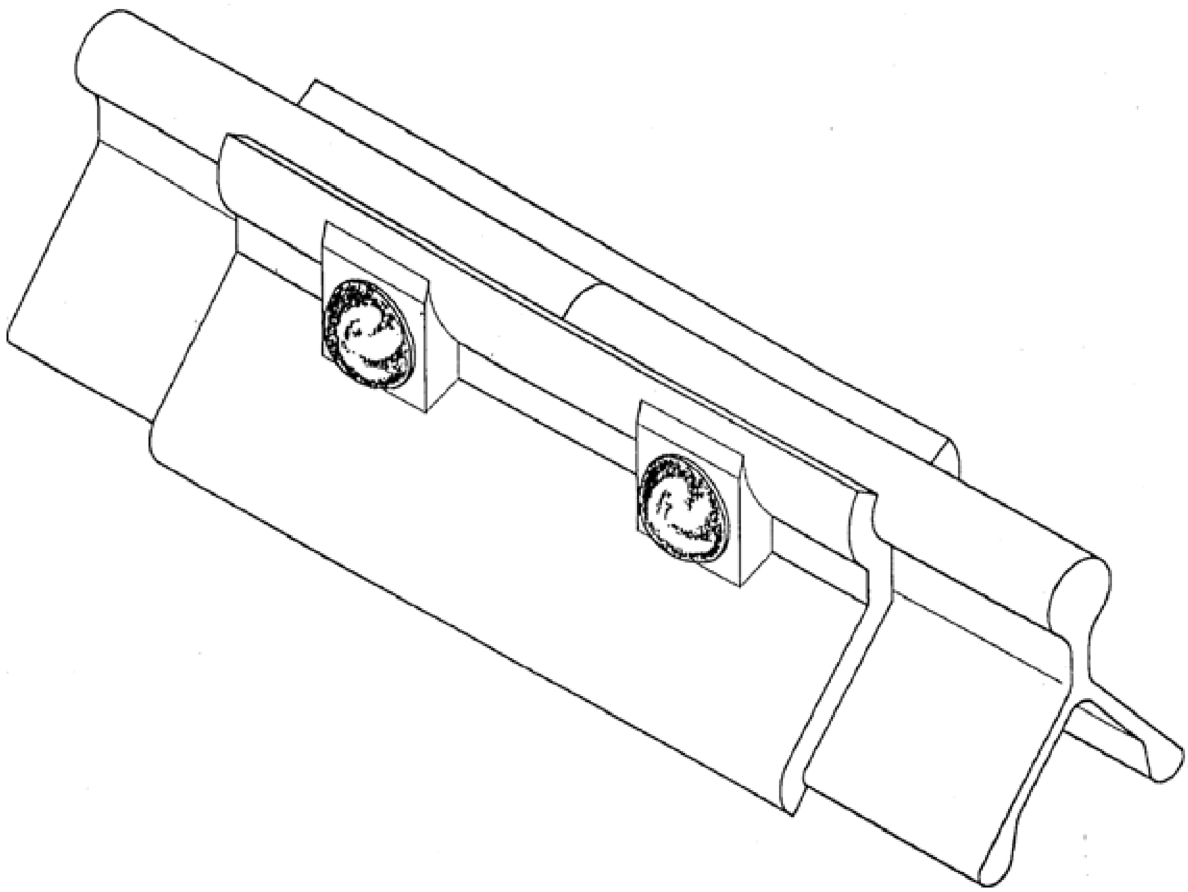
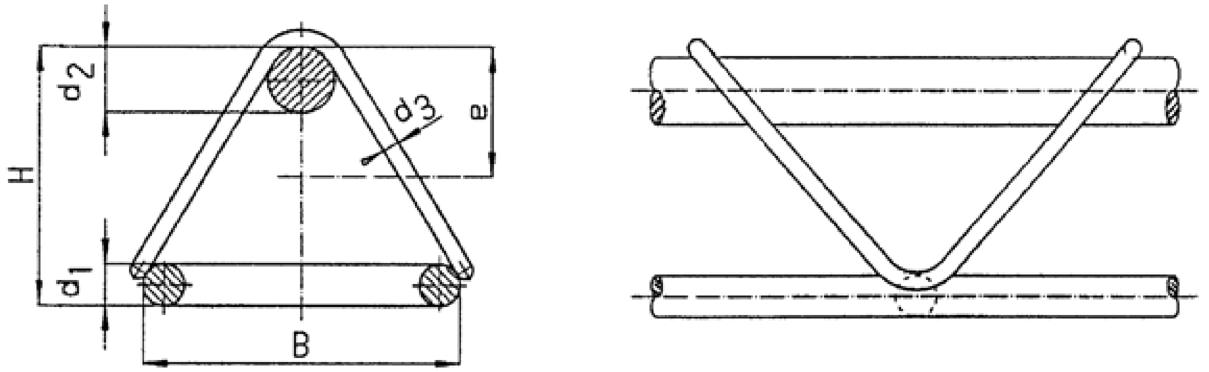


Fig. 2.14: Star profile joint (Heintzmann Ironworks Bochum, Germany)

For the same bending stiffness, lattice girders have less weight per meter of girder length than plain girders. They are therefore easier to handle. Lattice girders are distinguished into 3-stringer girders (Fig. 2.15) and 4-stringer girders. Fig. 2.16 shows the specifications for 3-stringer lattice girders in which the rods are welded to the stiffening elements from the inside. The rods of the Pantex 3-stringer and 4-stringer PS-girders are welded to the stiffening elements from the outside (Fig. 2.17, 2.18 and 2.19).

Fig. 2.15: 3-stringer lattice girder (Dernbach Tunnel, new railway line Cologne – Rhine/Main)



steel grade BSt 500S

H mm	B mm	d_1 mm	d_2 mm	d_3 mm	I_x cm^4	W_x cm^3	I_y cm^4	W_y cm^3	e cm
125	150	18	25	10	224	34	225	30	6.52
		20	28	10	323	50	270	36	6.50
		22	32	12	382	61	339	45	6.30
145	175	18	25	10	384	51	317	36	7.54
		20	28	10	460	61	382	44	7.51
		22	32	12	551	75	450	51	7.30
165	200	18	25	12	518	61	425	42	8.55
		20	28	12	623	73	513	51	8.52
		22	32	12	751	90	610	61	8.31
185	225	18	25	12	672	70	550	44	9.57
		20	28	12	810	85	664	59	9.53
		22	32	12	983	106	790	70	9.28

Fig. 2.16: Specifications of 3-stringer lattice girders (Heintzmann Ironworks Bochum, Germany)

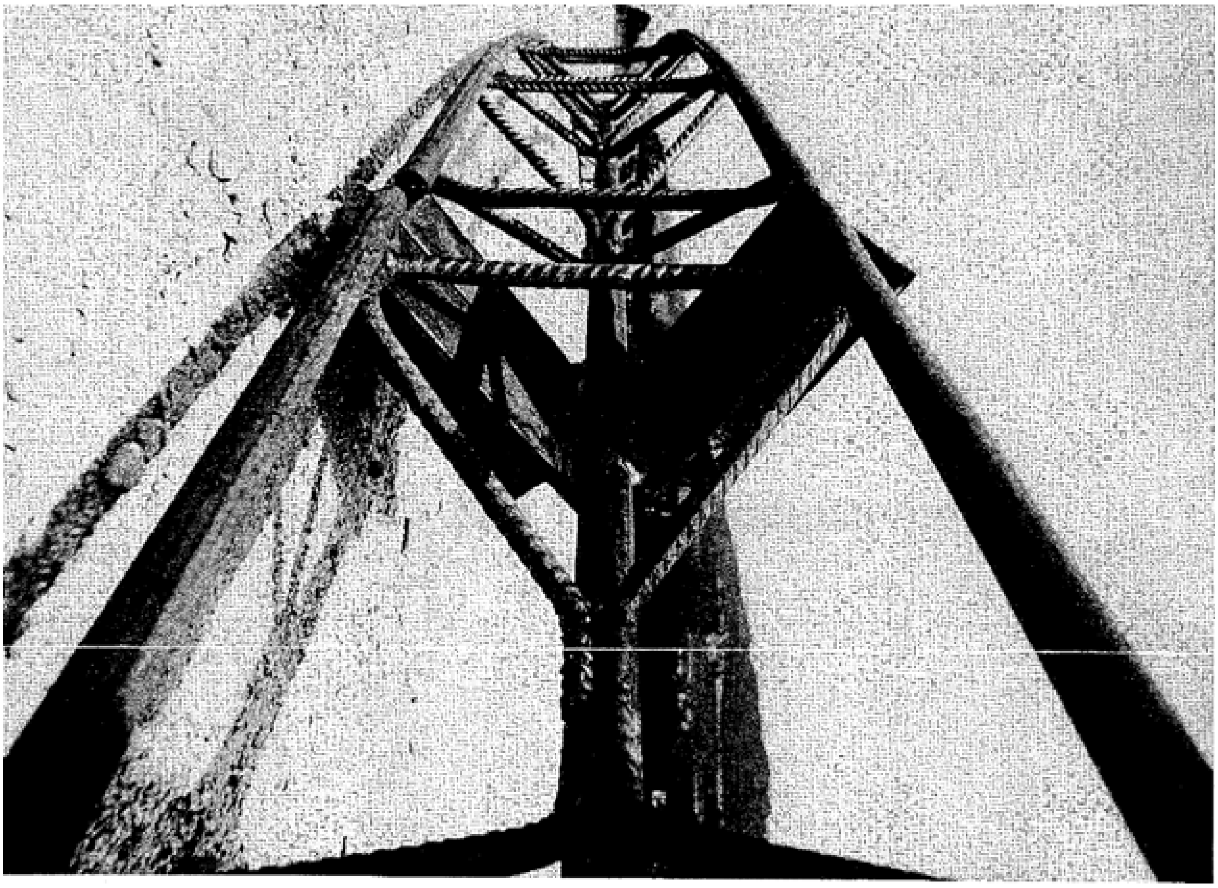
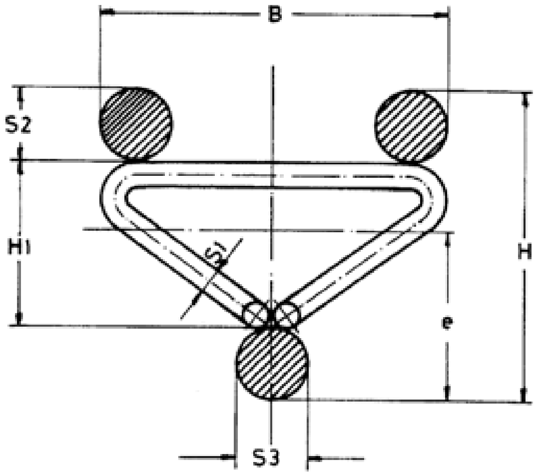


Fig. 2.17: Pantex 3-stringer PS-girder (Tunnel-Ausbau-Technik Ltd., Germany)

For the same height, plain girders like GI, TH and other profiles have a far greater normal and bending stiffness than lattice girders (Fig. 2.20). The normal stiffness of lattice girders is independent of their height, if the cross sectional area of the stringer rods A_s remains constant. In Fig. 2.20, $A_s = 12.4\ cm^2$ was assumed for the stringer rods ($d_1 = 20$ mm, $d_2 = 28$ mm, see Fig. 2.16).

Structure:
- welded structural steel girder
- stiffening elements connect the stringer rods in space



Properties:
- high stability and bearing capacity
- high moments of inertia
- sound bonding with the concrete
- small spray shadows
- simple handling

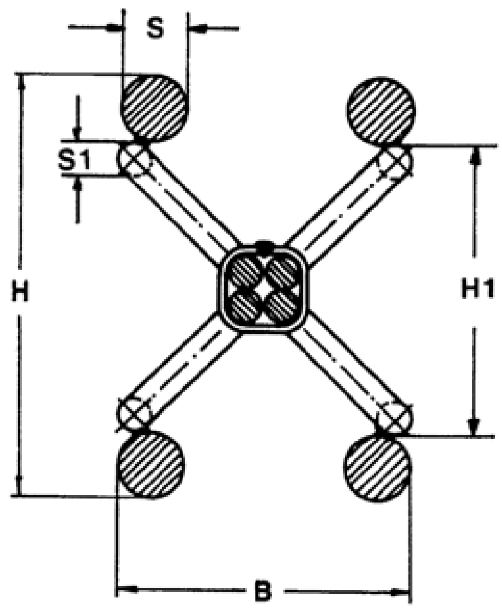
steel grade 500 / 550/ GK / E

type $H_1/S_2/S_3$ mm/mm/mm	S1 mm	S2 mm	S3 mm	e cm	H mm	B mm	kg/m	I_x cm^4	W_x cm^3	I_y cm^4	W_y cm^3
50/18/22	8	18	22	5.10	90	100	8.20	109	21	87.0	17.0
50/20/26	8	20	26	3.70	96	100	10.32	185	32	104.0	21.0
50/26/30	8	26	30	6.18	106	100	15.10	267	43	153.0	31.0
70/18/18	10	18	18	6.76	106	140	8.20	133	20	190.0	27.0
70/18/20	10	18	20	6.50	108	140	8.67	155	24	191.0	27.0
70/18/22	10	18	22	6.25	110	140	9.18	179	29	191.2	27.3
70/18/26	10	18	26	5.80	114	140	10.37	233	38	192.3	27.5
70/20/26	10	20	26	6.34	116	140	11.31	253	40	230.0	33.0
70/26/30	10	26	30	7.38	126	140	16.09	413	56	333.0	48.0
90/18/18	10	18	18	8.10	126	180	8.55	199	25	335.0	37.0
90/18/20	10	18	20	7.74	128	180	9.02	232	30	335.0	37.0
90/18/22	10	18	22	7.39	130	180	9.53	265	36	335.5	37.3
90/18/26	10	18	26	6.78	134	180	10.72	329	48	336.5	37.4
90/20/26	10	20	26	7.42	136	180	11.66	370	50	406.0	45.0
90/26/30	10	26	30	8.58	146	180	16.44	590	68	637.0	71.0
100/18/18	12	18	18	9.43	146	220	10.16	275	29	520.0	47.0
100/18/20	12	18	20	8.97	148	220	10.64	325	36	520.0	47.0
100/18/22	12	18	22	8.54	150	220	11.15	369	43	520.1	47.3
100/18/26	12	18	26	7.76	154	220	12.34	455	59	521.2	47.4
100/20/26	12	20	26	8.50	156	220	13.28	512	60	632.0	57.5
100/26/30	12	26	30	9.78	166	220	18.06	811	83	1001.0	91.0
100/26/34	12	26	34	9.25	170	220	19.64	965	105	1007.0	91.6
130/18/26	12	18	26	8.98	174	220	12.58	604	67	521.0	47.4
130/20/26	12	20	26	9.60	176	220	13.52	677	71	632.0	57.5
130/26/30	12	26	30	10.98	186	220	18.30	1065	97	1001.0	91.0
130/26/34	12	26	34	10.32	190	220	19.88	1263	122	1007.0	91.6

Fig. 2.18: Specifications of Pantex 3-stringer PS-girders (Tunnel-Ausbau-Technik Ltd., Germany)

Structure:

- welded structural steel girder
- stiffening elements, positioned at 90° to each other, connect the stringer rods in space



Properties:

- high stability and bearing capacity
- high moments of inertia
- sound bonding with the concrete
- small spray shadows
- simple handling

steel grade 500 / 550/ GK / E

type H_1/S mm/mm	S1 mm	H mm	B mm	system A kg/m	system B kg/m	I_x cm^4	W_x cm^3	I_y cm^4	W_y cm^3
60/18	8	96	100	10.10	9.58	156	33	173	35
60/20	8	100	100	12.00	11.46	204	41	204	41
60/22	8	104	100	14.05	13.50	260	50	234	47
70/18	8	106	100	10.16	9.62	157	38	173	35
70/20	8	110	100	12.04	11.50	257	46	204	41
70/22	8	114	100	14.08	13.54	326	57	234	47
100/18	8	136	100	10.37	9.78	356	52	173	35
100/20	8	140	100	12.25	11.66	456	65	204	41
100/22	8	144	100	14.29	13.70	570	79	234	47
100/26	8	152	100	19.05	18.46	851	112	299	59
140/18	10	176	140	12.35	11.26	637	72	381	54
140/20	10	180	140	14.23	13.14	807	90	456	65
140/22	10	184	140	16.27	15.18	1002	109	534	76
140/26	10	192	140	21.03	19.94	1472	153	699	100
180/18	10	216	180	13.10	11.77	999	93	670	74
180/20	10	220	180	14.99	13.65	1260	115	807	90
180/22	10	224	180	17.02	15.69	1555	139	953	106
180/26	10	232	180	21.78	20.45	2262	195	1268	141
180/30	10	240	180	27.30	25.97	3133	261	1606	178
220/18	12	256	220	16.27	14.21	1443	113	1040	95
220/20	12	260	220	18.15	16.09	1883	139	1399	127
220/26	12	272	220	24.95	22.89	3222	237	2007	182
220/30	12	280	220	30.47	28.41	4434	317	2567	233

Fig. 2.19: Specifications of Pantex 4-stringer PS-girders (Tunnel-Ausbau-Technik Ltd., Germany)

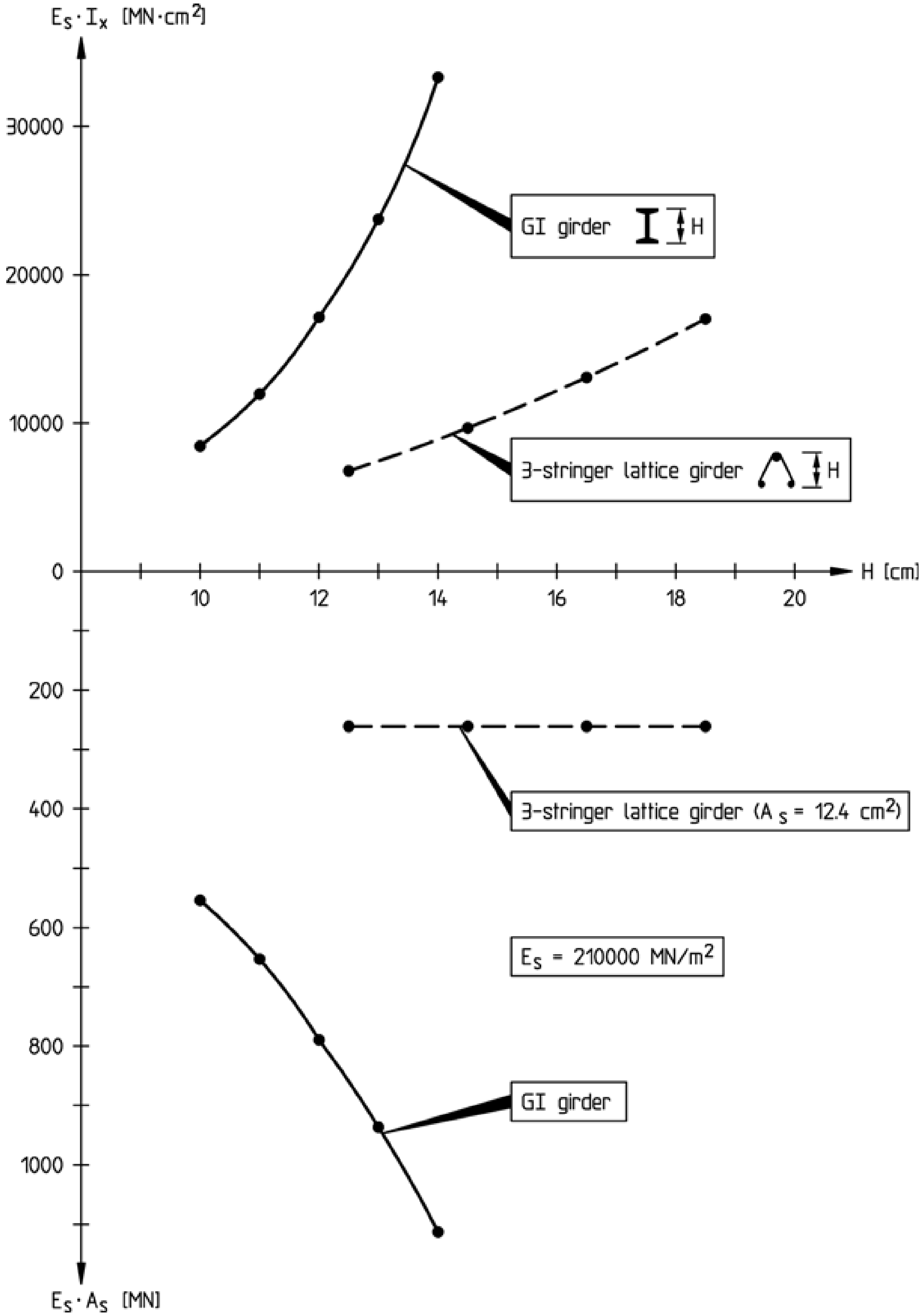


Fig. 2.20: Bending and normal stiffness vs. height of steel sets

2.2.2 Load-carrying behavior

Steel sets only become fully effective as a support if they form a closed ring. In this way they are often used as support in mining without additional support measures.

In tunneling according to the NATM, the steel sets have further tasks as:

- Immediate support of the tunnel face area over the length of the foremost round,

- template of the tunnel profile for the application of the shotcrete and the excavation of the next round,

- support for spiles installed as advance support for the next round (see Fig. 2.15 and Chapter 2.4.1).

In tunneling according to the NATM, steel sets are only rarely installed as a closed ring after each round. Therefore, immediately after their installation, they only have very little bearing capacity.

For the NATM, the load-carrying behavior of the steel sets is based on their bond with the shotcrete membrane. Immediately after a steel set is installed and covered with shotcrete, when the shotcrete still has a very low Young's modulus, it is mostly the steel set that carries the loads resulting from rock mass pressure. Since steel sets are usually installed closely behind the tunnel face, this loading at the beginning is generally small. With progressing hardening of the shotcrete with time, the normal strength and thus the bearing capacity of the shotcrete membrane increases. Finally, after the shotcrete has fully hardened, the normal stiffness of the steel sets can be neglected compared to the one of the shotcrete membrane.

Fig. 2.21 illustrates this for the example of a 30 cm thick shotcrete membrane. It shows the ratio of the normal stiffnesses of the shotcrete membrane and the steel sets vs. the Young's modulus and the age of the shotcrete, respectively for two GI profiles and one 3-stringer lattice girder. A spacing of the steel sets of 1 m is assumed for all cases. It becomes evident that already at the age of a few hours the normal stiffness of the shotcrete membrane exceeds the one of the steel sets. After 24 hours, the normal stiffness of the shotcrete membrane amounts to 17 times of that of the lattice girders and 4 to 8 times the one of the GI profiles.

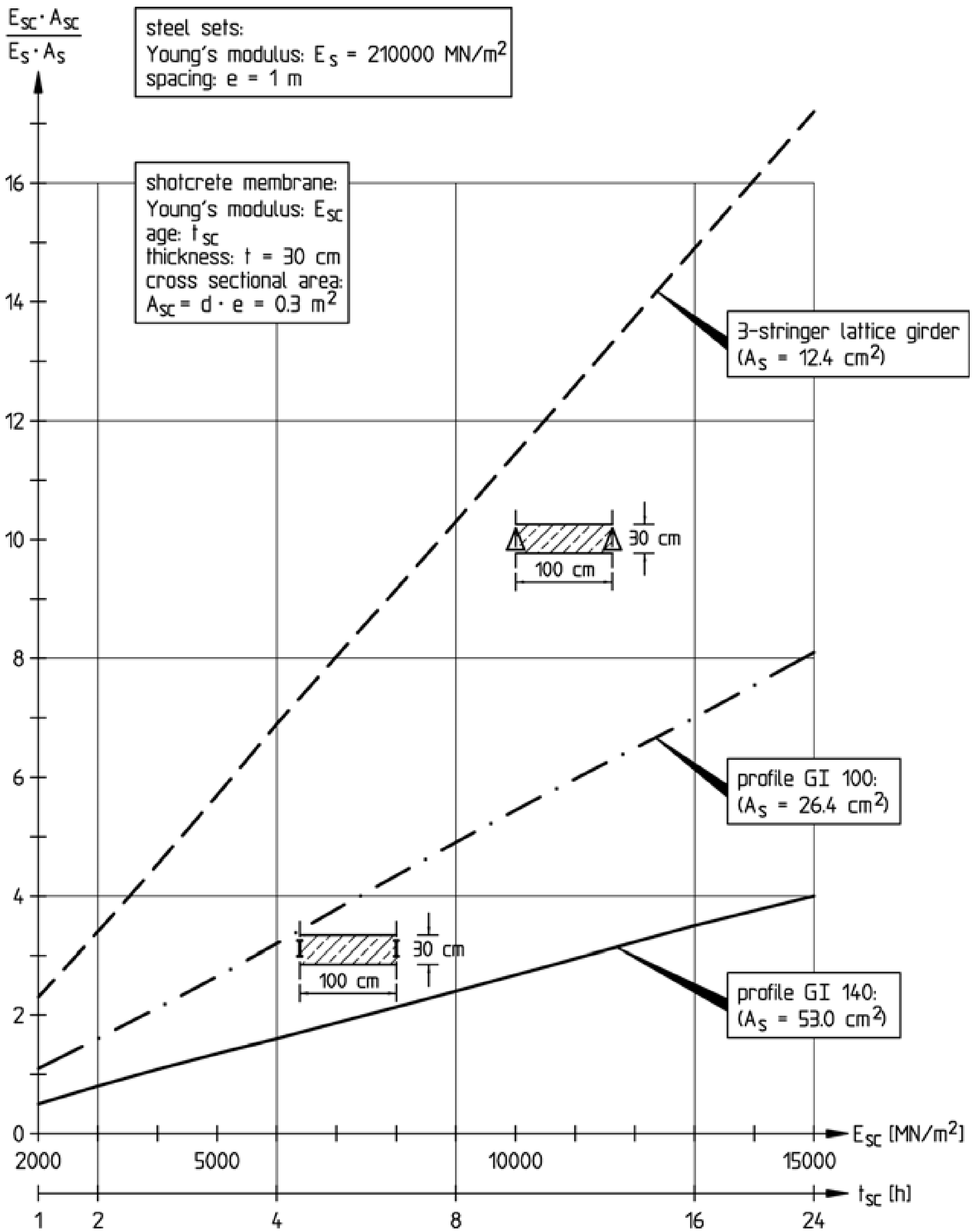


Fig. 2.21: Ratio of normal stiffnesses of shotcrete membrane and steel sets vs. Young's modulus and age of the shotcrete

The steel sets covered with shotcrete can also be accounted for as part of the reinforcement for the dimensioning of the shotcrete membrane (e. g. stringer rods of lattice girders). This requires however that the steel sets are completely covered with shotcrete. Especially if plain girders are used, however, the bond is reduced due to spray shadows. Therefore, in general steel sets are conservatively disregarded as reinforcement and in finite element analyses, the steel sets are generally not modeled.

In Guideline 853 of German Rail (DB, 1999), the following criteria for the selection of steel sets are given:

- Plain girders yield more stable immediate support than lattice girders. This requires, however, friction-locked connections between the supports and the rock.

- Lattice girders bond better with the shotcrete and lead generally to tighter shotcrete membranes than plain girders.

- Spray shadows are generally smaller for lattice girders than for plain girders.

Details regarding the better bonding effect of lattice girders with shotcrete and the consequences for the strength and tightness of the shotcrete membrane can also be found in Eber et al. (1985).

2.3 Anchors

2.3.1 Basic types

Except for special cases, in tunneling mainly non-prestressed (untensioned) anchors, termed rock bolts, are used in boreholes. A detailed description of the terms and designations of rock bolts is given in the German standard DIN 21521 "Gebirgsanker für den Bergbau und den Tunnelbau" ("Rock bolts for mining and tunneling").

With respect to the load-carrying behavior (see Chapter 2.3.2), bond anchors, which are form-locked with the rock mass (Fig. 2.22a to c), are distinguished from anchors that are friction-locked with the rock mass (Fig. 2.22 d). In addition, there are anchors that are form-locked as well as friction-locked with the rock mass (Fig. 2.22e).

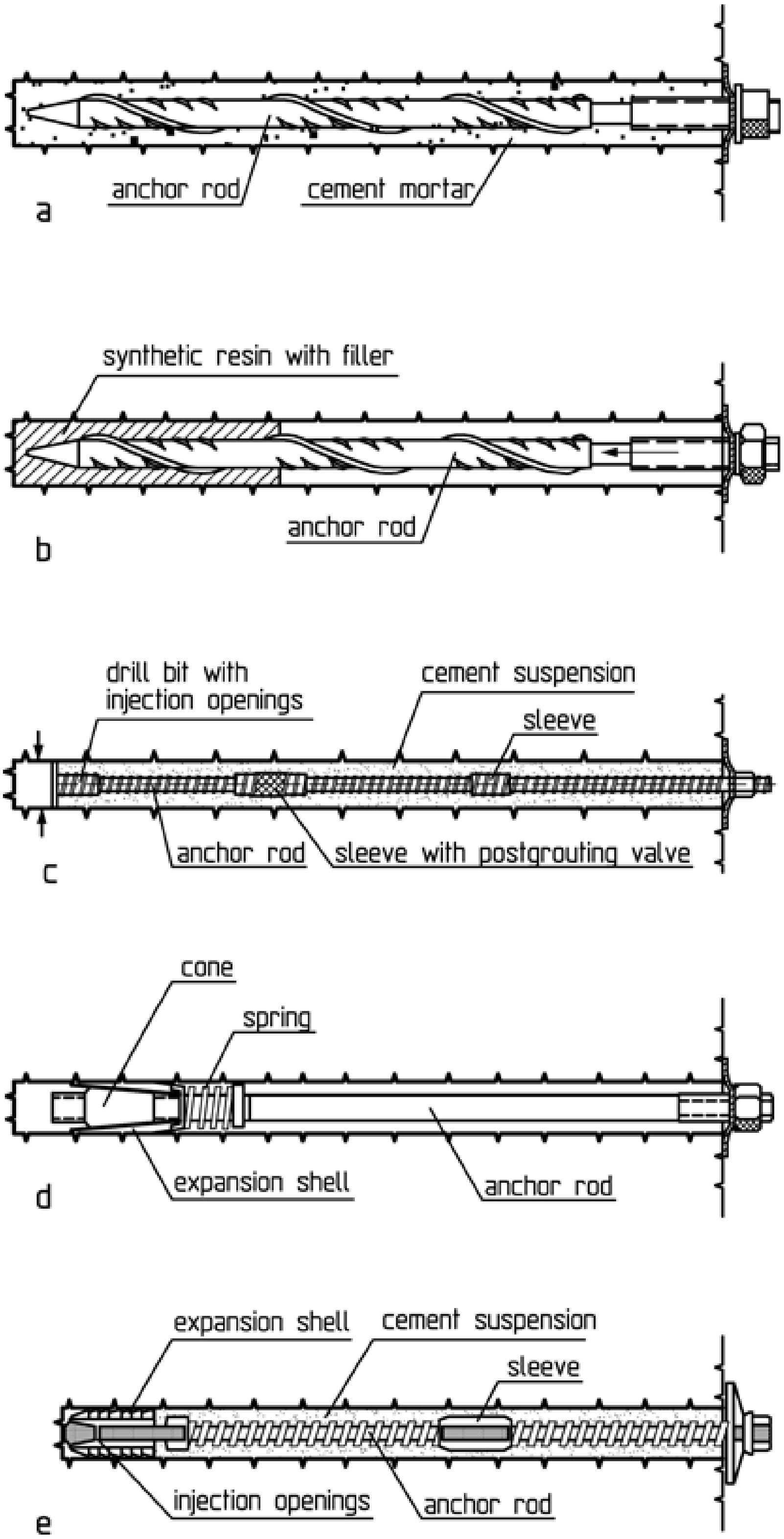


Fig. 2.22: Anchors: Cement mortar anchor (SN-anchor); b) synthetic resin anchor; c) injection drill bolt (IBO-bolt); d) expansion shell bolt; e) injection bolt with expansion shell

Among the form-locked anchors are the mortar anchors (e. g. SN-anchors, IBO-rods), the resin anchors and the friction pipe anchors (e. g. Swellex anchors, split-set anchors). A friction-locked bond with the rock mass results for the expansion shell bolt.

In the case of the mortar anchor and the resin anchor, the bond between the anchor rod and the rock is effected by a setting agent (cement mortar or synthetic resin mortar) over a specific length of the borehole (Fig. 2.23). If the bond extends over the full length of the borehole, the terms full bond anchor or fully cemented anchor are used as well. Among the full bond anchors are also the friction pipe anchors, the anchor rod of which consists of a pipe folded in the longitudinal axis (Swellex-anchor) resp. slit open (split-set-anchor). In the borehole, this pipe is braced against the rock mass by pressing it against the borehole wall. Expansion shell bolts are rock bolts in the case of which the bottom end of the anchor rod is braced against the borehole wall using wedge-shaped or conical elements (expansion elements) (DIN 21521, Fig. 2.22d).

The use of SN-anchors, resin anchors, friction pipe anchors and expansion shell bolts requires that the boreholes for the installation of the anchors are stable. Fig. 2.23 shows the working steps for the installation of a mortar anchor (SN-anchor). For unstable boreholes so-called injection drill bolts (IBO-bolts) are used (Fig. 2.22c and 2.24). Injection drill bolts consist of an anchor pipe which is made of high-strength steel with a continuously rolled-on thread and constitutes the drill bar. A drill bit is screwed on to the bottom end of the anchor pipe. After the borehole has been drilled to the desired depth, it remains with the anchor pipe in the borehole. The bond between the anchor pipe and the rock mass is accomplished by the injection of cement suspension via the anchor pipe through injection openings in the drill bit and the anchor pipe. The loosened rock mass surrounding the borehole is also injected and stabilized in the process (Fig. 2.24).

Expansion anchors constructed as injection bolts (Fig. 2.22e) represent a combination of a friction-locked and form-locked connection between the anchor rod and the rock mass. The working steps for the installation of this anchor type are shown in Fig. 2.25.

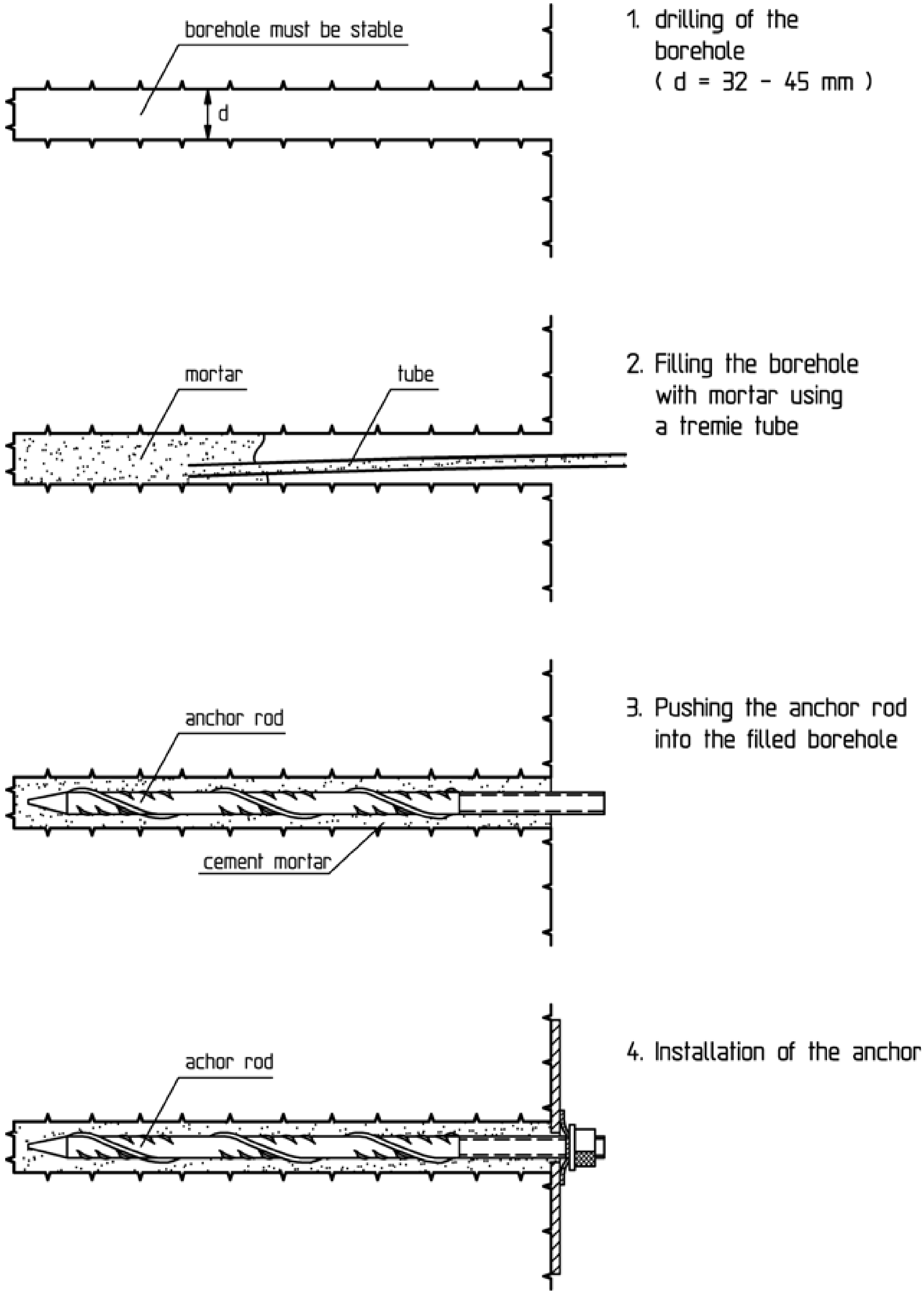


Fig. 2.23: Installation of a mortar anchor (SN-anchor)

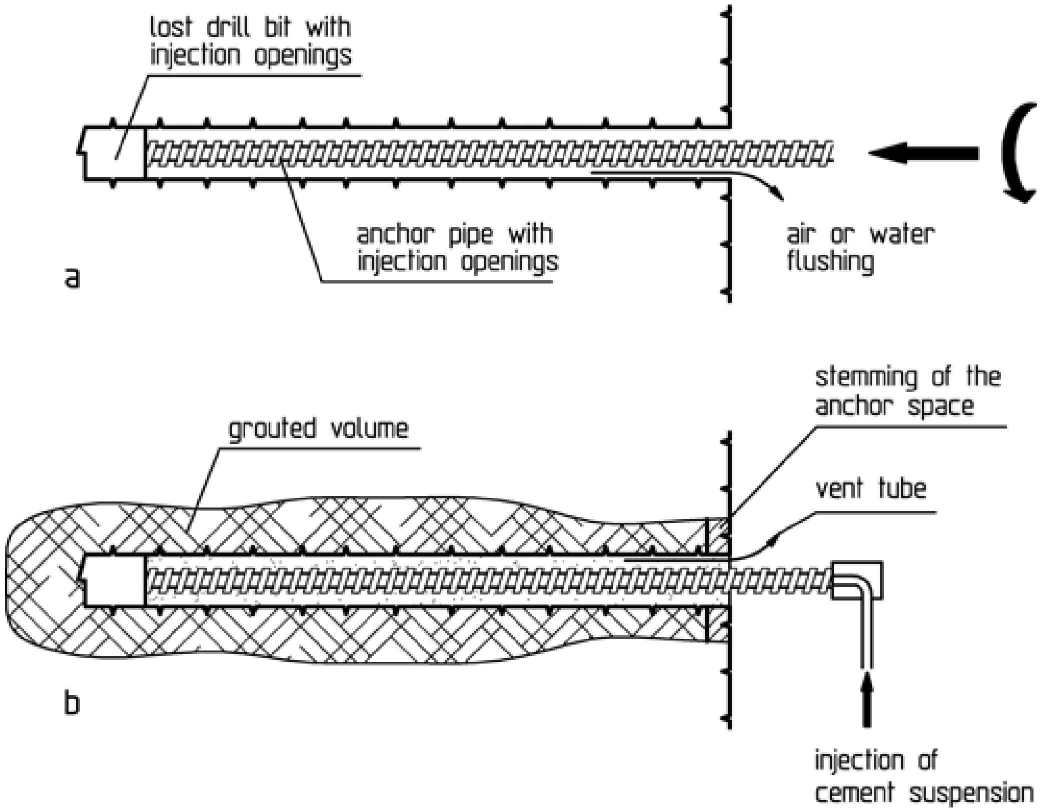


Fig. 2.24: Installation of an injection drill bolt: a) Drilling; b) grouting

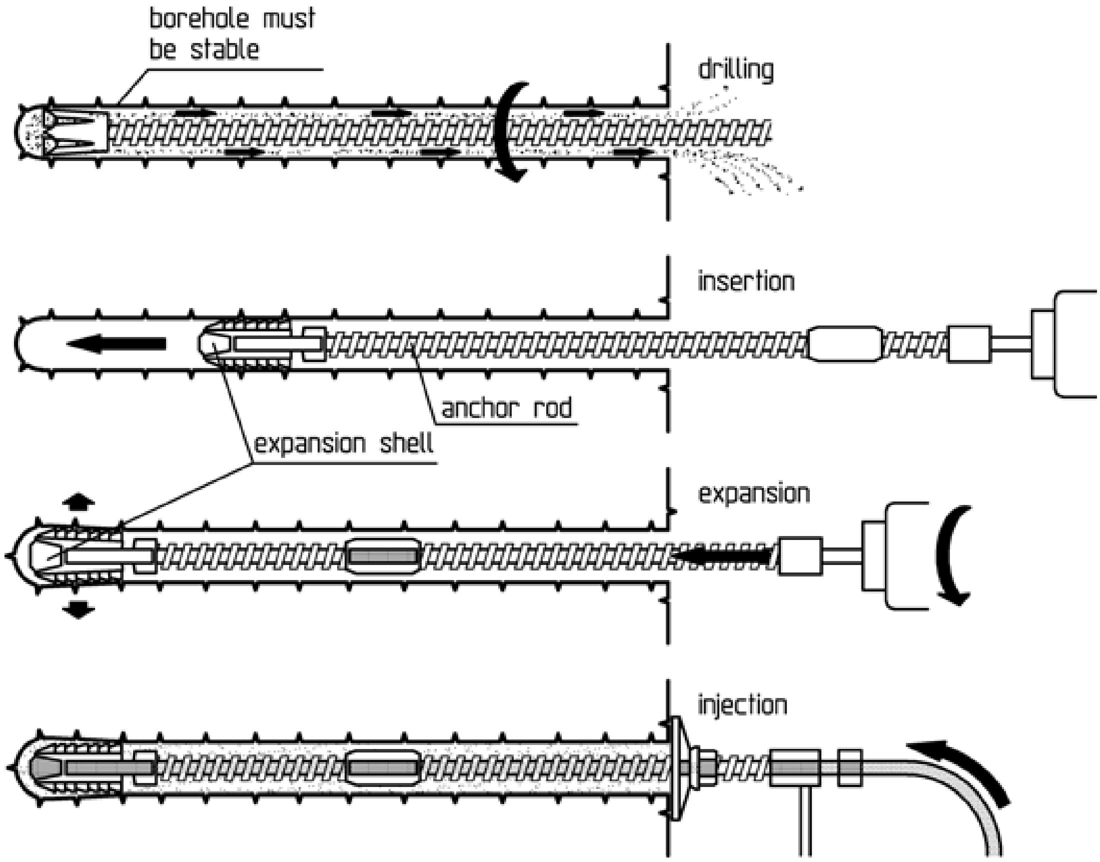


Fig. 2.25: Installation of an expansion anchor constructed as an injection bolt (Ischebeck Titan Ltd.)

In tunneling, non-prestressed (untensioned) anchors are generally installed as systematic anchoring for the excavation support in a raster determined on the basis of statical criteria. Anchor lengths between 3 and 6 m are common. Using sleeve connections, anchor lengths up to approx. 18 m are feasible. The anchors basically consist of an anchor rod and an anchor head with an anchor plate.

Anchor manufacturers offer anchor rods made from steel and glass fiber reinforced synthetics with different strengths and cross-sections (plain section, pipe cross-section). The advantages of glass fiber anchors over steel anchors lie mainly in the fact that they can be cut as well as bent. Glass fiber anchors are therefore often installed at locations where they have to be removed in the course of further excavation, e. g. for the support of the inner walls during sidewall adit heading or for the support of the tunnel face. Disadvantages of glass fiber anchors are the facts that they can carry only very small shear forces and that the transfer of point loads into the anchor at the anchor head is difficult to enable with the anchor design.

The anchor heads should always be constructed in such a way that the anchor plates lie flat on the excavation surface or on the shotcrete and that the load transfer from the anchor plate to the anchor rod does not lead to bending or shear loading of the anchor rod. Therefore, mainly so-called sphere cap anchor plates are used that are spherically shaped around the hole. For the nuts screwed onto the anchor rods, so-called sphere cap nuts are used, which have a spherical surface in the contact area with the anchor plate. The anchor plates should have minimum dimensions of 150 x 150 x 10 mm.

Special anchor head designs have been developed for the use of non-prestressed anchors in squeezing rock. If a certain anchor load is reached, these anchor heads yield, thus avoiding over-stressing of the anchors. Since these anchor types are special-purpose designs adjusted to the individual case, they will not be dealt with here in more detail.

The admissible anchor force is the maximum force the rock bolt is permitted to be subjected to (maximum tensile force, maximum bond force or limit creep force) divided by a factor of safety. The maximum tensile force of the anchor rod is calculated according to

DIN 21521 from the relevant cross-section in connection with the design strain limit and the design tension yield stress of the used material, respectively. The maximum bond force between the anchor rod and the rock mass must be determined for the individual case by pull tests according to DIN 21521. The limit creep force is the force which leads to the chosen creep rate according to DIN 4125 in the pull test.

For glass fiber anchors, attention must be paid to the fact that the failure load of the anchor rods is generally far higher than the failure load of the anchor head parts.

2.3.2 Load-carrying behavior

In tunneling, rock bolts are installed as single anchors to support individual rock wedges susceptible to sliding, as surface anchoring to support the excavation surface (tunnel walls, tunnel face), and as systematic anchoring to improve the load-carrying capacity of the rock mass. Surface and systematic anchorings furthermore serve the purposes of supporting regions prone to collapse and of improving the load transfer between the steel sets and the shotcrete on the one hand and the rock mass on the other. For systematic anchorings mostly fully cemented anchors (SN-anchors) or injection drill bolts (IBO-bolts) are used.

Rock bolts must carry tensile and possibly shear forces. Failure of the anchors due to overstressing may occur. The failure mode of bond anchors depends on whether they are placed in rock or in soil. In rock, generally the anchor rod breaks before the bond fails. In soil, failure of the mortar over the bond length occurs first.

Systematic investigations into the mechanism of operation and the load-carrying capacity of cemented steel anchors were first carried out by Bjurström (1974). These investigations led to formulae for the shear resistance of inclined and not inclined anchors. Further relations for the shear resistance were provided by Azuar and Panet (1980). Empirical equations for the shear and tensile resistance were developed by Dight (1983). Spang and Egger (1989) provide formulae to determine the shear resistance contribution T_0 by which the shear resistance of a discontinuity increases as a consequence of anchoring. This contribution can be converted into an "anchor cohesion"

$$c_A = \frac{T_0}{A} \tag{2.1}$$

by which the cohesion of the rock mass increases due to a systematic anchoring. In (2.1) A is the effective area per anchor of a systematic anchoring. Since glass fiber anchors only have very little shear resistance, they are not suited for a systematic anchoring to increase the rock mass strength.

Equation (2.1) makes it possible to model a systematic anchoring in FE-analyses by an increase in the cohesion of the rock mass. As an alternative, individual anchors can be modeled by truss elements, which, however, can only transfer axial forces. The combined tension and shear loading of cemented anchors can therefore not be captured by the common truss elements.

Erichsen and Keddi (1990) and Keddi (1992) report on numerical analyses of the influence of anchor design and bond between rock bolt and rock mass on the load-carrying behavior of cemented anchors.

2.4 Advance support

2.4.1 Spiles

In loose rock, steel spiles are used as advance support of the workspace at the tunnel face in order to limit overbreak and to protect the miners against falling rock. The spiles are arranged in the roof area of the tunnel approx. parallel to the tunnel axis in the form of a fan. They are installed before the underlying round is excavated. The length of the spiles should be at least three times the round length in order to enable sufficient overlap. The spacing between the spiles should not exceed 30 cm. Fig. 2.26 shows exemplarily the advance support using spiles at the sidewall adit excavation of the Limburg Tunnel of the new railway line Cologne – Rhine/Main in longitudinal section and cross-section.

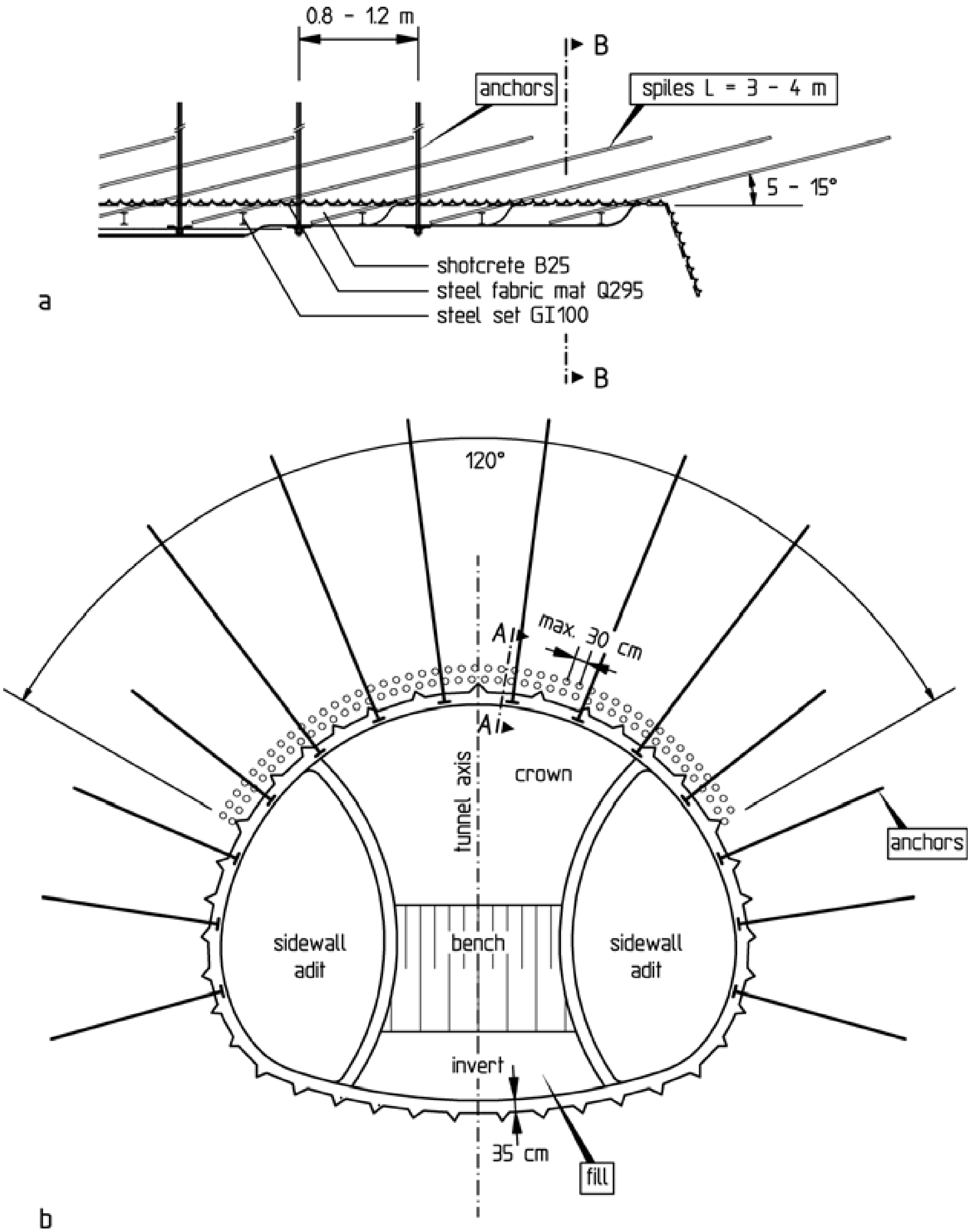


Fig. 2.26: Advance support using spiles (spile umbrella Limburg Tunnel, new railway line Cologne – Rhine/Main): a) Longitudinal section A-A; b) cross-section B-B

One distinguishes mortar spiles, driven spiles and injection drill spiles.

Mortar spiles made from rebars are installed with lengths of 3 to 6 m. They are designed for a rock mass in which the drillholes are stable without a casing. The rebars (∅ 25 – 28 mm) are installed in mortar-filled drillholes. By the use of a accelerating admixture or by choosing an appropriate time of installation, respectively, it is ensured that the mortar achieves sufficient strength by the beginning of the following excavation works. Fig. 2.27 shows mortar spiles that were blasted free during excavation. In this case they do not fulfill their original purpose, which is to limit overbreak and to protect the workspace against falling rock. The blasting was not carried out smoothly with respect to the rock.

Fig. 2.27: Mortar spiles with lattice girder, blasted free

Driven spiles consist of steel pipes 3 to 6 m long, which are driven into pre-drilled holes. The diameter of the steel pipes is slightly larger than the diameter of the drillholes. If the steel pipes are designed correspondingly and the rock mass is groutable, a later rock mass improvement using injections is possible. Fig. 2.28 shows driven steel pipe spiles installed as advance support in the Tunnel Deesener Wald of the new railway line Cologne – Rhine/Main.

Fig. 2.28: Driven steel pipe spiles (Tunnel Deesener Wald, new railway line Cologne – Rhine/Main)

Injection drill spiles (IBO-spiles) are installed with a length of 4 to 12 m. They are used in rock mass conditions where stable holes cannot be drilled and/or the rock is to be strengthened by grouting with cement suspension at the same time. Depending on the diameter of the anchor rod, the holes for the spiles are drilled with a diameter ranging from 42 to 76 mm. The spiles consist of steel pipes with a lost drill bit, which are used for flushing during drilling and for grouting the drillhole together with the surrounding rock mass after the planned depth is reached. Installation and grouting of injection drill spiles are carried out corresponding to Fig. 2.24.

2.4.2 Pipe umbrellas

If the spiles described in section 2.4.1 do not offer sufficient support, pipe umbrellas are used. They are applied mostly in cohesionless, loose ground if streets or structures are undercut with little cover. Just as spiles, pipe umbrellas are constructed over a certain part of the circumference of the excavation profile, preceding the excavation. Because of the dimensions of the pipes, pipe umbrellas are far stronger than spile umbrellas and extend further in advance of the excavation.

Instead of pipe umbrellas, composite pile and jet grouting umbrellas are also constructed in connection with the NATM for the advance support of the workspace at the tunnel face. In the following, only pipe umbrellas will be covered. Examples for composite pile umbrellas and jet grouting column are described in Chapters 3.2 and 7. The jet grouting technique is covered in detail in Chapter 7.

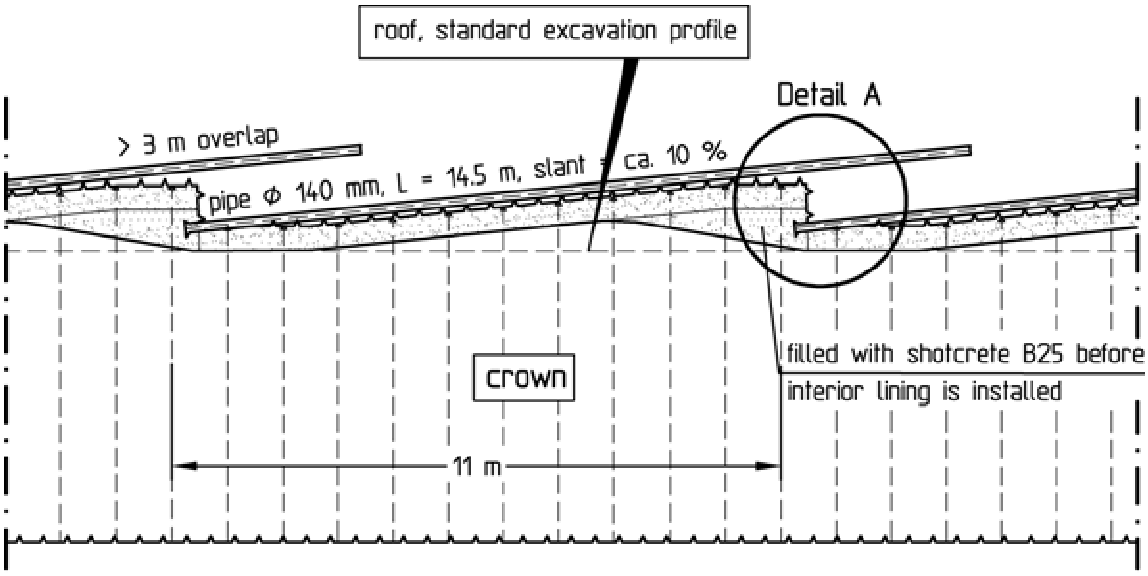
Two systems for the construction of pipe umbrellas are distinguished:

- Pipe umbrella with niches (Fig. 2.29),
- pipe umbrella without niches (Fig. 2.30).

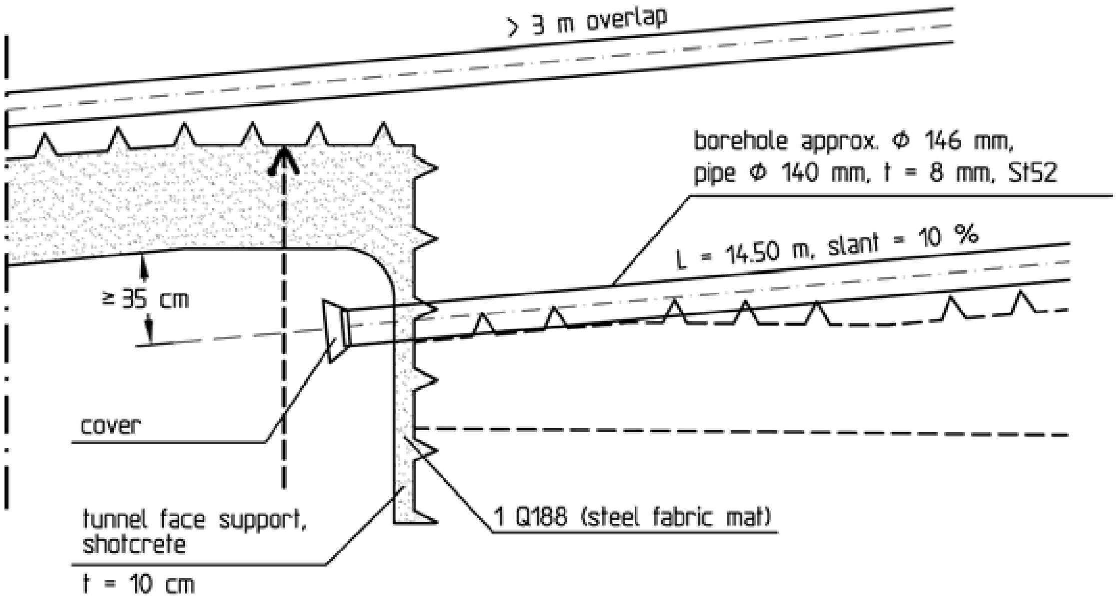
If the pipe umbrella is constructed from niches, the excavation profile of the tunnel is widened in the course of the excavation, so that the drill points for the steel pipes of an umbrella are located outside of the shotcrete membrane (Fig. 2.29). Thus, the geometry of the steel sets must be adapted for each round and the niches must at least partially be filled with shotcrete before the sealing and the interior lining are installed.

If the pipe umbrella is constructed without niches, the drill points for the steel pipes are located in the tunnel face of the standard excavation profile (Fig. 2.30). In the course of further excavation, the pipe sections lying within the cross-section of the shotcrete membrane must be cut off after each round.

Pipe umbrellas are constructed with lengths of ca. 15 to 30 m with a heavy drill rig (e. g. drill carriage, Fig. 2.31). The outer diameter of the steel pipes used varies between 76 mm and 200 mm, while the wall thickness of the pipes ranges from 8 mm to 25 mm. The pipes serves as the casing while the boreholes are drilled, and remain in the soil or rock, respectively, as a structural element of the pipe umbrella. After the installation of the pipes, the annular gap between the pipes and the rock mass and as far as possible also the surrounding rock mass itself are grouted with cement suspension from out of the pipes in order to improve the support effect of the pipe umbrella. To this end, the pipes are provided with injection valves at a spacing of 0.5 to 1 m (Fig. 2.32). The pipes are subsequently closed with a cover and filled with suspension or mortar.



a



b

Fig. 2.29: Pipe umbrella with niche (Niedernhausen Tunnel, new railway line Cologne – Rhine/Main): a) Longitudinal section; b) Detail A

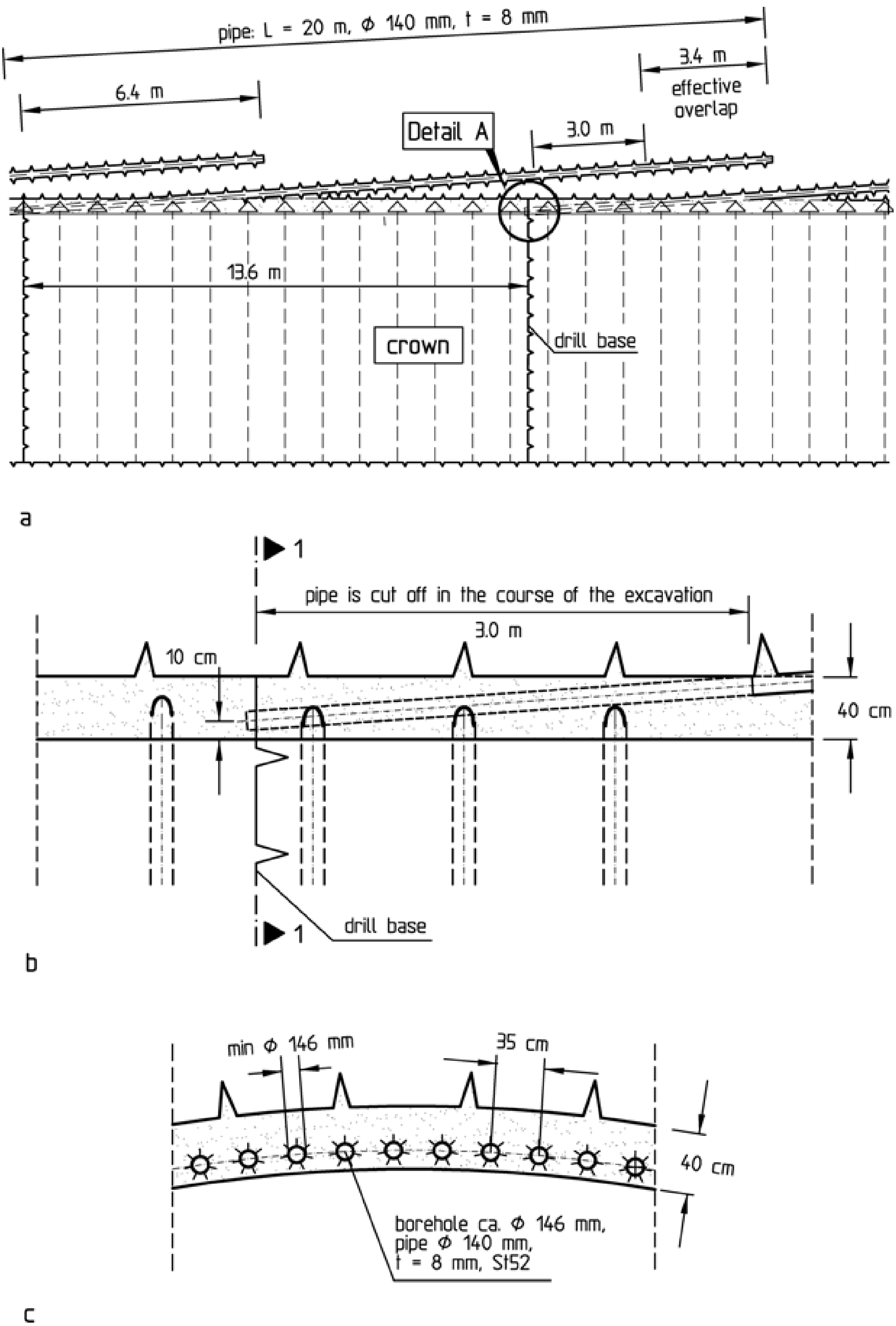


Fig. 2.30: Pipe umbrella without niche (Dernbach Tunnel, new railway line Cologne – Rhine/Main): a) Longitudinal section; b) Detail A; c) Detail section 1-1

a

b

Fig. 2.31: Pipe umbrella in the Dernbach Tunnel (new railway line Cologne – Rhine/Main): a) Pipe umbrella; b) installation of a pipe

Fig. 2.32: Pipe with injection valve

The boreholes for the pipe umbrella are arranged over a certain portion of the circumference of the excavation profile at a spacing of 30 to 50 cm. They are drilled ascending at an angle of approx. 5° with respect to the tunnel axis. The pipes of two pipe umbrellas should overlap by at least 3 m in the longitudinal tunnel direction (Fig. 2.29 and 2.30).

In soil and in weathered, soft rock the boreholes are generally drilled with an auger and air and/or water flushing (Fig. 2.33). In rock or for large pipe diameters, however, mostly down-hole hammers are used. Here, the drill bit and the pipe are either connected by a bayonet joint so that the casing is continuously pulled into the borehole with the advance of the drill pipe, or the pipes are pushed hydraulically by the drill rig. After the final depth is reached, the drill pipe and the drill bit are released from the pipe and pulled out.

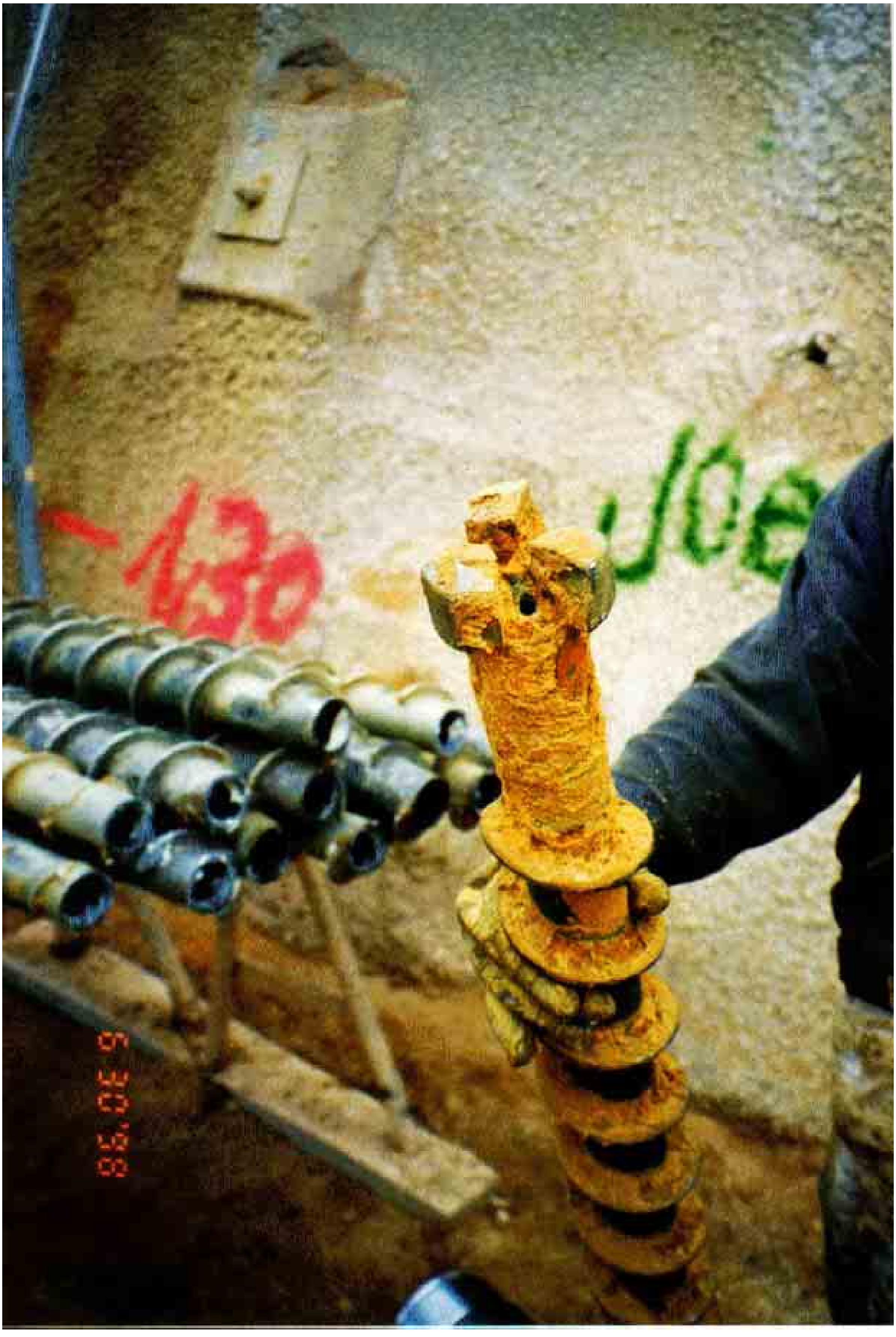


Fig. 2.33: Auger for the installation of pipes

Fig. 2.34: Construction of the pipe umbrella at the starting wall of the Dernbach Tunnel (new railway line Cologne – Rhine/Main)

Fig. 2.34 shows the construction of a pipe umbrella through the starting wall of the Dernbach Tunnel of the new railway line Cologne – Rhine/Main. The starting wall is supported by shotcrete and anchors. In this case, the shotcrete support of the starting wall was drilled through before the pipes for the pipe umbrella were installed.

2.5 Geotechnical mapping and monitoring

2.5.1 Mapping

Before the excavation of a tunnel, in general only surface exposures and boreholes are available to assess the ground. Their results must be extrapolated to the planned tunnel and its surrounding area. Because of the ensuing uncertainties in assessing the ground conditions, mapping of the temporary tunnel face during construction is absolutely necessary. They form an essential basis for adapting the support to the local conditions.

The extent of a geotechnical mapping of the tunnel face depends essentially on the available time. A "detailed mapping" including comprehensive information on the intact rock, the fabric and the groundwater can usually only be carried out if the tunnel excavation is halted. With routine mappings during the excavation, it is in general only possible to record these properties randomly. In this case, however, it is important to recognize and document deviations or changes in the local conditions.

In a detailed mapping, the following properties should be recorded as far as possible (Wittke, 1990):

- Rock types and rock boundaries (boundaries of layers),
- degree of weathering of the intact rock,
- orientation of the discontinuities (strike and dip angle),
- location, spacing and trace lengths of discontinuities, i. e. of the intersections of the discontinuities with the excavated rock surface,
- location and shape of visible discontinuities lying at the rock surface,

- opening widths of open or filled discontinuities,

- fillings and coatings of discontinuities,

- location, dimensions and properties of large discontinuities and faults,

- location of seepage water and quantity of discharge.

Apart from these information, the field protocol should include further peculiarities, such as for example information on discontinuities which tracings are so closely spaced that they cannot be recorded individually for reasons of time alone. Half-quantitative descriptions such as "strongly jointed" or "strongly fractured" are sufficient in this context. These descriptions should be complemented with information on the magnitude of the spacing in cm or dm. Further, recordings of the unevenness and roughness of the discontinuities and of possible indications of weathering on the discontinuity surfaces should be made. The discontinuity fillings just as bedding parallel and foliation parallel discontinuities can be labeled by special signatures (Wittke, 1990).

During mapping, samples for complementing laboratory tests must be taken, if indicated.

The standard mapping gear consists of

- a geological compass to measure the orientation of discontinuities,

- a measuring tape to record the location of the outcrops and the tracings of discontinuities,

- a rock hammer to make discontinuities visible that are e. g. covered by a thin weathered layer, to loosen rock samples or to sound the rock (clear or dull sound),

- a note pad in which a sketch of the mapping is entered. It is more appropriate, however, in most cases to use prepared forms for the mapping (Fig. 2.35).

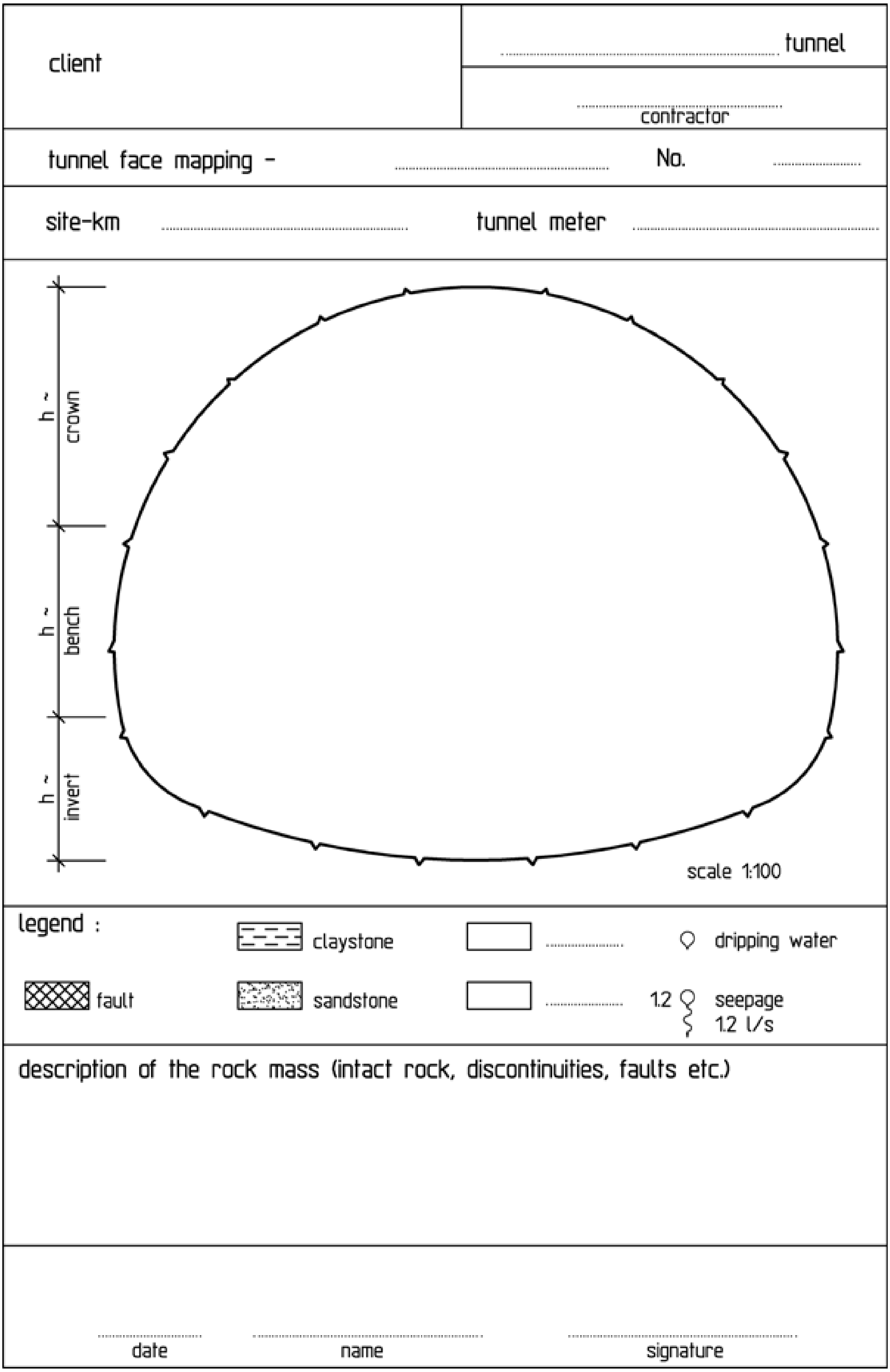


Fig. 2.35: Example of a tunnel face mapping form

Fig. 2.36 shows an example of a detailed mapping of a tunnel face. For reasons of clarity, the information on the orientation and nature of the discontinuities is omitted here.

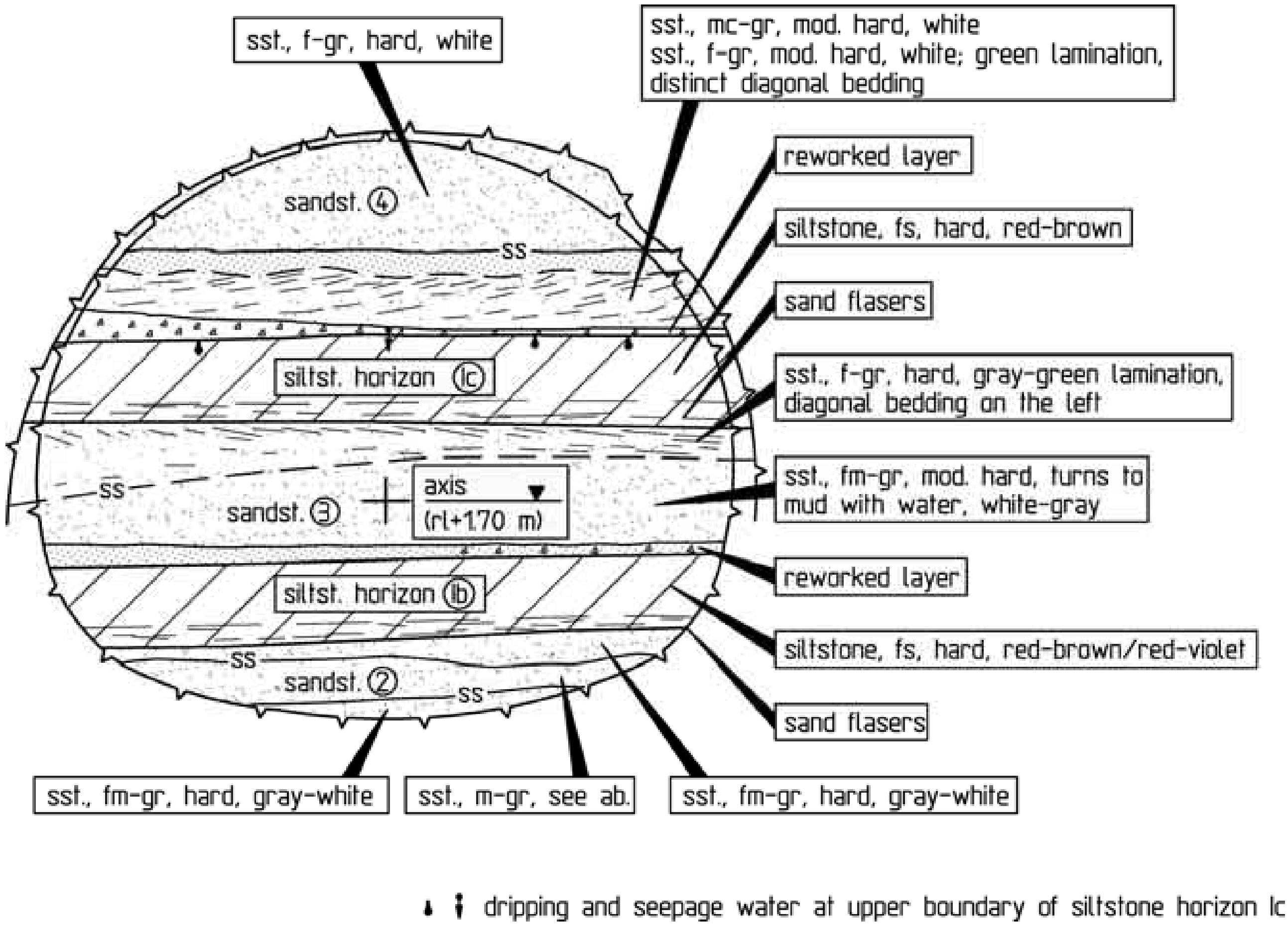


Fig. 2.36: Detail mapping of a tunnel face (Hasenberg Tunnel, Stuttgart urban railway)

Examples for mappings during excavation are given in Chapters 3.1, 3.3 and 4.2 (see Fig. 3.20, 3.41 and 4.43).

Discontinuity orientations are measured using the geological compass (Fig. 2.37). The dip direction α_D is read from the compass circle, while the dip angle β is read from the vertical circle. The dip direction α_D is the angle between the projection of the dip line of the discontinuity onto the horizontal plane and north. α_D is correlated with the strike angle α by the relation given in Fig. 2.37 (Wittke, 1990).

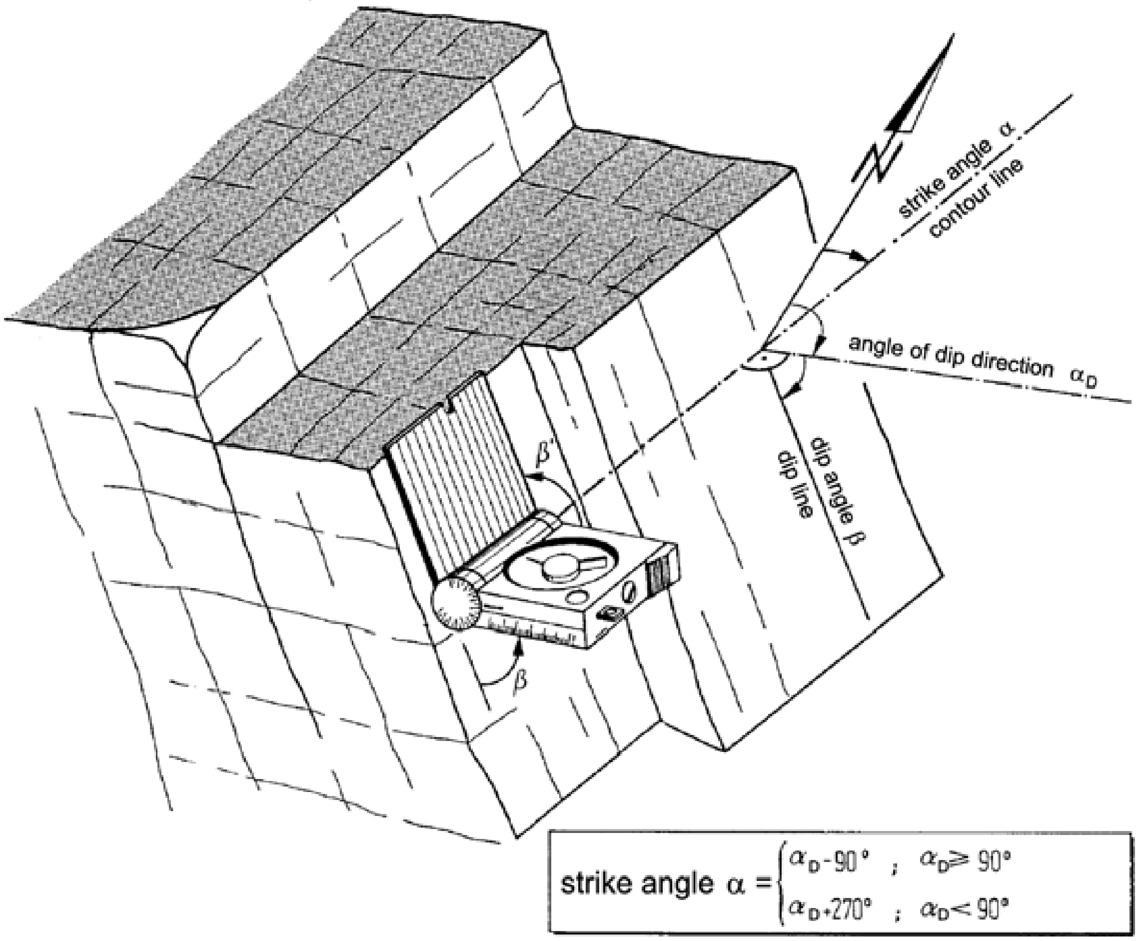


Fig. 2.37: Measuring the orientation of a discontinuity using the geological compass (Wittke, 1990)

The orientation of a discontinuity, given by the angles α_D and β or α and β, respectively, can be represented in a polar equal area net by a point, the so-called pole. A polar equal area net is an equal-area hemispheric projection of the lower half of the reference sphere. The term reference sphere denotes a sphere the equator of which is located in the horizontal plane. The pole is the projection of the point of intersection of the normal to the discontinuity through the center of the reference sphere onto the polar equal-area net (Wittke, 1990).

Entering the discontinuity orientations measured during one or several mappings in the polar equal-area net leads to a so-called polar diagram. An example of a polar diagram prepared in this way and for the grouping of the measured discontinuities into several sets is shown in Fig. 3.42 (Chapter 3.3.6).

For a great number of measured discontinuity orientations, or if the measured discontinuity orientations scatter considerably, a statistical evaluation of the discontinuity orientations may be appropriate. To this end, instead of the poles, areas representing the angular regions of equal relative frequency of the poles are entered in the polar diagram. These areas are assigned to pole densities. The pole density denotes the number of poles located on 1 % of the area of the polar equal area net, relative to the total number of measured discontinuity orientations, in percent (Wittke 1990). In this way, the areas with the most frequent orientations can be identified for the individual discontinuity sets. An example of the representation of discontinuity orientations by areas of equal pole density is shown in Fig. 5.39 (Chapter 5.2.6).

The mapping results should be summarized for homogeneous areas in a geometric structural model. To this end, first the parameters describing the geometry of the discontinuity fabric are determined on the basis of a statistical evaluation of the measured fabric data. For each discontinuity set, the mean values of the measured dip and strike angles, of the spacing and, if possible, of the tracings of the discontinuities must be specified. However, since especially the spacing and tracings of discontinuities scatter considerably, in addition to the mean values the standard deviations of these parameters should be specified as well (Wittke, 1990).

In addition to these data, a structural model should also include a description of the properties of individual discontinuities, which form the sets. This description should contain information on the unevenness and the roughness as well as on the degree of weathering of the discontinuity surfaces, which can be obtained e. g. from the mapping of profiles. Further, it should be noted whether the discontinuities are closed or open, and whether and to which degree they are filled.

Inhomogeneities such as discontinuities with an extent reaching the magnitude of the dimensions of the tunnel must be described individually with respect to their location and orientation.

Finally, the model should be illustrated in a block figure containing as many as possible of the referenced parameters (Fig. 2.38).

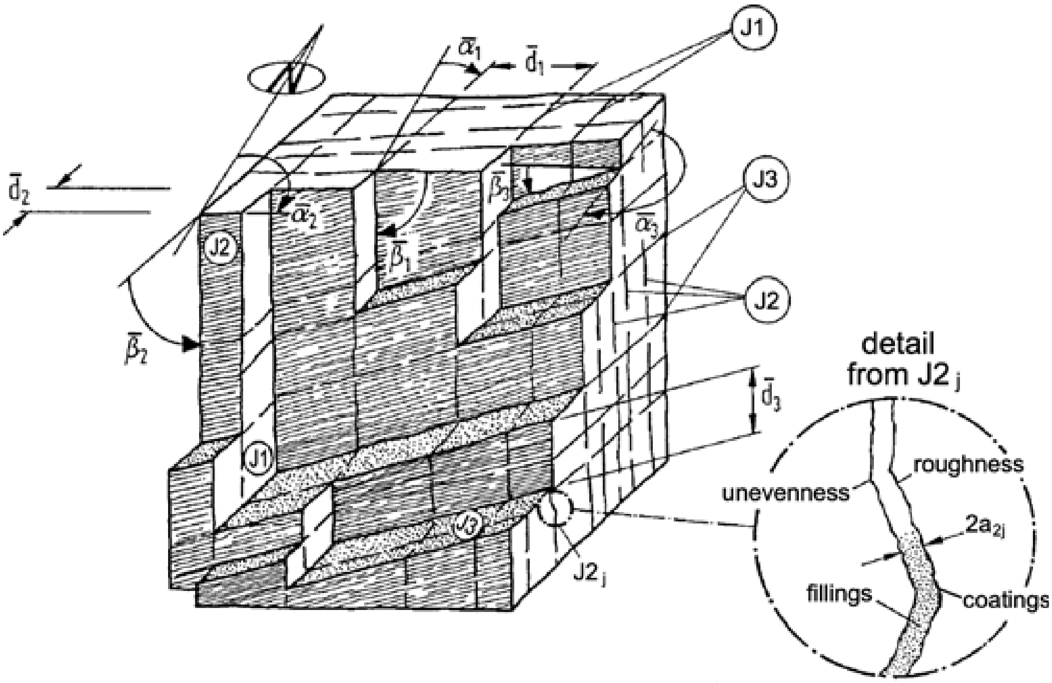


Fig. 2.38: Example of a structural model of a rock mass with three discontinuity sets (Wittke, 1990)

2.5.2 Monitoring

General

Just as geotechnical mapping, geotechnical measurements represent an essential element of the NATM. On the one hand, they serve to monitor

- the stability of the tunnel and of adjacent structures,
- the deformations in the ground and the displacements on the surface,
- the loading of the shotcrete membrane,
- the vibrations during heading.

On the other hand, just as the mapping, they form one of the foundations for adapting the support measures to the local ground conditions. Finally, the interpretation of already existing measurement results can and should be used to verify and, if indicated, optimize the dimensioning of the temporary and permanent lining. Stability analyses on the basis of measurement results furthermore permit to reduce or remove the uncertainties associated with the assumption of the parameters (back analysis) and to capture the influences from actual construction.

Geotechnical monitoring comprises:

- Positional surveying,
- leveling,
- convergency measurements,
- extensometer measurements,
- inclinometer measurements,
- stress measurements,
- anchor force measurements,
- vibration measurements,
- water level and water pressure measurements.

Positional surveying and leveling

Displacements of points located at the ground surface and at the tunnel contour (Fig. 2.39) are measured geodetically by leveling and by using a tachymeter.

To measure subsidence, measuring cross-sections with several leveling points are installed perpendicular to the tunnel axis (Fig. 2.39). In sloping locations the horizontal displacement components should be measured as well. The zero reading should be carried out when the tunnel face is still several tunnel diameters away from the respective measuring cross-section to capture the subsidence due to tunneling completely. If settlements are to be expected resulting from a lowering of the groundwater table due to tunneling, the zero reading should be carried out before the start of construction.

For the measurement of the displacement of points on the tunnel contour, the measuring locations are installed closely (≤ 1 m) behind the tunnel face. The measuring points are steel bolts cemented into the rock. Simple homing boards or triple prisms are mounted onto these bolts. The zero reading is carried out before the next round.

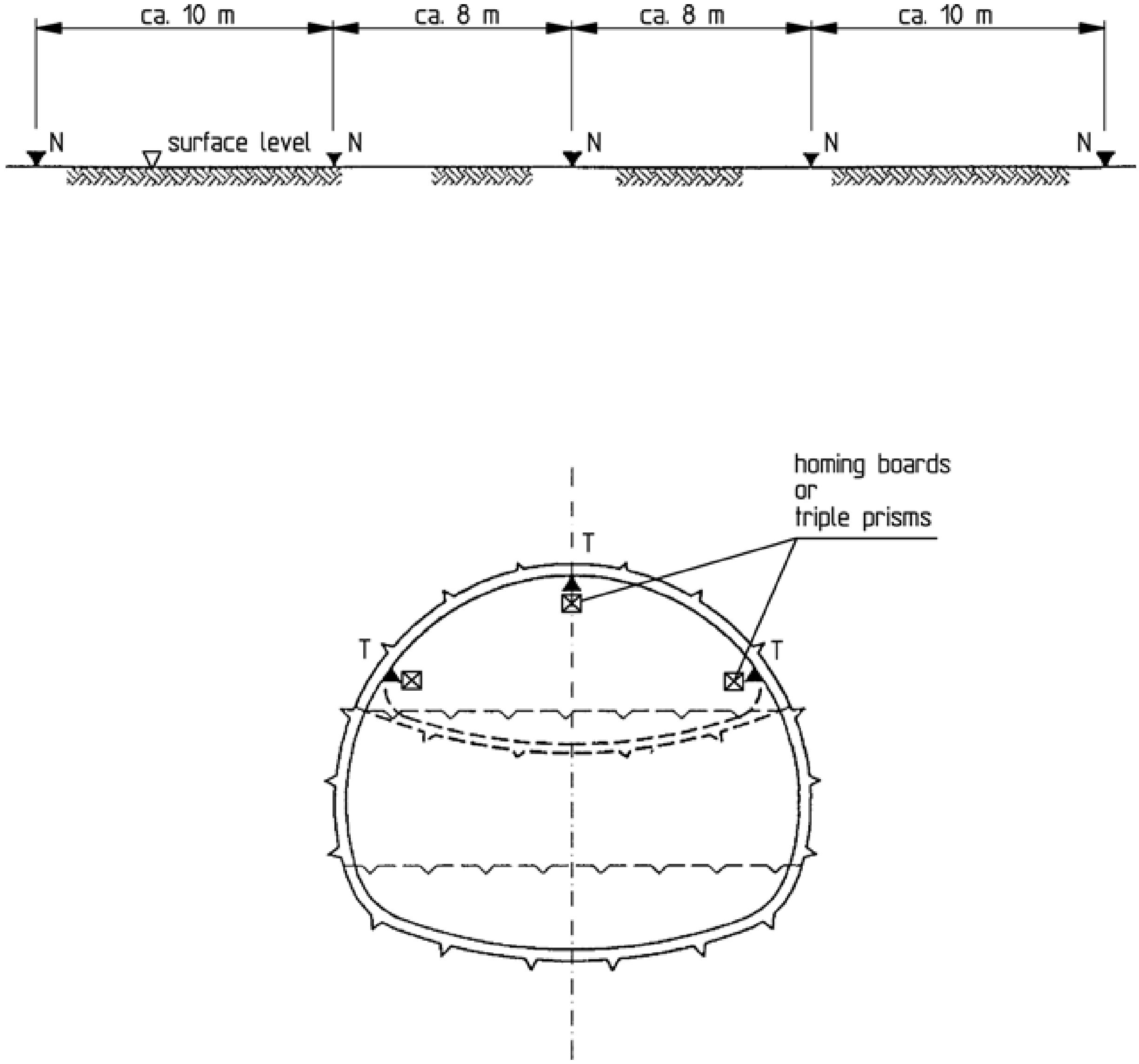


Fig. 2.39: Surface leveling perpendicular to the tunnel axis and positional survey of points on the tunnel contour

With the tachymeter, not only the vertical but also the horizontal displacement components parallel and perpendicular to the tunnel axis are measured. The measuring points are surveyed three-dimensionally from bench marks by determining the direction, distance and inclination from the bench mark to the measuring point for different points in time. The measurement data given in polar coordinates are converted to Cartesian coordinates. From the differences in coordinates from two measurements, the respective displacement vectors of the measuring points can be computed (Reik and Völter, 1996).

Fig. 2.40 shows as an example the vertical displacements of measuring points on the tunnel contour during the crown heading of the southern tube of the Gäubahn Tunnel in Stuttgart at chainage 113 m (see Chapter 3.2). The upper part shows the vertical displacements of the measuring points versus time. In the lower part the advance of the tunnel face is shown versus time.

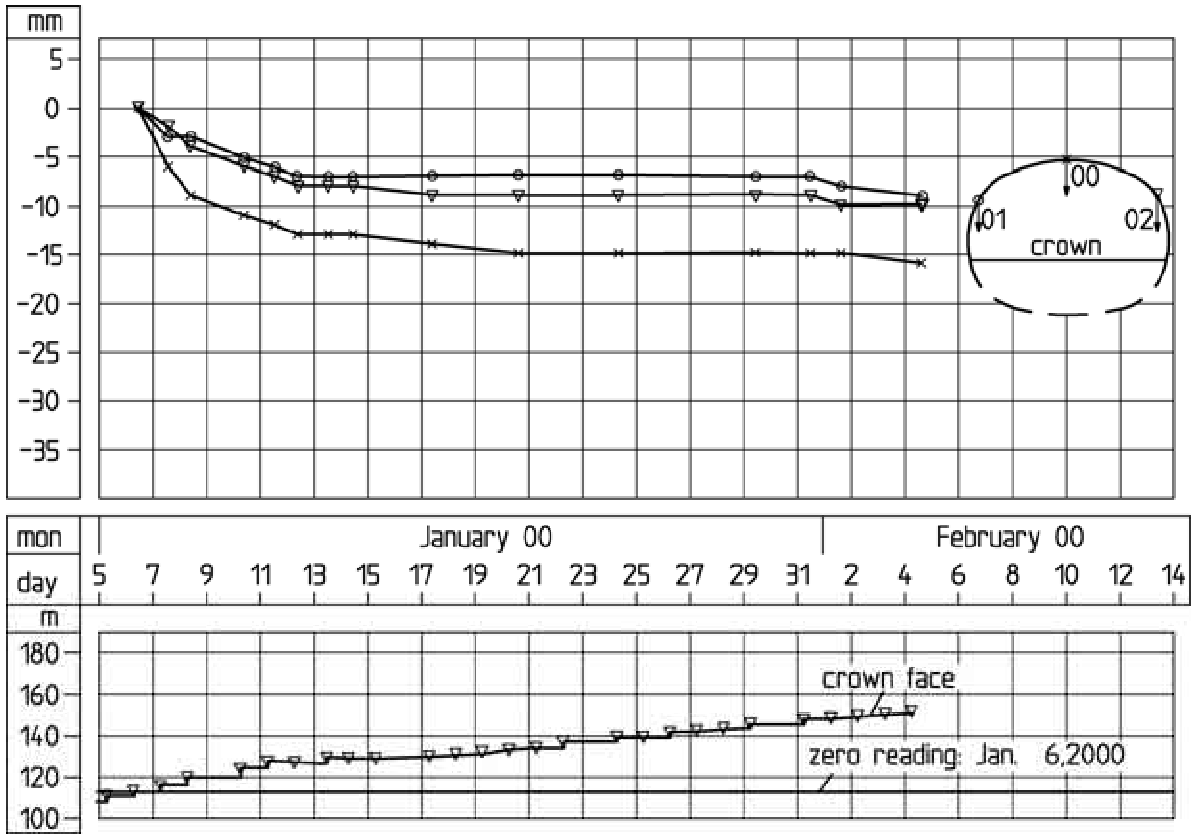


Fig. 2.40: Geodetically determined vertical displacements of measuring points on the tunnel contour during crown heading (Gäubahn Tunnel, southern tube, chainage 113 m)

Convergency measurements

Convergency devices allow the measurement of changes in distance of points on the tunnel contour.

Fig. 2.41 shows examples for the arrangement of convergency measurement sections for a crown heading and a sidewall adit heading. To measure the distance between two measuring points, the measuring points are connected by a tensioned measuring tape or a measuring wire and the convergency device (Fig. 2.42). The changes in distance can be determined as the difference of the measured lengths of consecutive measurements (Reik and Völter, 1996). Changes in length in the order of 0.1 mm can be registered with this technique.

Since convergency measurements interfere with tunnel heading, measurements with a convergency device are rarely carried out any more in tunneling nowadays. Optical three-dimensional measurements using a tachymeter and triple prisms are preferred instead (Fig. 2.39).

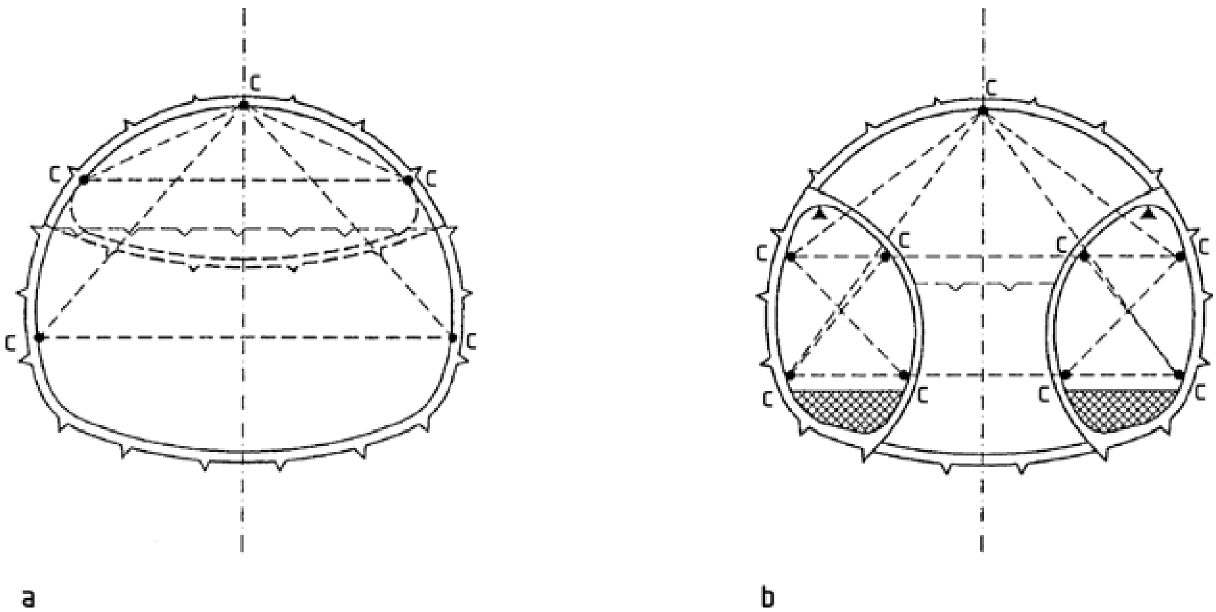
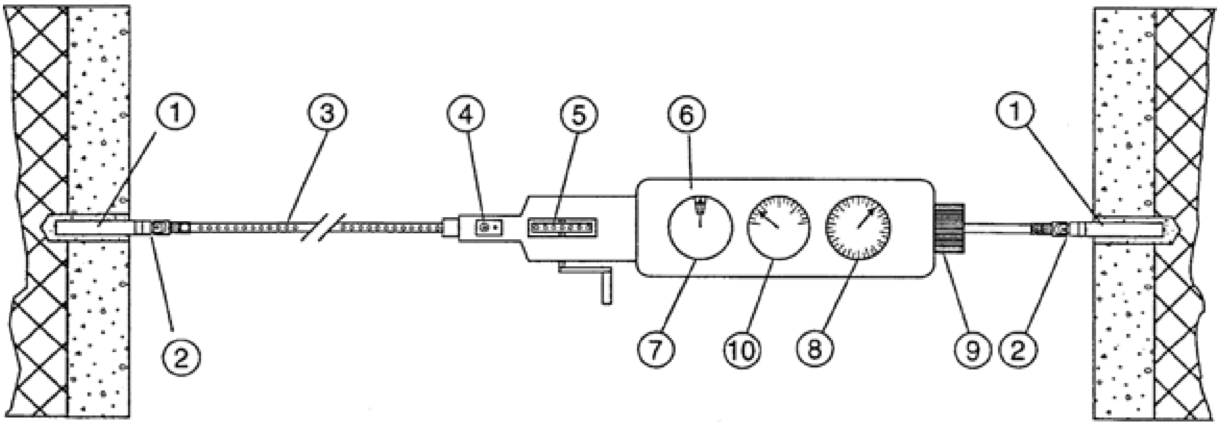


Fig. 2.41: Convergency measurement cross-sections: a) Crown heading; b) sidewall adit heading



1 Convergency gage bolt
2 connection elements with free rotation joint
3 Steel measuring tape (measuring wire)
4 Lock with pin for coarse length adjustment in the case of measurement devices with punched measuring tapes
5 Measuring tape reel (omitted in the case of measurements with prefabricated measurement wires)
6 Casing with linear gage and tensioning device
7 Tape tension indicator
8 Dial indicator for linear measurements
9 Fine length adjustment
10 Thermometer

Fig. 2.42: Schematic drawing of a convergency measuring system (Reik and Völter, 1996)

Extensometer and inclinometer measurements

Displacements and relative displacements (extension or compression) in the ground are measured in boreholes with stationary gages or probes.

Displacements parallel to the axis of a borehole are generally measured using extensometers (stationary devices) or sliding micrometers (borehole probes).

Multiple extensometers are suitable for the determination of relative displacements of measuring points with larger distances. Fig. 2.43 shows the setup of a multiple-rod extensometer. Absolute displacements can be determined by tying the extensometer head (1 in Fig. 2.43) in to a bench mark by leveling, or if the deepest anchor is a fixed point. Details on displacement measurements with extensometers are given e. g. in Recommendation No. 15 of the German Geotechnical Society (DGGT) (Paul and Gartung, 1991).

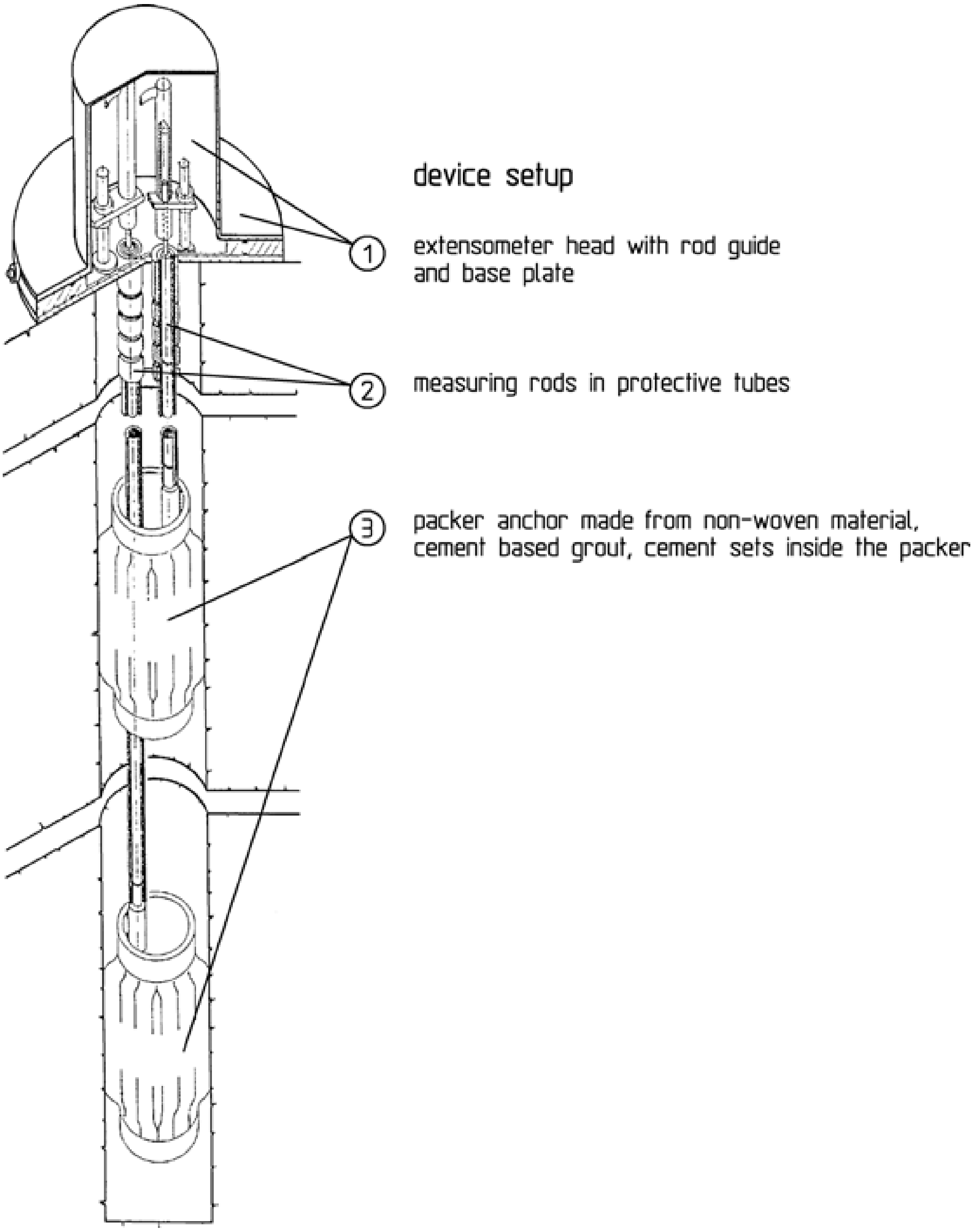


Fig. 2.43: Setup of a multiple-rod extensometer (Interfels, 2000)

With a sliding micrometer, rock displacements can be measured at short distances along the axis of a borehole. To this end, a plastic casing is cemented into a borehole ca. 100 mm in diameter. Measuring marks are fixed in this casing at a spacing of 1 m. Using an inserted probe, the changes in distance between the measuring marks are recorded one after the other (German Rail, Guideline 853, DB 1999).

Displacements perpendicular to the axis of a borehole are usually measured using inclinometer probes or stationary inclinometers.

In the case of the inclinometer probe, a plastic pipe with four guiding grooves is installed in a borehole ca. 100 mm in diameter. With a measuring probe, the changes in inclination with respect to the borehole axis and thus the displacements perpendicular to the borehole axis can be determined in two directions at right angles to each other (Fig. 2.44).

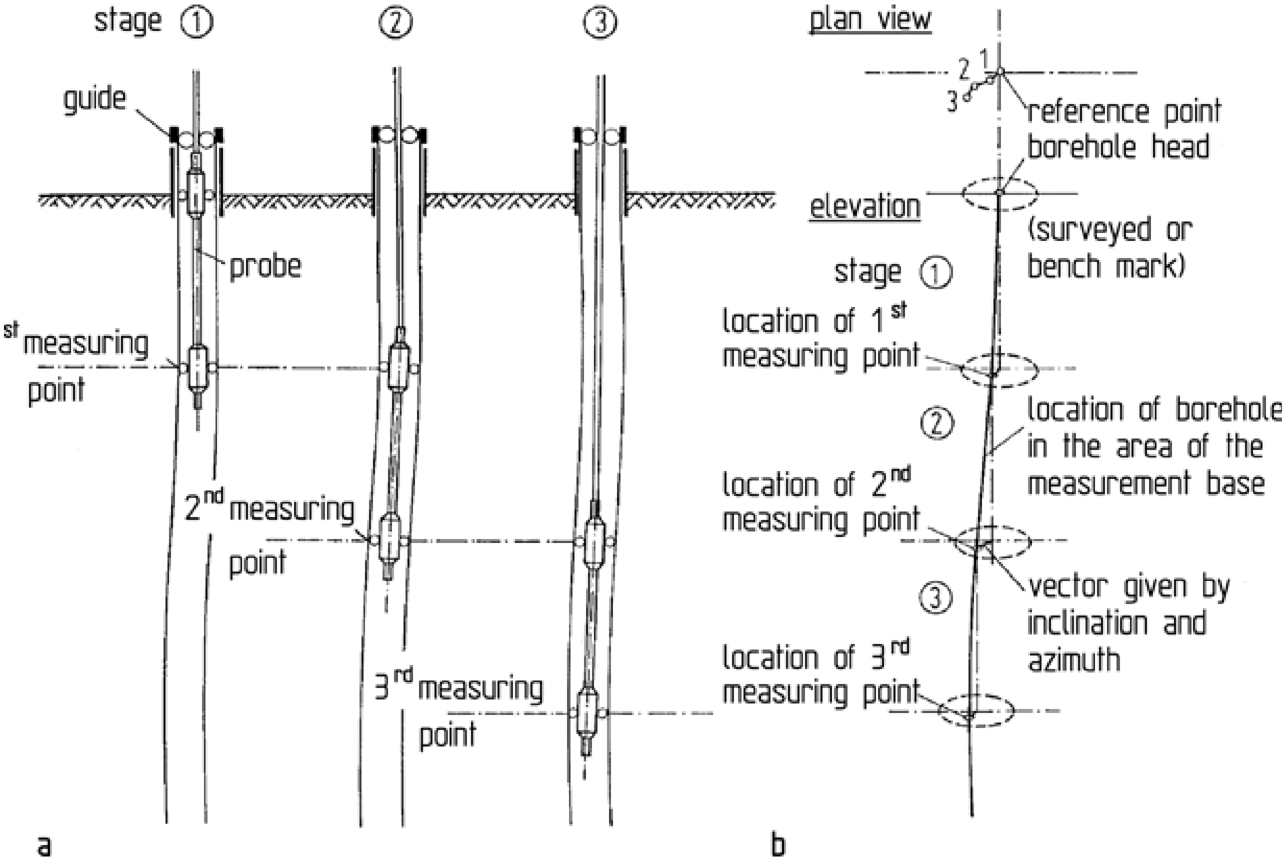


Fig. 2.44: Measuring principle of an inclinometer probe: a) Measurement; b) evaluation

Fig. 2.45 shows an example of the arrangement of the measuring points in the case of a combined extensometer and inclinometer measuring cross-section. Since during the undercrossing of the measuring cross-section by the tunnel none of the measuring points constitutes a fixed point, the measuring points on the ground surface are tied in to bench marks by levelings (L) and positional surveys to determine absolute displacements.

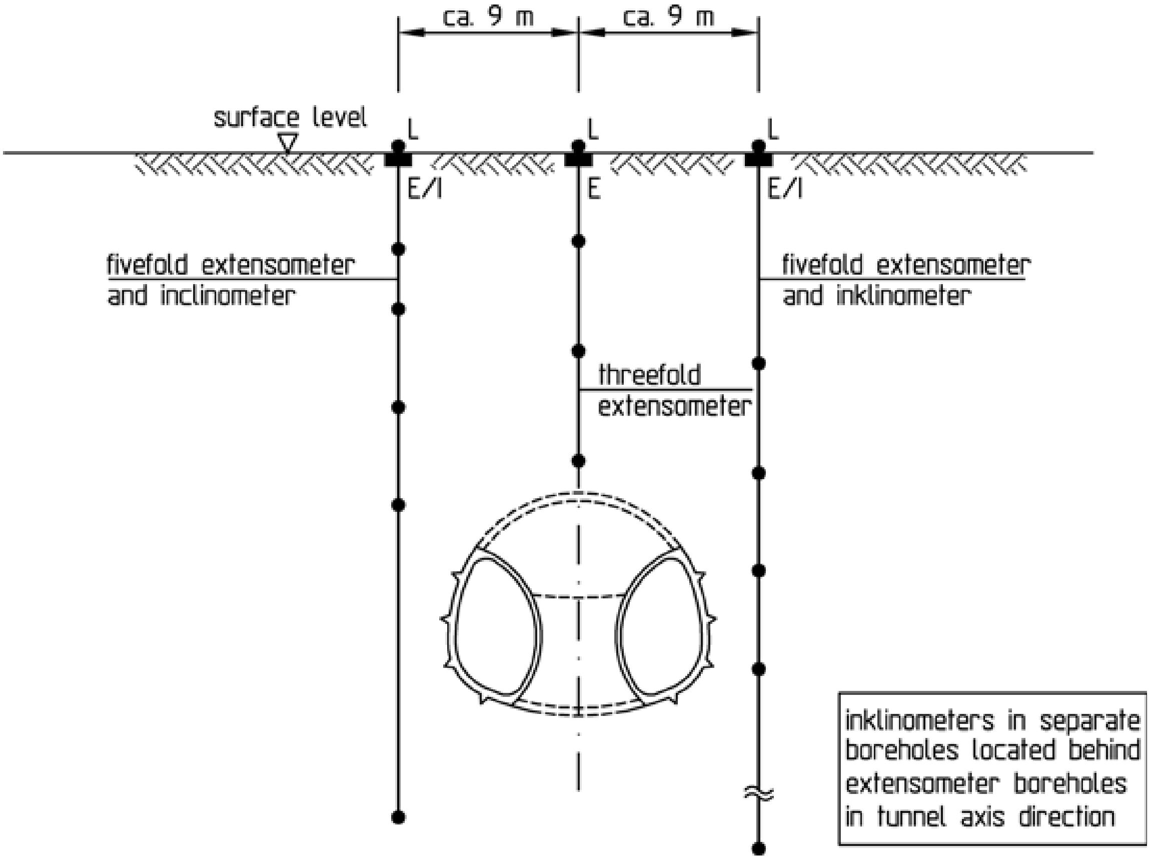


Fig. 2.45: Measuring cross-section with extensometers and inclinometers (constructed from the ground surface)

If the vertical as well as the horizontal displacements in longitudinal and transverse direction are to be determined in one borehole instead of separate boreholes, a device can be used that allows to measure the displacements in three directions perpendicular to each other at the same time. Such a device is e. g. the Trivec probe, a combination of sliding micrometer and inclinometer probe (German Rail, Guideline 853, DB 1999).

Just as for the positional surveys, the zero reading should be carried out as early as possible before the corresponding measuring cross-section is undercrossed.

Stress measurements

Stress measurements are carried out e. g. to assess the loading of shotcrete membranes and interior linings. Typical arrangements of pressure cells in a shotcrete membrane are sketched in Fig. 2.46. Here, one measuring cell is arranged tangentially on the outside of the shotcrete to measure the radial stress (rock mass pressure). The other, radially arranged measuring cell, however, serves to measure the tangential stress resulting from the loading (concrete pressure).

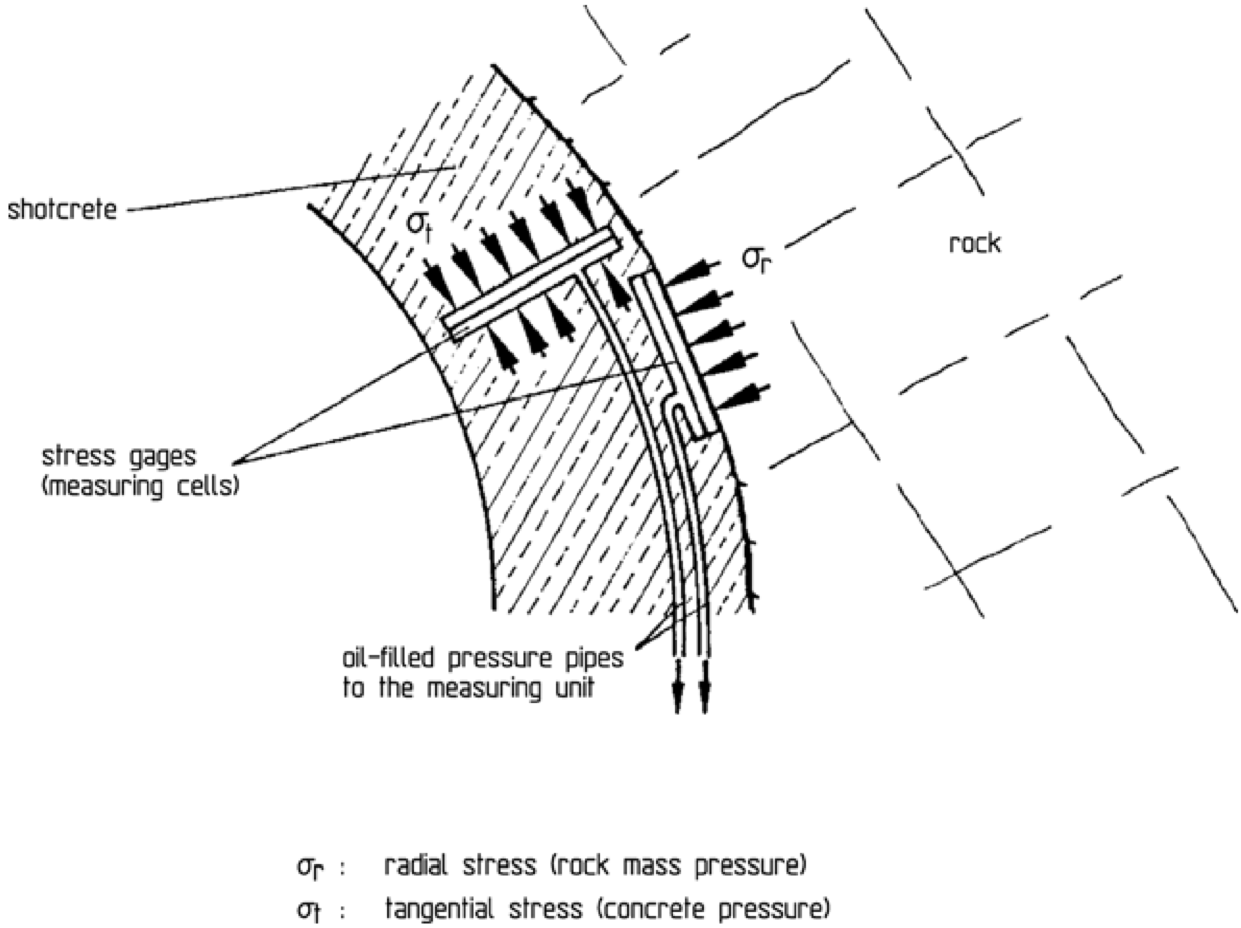


Fig. 2.46: Arrangement of pressure cells in a shotcrete membrane for the measurement of concrete and rock mass pressure (Wittke, 1990)

The stresses measured with the pressure cell arranged tangentially to the tunnel contours generally scatter significantly. On the one hand, this is due to the inhomogeneous stress distribution in the ground and the comparatively small dimensions of the measuring cells. On the other hand, the excavation process leads to local loosening along the tunnel contour. This exaggerates the non-uniform distribution of the radial stress, which is also referred to as rock mass stress or contact stress. Most of the times, therefore, radial stress measurements yield relevant results only in special cases (e. g. swelling rock).

For radially arranged measuring cells, the scatter of the measurement results is generally significantly smaller. The tangential stresses in the shotcrete measured with these cells scatter less strongly, since even an inhomogeneous radial loading of the lining results in a relatively even distribution of the normal thrust and thus also of the tangential stresses. In addition, the dimensions of the measuring cells are generally in the same order of magnitude compared to the thickness of a shotcrete membrane. Finally, the absolute value of the tangential stresses is in general many times greater than that of the radial loading of a shotcrete membrane. Nevertheless, the measured tangential stresses may also be non-uniform, if for example the shotcrete membrane has a greatly varying thickness as a consequence of an uneven excavation profile (Wittke, 1990).

Measurements of the tangential stresses in the lining and the radial stresses between lining and rock mass are carried out using hydraulic valve gages or oscillating chord gages.

The valve gage of Glötzl Co. (Fig. 2.47) is a hydraulic pressure cell, in which a compressive stress develops due to the loading of the oil-filled flat jack. The measurement is effected by increasing the fluid pressure in a circuit separate from the flat jack, until the fluid circulates back due to a slight deformation of a membrane located between an inner and an outer chamber. The reflux leads to a pressure drop at a pressure gage positioned in the circuit. The maximum stress measured before the pressure drop at the pressure gage can therefore be equated to the loading of the pressure cell (Wittke, 1990).

With oscillating chord gages, changes in length of a chord clamped freely oscillating in the gage are measured. These changes lead to a change in the natural frequency of the measuring chord excited to oscillate by a direct current impulse. The altered natural frequency measured while the oscillation of the chord fades out yields the chord strain. The change in stress is proportional to the measured change in length (Schuck and Fecker, 1997). Fig. 2.48 shows two oscillating chord gages of Geokon Co.

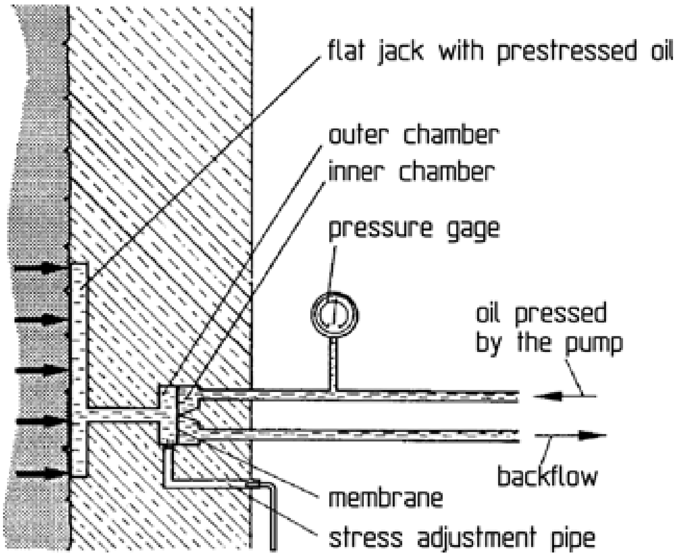


Fig. 2.47: Stress compensation measurement with pressure valve gage, system Glötzl

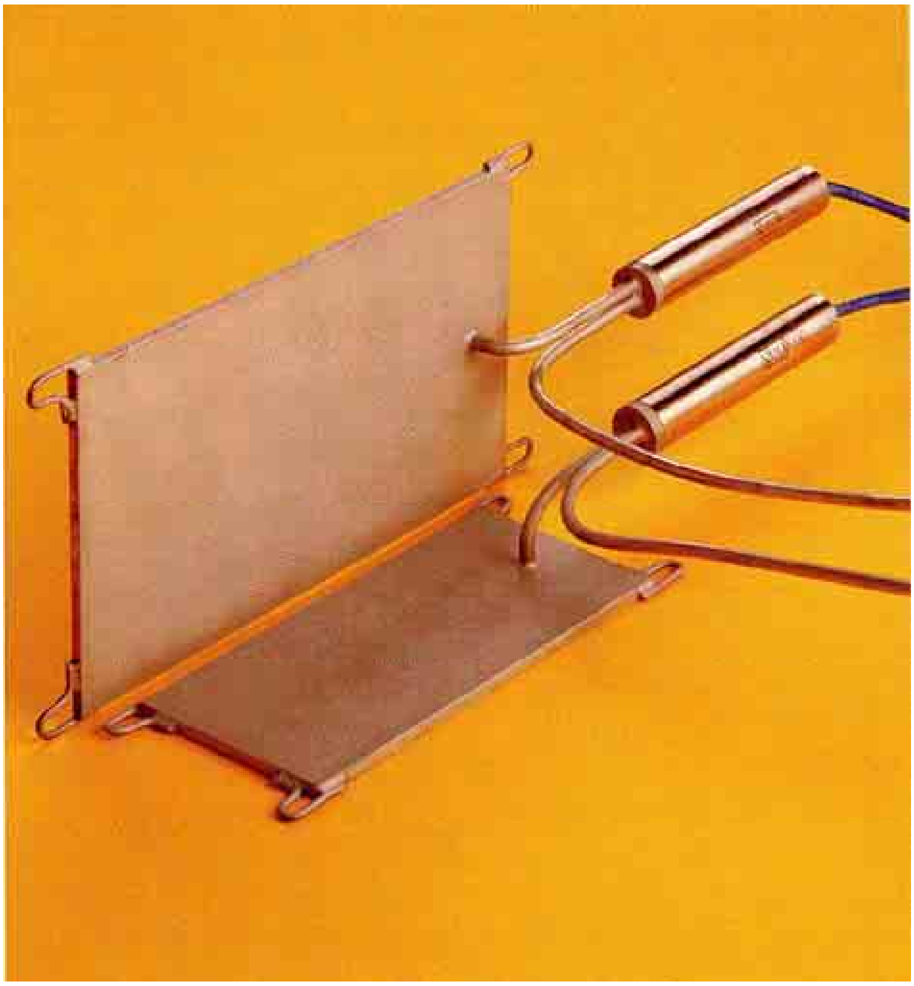


Fig. 2.48: Oscillating chord gages, system Geokon (Geokon, 1993)

If high stresses must be expected in the ground, the in-situ or primary stress state in the rock mass is determined. Aside from the already mentioned interpretation of displacement measurements using numerical analyses, which in general is only possible during construction, stresses can be measured by stress relief overcoring tests, also referred to as stress measurements by the overcoring method. In this method, which can only be applied to rock, the stress state in the rock mass is derived from the deformations of a drill core due to unloading. To this end, closed-form solutions for elastic isotropic and elastic anisotropic stress-strain behavior of the rock mass are applied (Kiehl and Pahl, 1990). Further, in-situ stresses can also be determined by the hydraulic fracturing method or the hard inclusion method in boreholes. Another possibility to measure stresses or stress changes are compensation measurements using flat jacks inserted in sawed or drilled slots.

Anchor force measurements

The anchor forces of untensioned anchors are measured in special cases only. In the case of tendons with a free anchor length, force measuring cells equipped with strain gages or oscillating chord gages can be installed at the anchor head. Lately, the technique of tension measurement with integrated optical fiber sensors has been developed that can be used also with fully cemented anchors.

Vibration measurements

In the case of a smooth blasting excavation, vibrations generally need not be considered with respect to the stability of tunnels. They may have an impact, however, on neighboring structures. Therefore, in these cases vibration velocity measurements are carried out to verify and ensure that the reference values according to DIN 4150 (Parts 2 and 3, 1999) are complied with. Compliance with these values is controlled primarily by limiting the maximum charge per ignition step in the case of a smooth blasting excavation (Wittke and Kiehl, 2001; DIN 4150, Part 1, 2001).