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PRESTRESSED PRESSURE TUNNELS

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Prestressed Concrete Pressure Tunnels

by P. Matt, F. Thurnherr and I. Uherkovich

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Prestressed concrete pressure tunnels

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Since the end of the Second World War, prestressed concrete has undergone a period of turbulent development, attributable predominantly to the intense activity in construction during this period and partly also to the change in the cost structure, which resulted in a shift from steel to prestressed concrete. In lining pressure tunnels and shafts, and in other types of construction, the tendency to move from steel to concrete can be seen. Projects constructed in Italy and Switzerland have recently demonstrated that prestressed concrete can be a complete substitute for the traditional steel lining.

THE TERM "pressure tunnel" in the broad sense includes all hollow spaces, aligned along an axis and surrounded by rock, which are suitable for pressurisation. In particular this category includes:

- horizontal or slightly inclined water pressure tunnels;
- vertical or inclined pressure shafts;
- pressure chambers or shafts of surge chambers; and,
- compressed air storage tunnels.

The main problems in the design and construction of pressure tunnels result less from their length or size than -om the requirement for watertightness and, dependent upon this, reliability in operation and economy in construction.

Where the lack of sufficient overburden does not permit the rock to accept the internal pressure in the tunnel, or where this pressure is so high that the watertightness is in doubt although the stability of the tunnel shell is not in question, a pressure tunnel must nevertheless be reinforced. Surge chambers, pressure tunnels and tailraces which are subjected to relatively low internal pressure, usually not much in excess of 1.5 MN/m^2, are eminently suitable for construction in post-tensioned concrete. Pressure shafts, which as a consequence of the high water head are subject to a considerable maximum service pressure, are usually lined with a steel tube, a considerable proportion of the internal pressure being transmitted to the rock by backing concrete.

Types of lining for pressure tunnels

In general, the type of lining will depend upon the nature of the rock. As a result, pressure tunnels usually are not uniform in construction over their entire length, but are of different forms of construction in different parts of their length. A distinction can be made in principle between the following types of lining:

- construction without prestressing; and,
- construction with prestressing.

Construction without prestressing- When the rock can respond elastically to all the loading conditions which occur and if, furthermore, it contains few fissures and is not very permeable, a carefully constructed unreinforced concrete lining of medium thickness is sufficient for a pressure tunnel. It is essential, however, that the voids which always occur between the concrete and the rock in the roof region be sealed by grouting.

Complete watertightness cannot be achieved with a simple concrete lining. Attempts to approach this ideal objective have resulted in the concrete lining being constructed in rings without any longitudinal joints in one operation. However, local leakage of water into the rock must always be expected; this leads to an additional relief of the load on the lining and therefore cracking frequently does not occur, although, according to theory, it should.

To provide increased assurance against cracking, a reinforced inner shell is sometimes chosen in addition to the concrete lining for a pressure tunnel; this inner shell is usually constructed of gunned concrete. The value of the reinforced inner shell is, however, not very high; the only effect of the reinforcement, as in most reinforced concrete structures, is to distribute the cracks and thus reduce the occurrence of single wide cracks.

A steel-plate lining, if correctly designed and constructed, constitutes a tension-resisting and watertight sheath to the tunnel. Its principal use is where the rock, because of lack of sufficient overburden, is not able to accept the internal pressure in the tunnel, or where this pressure is so high that-although the stability of the tunnel shell is not in question-other systems are not capable of guaranteeing the necessary watertightness. In such cases, in spite of the relatively high price of steel, this is the most suitable material to give a satisfactory engineering solution at an economic cost'.

Construction with prestressing. Basically, both unreinforced and reinforced concrete linings are quite unsuitable for one of the most important tasks, namely the prevention of water losses because of their low tensile strength. Recognition of this fact led soon to the

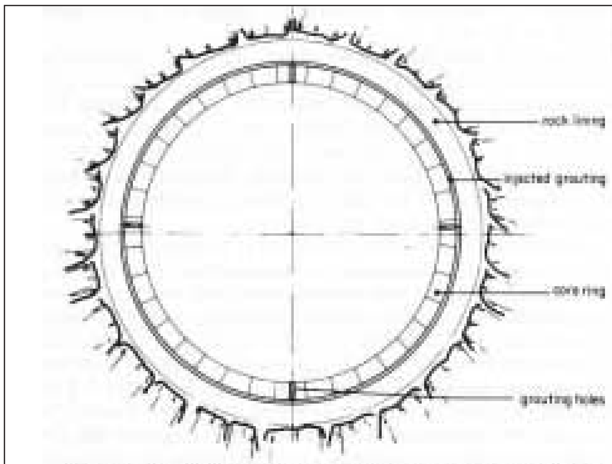


Fig 1. Core ring lining by the Kieser method. Construction with core ring of blocks.

* Losinger Ltd, 74 Konizstrasse, CH3001 Berne, Switzerland. Paper presented at the 8th FIP Congress, London. April 30-May 5, 1978.

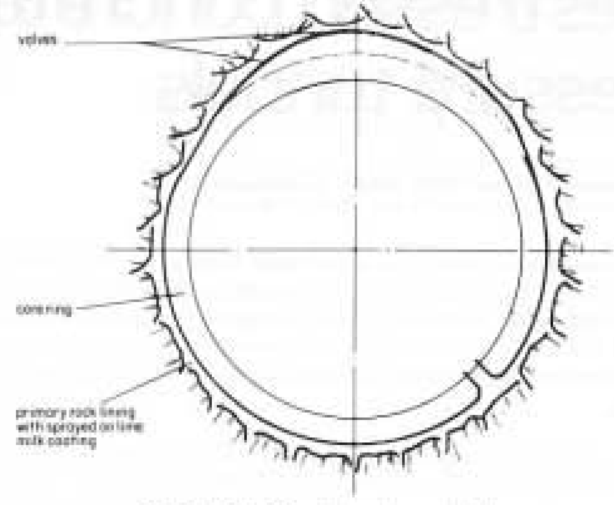


Fig. 2. TIWAG gap injection method.

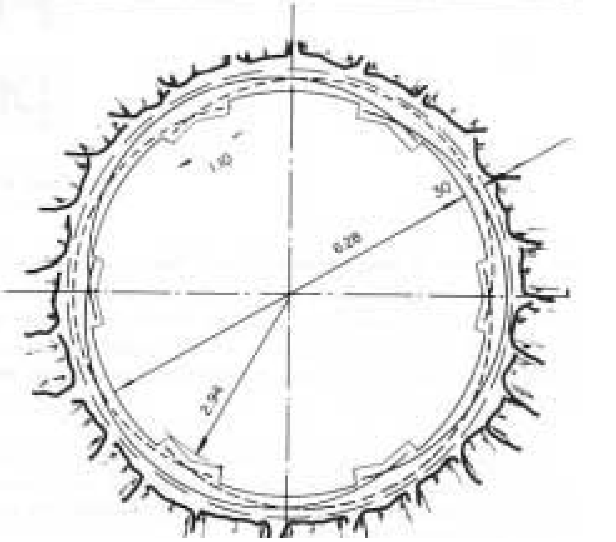


Fig 3. Active prestressing as used for the Lünerseewek project in Austria. Dimensions era in metres

development of forms of construction comprising a socalled passive prestressing, where the prestressing is produced by support from the surrounding rock. The best known of these methods include the core ring lining by Kieser' and the gap injection method of the Tyrol Hydro-electric Company[2] (TIWAG).

The method of Kieser is characterised by the fact that an annular void is maintained around the inner lining ring, subdivided longitudinally into sections; this void is grouted with cement mortar which then sets under pressure. The void is produced either by making the core ring from prefabricated blocks possessing humps or by concreting it in situ, using external formwork for the concrete made from plates with humps to maintain the distance from the rock (Fig. 1).

The TIWAG method is similar to the Kieser method. Cement grout is forced under high pressure into the contact joint between the rock and lining (which is of concrete or concrete and steel), causing the gap to open and to be filled with the injected material. Cement grains continue to be deposited by filtration from the injected grout until the gap is filled with densely compacted cement (Fig. 2).

In both these methods, the prestress is applied by hydraulic pressure acting in a cavity between the core ring and the supporting rock. A necessary condition for the permanent maintenance of prestress in these methods is adequate strength or adequate overburden depth in the rock.

Whereas the development of passive prestressing arose predominantly because of engineering reasons, economical considerations were the main deciding factors in replacing steel linings with concrete linings actively prestressed by post-tensioning steel. This is borne out by the fact that various methods have been developed or used for the first time in periods of acute steel shortages or high steel prices.

In these methods, the prestress is produced by a prestressing tendon running in or around the concrete ring, although here again the surrounding rock can be allowed to contribute to the resistance to the operating pressure. Among the first methods of this type was the system Wayss and Freytag, which was used in 1944 for a section of pressure tunnel of length 1316 m and diameter 3.20 m for the Kaprun power scheme in Austria. The lining consists of reinforced precast concrete rings of 340 mm length and 300 mm wall thickness, each lining ring being composed of six segments. Each ring was wound on a special machine with high strength steel wire of 6 mm diameter. The finished rings were advanced in the tunnel and placed on previously concreted pads. The space between the rings and the rock was closed in sections of 2 m length by injecting with cement grout.

In 1956-57, the firm of Dyckerhoff and Widmann prestressed, to its own design, a pressure shaft of 200 m length, 6.28 m and 5.64 m diameter, designed for an internal pressure of 1.2 MN/M^2, for the Lunerseewerk project in Austria.

This was the first time that prestressing was applied in tunnels by using individual post-tensioned tendons, in a similar manner to that used in the construction of prestressed concrete tanks. The individual tendons each extend around a portion of the circumference of slightly more than one third, the ends of the cables overlapping in concrete block-outs or buttresses. The gap produced during stressing between the concrete ring and the rock wall is subsequently injected with cement grout (Fig. 3).

Neither of these methods nor other methods of similar type have; however, become popular.

Prestressed solution by VSL patent

This technique falls within the group of the prestressing systems described last, in which the prestress is applied by individual tendons (Fig. 4). It is based upon the development of an annular tendon, which acts like a barrel hoop and thus requires no buttresses whatever.

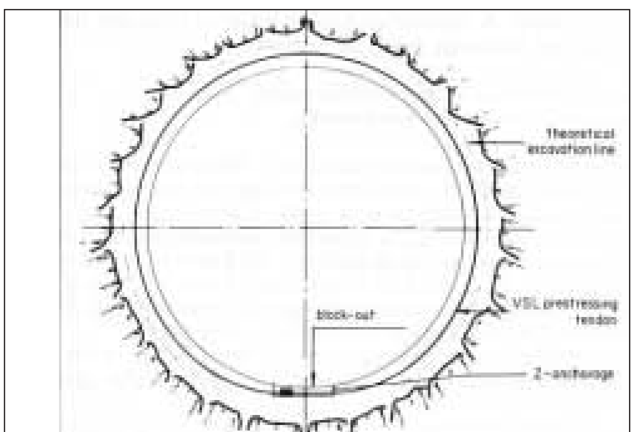


Fig. 4. VSL, Active prestressing system.

Centre stressing anchorage VSL type Z.

Centre stressing anchorage VSL type ZU.

The anchorage used, known as a centre stressing anchorage, at which the cable is closed to a ring, floats inside a concrete block-out and can move during stressing in the direction of the longitudinal axis of the cable. The cable is composed of a bundle of high-strength prestressing steel strands, which are either laid in a duct which is grouted later or are surrounded by a protective coat of anti-corrosion grease and a polyethylene cover and individually laid in the concrete. Two forms of centre stressing anchorage known as types Z and ZU are in use at present. Whereas in type Z (see picture top left) each strand is anchored, both at its dead-end and at its stressing end, by a pair of wedges in conical bores in the anchor head, the dead-end anchorage in type ZU is formed by looping the strands around the anchor head (see picture top right).

The prestressing steel used is strand with a nominal diameter of 13 mm or 15 mm and a maximum ultimate strength of 210 kN and 300 kN respectively. At present, the maximum number of strands per cable is limited to 12 where anchorage type Z is used and to six for type ZU, which gives maximum values for the cable's ultimate load of 3600 kN and 1800 kN respectively.

To stress the tendons, the strands are brought out of the block-out into the tunnel by a curved chair, consisting of a number of segments, which bears directly against the anchor head. The hydraulic centre-hole jack is then mounted on this chair and the cable is stressed in the usual way (see Fig. 5). The additional friction losses resulting from the deflection of 40° are compensated by over-stressing the cable.

The contact joint between the concrete ring and rock or make-up concrete, which has opened during prestressing, is later grouted with cement grout to ensure composite action; the grout is introduced through grout tubes previously cast in the concrete.

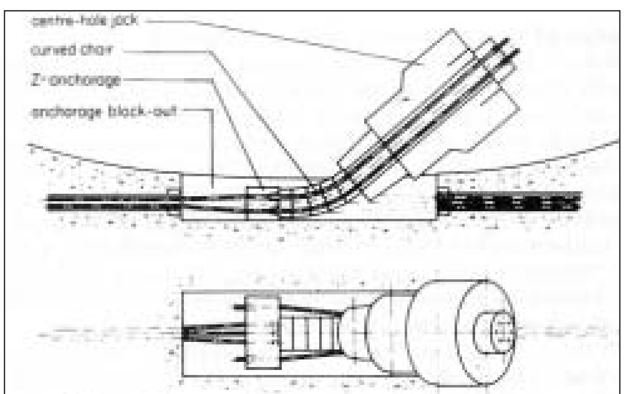


Fig. 5. Diagram showing stressing procedure.

The next operation is to concrete the block-outs or fill them with gun-sprayed concrete. Finally, if applicable, the ducts are grouted.

The advantages of a prestressed solution of this type can be summarised as follows:

- more economical, as a result of considerably higher tensile strength of the prestressing steel as compared with a steel-sheet lining, leading to a considerable reduction in the quantity of steel. Since pressure tunnels are usually located at remote sites where access is difficult, this also simplifies the transportation problems; this applies particularly to tunnels of large diameter, for which the steel lining is supplied in ring segments and must be welded together to complete pipe lengths on site; .
- reduced steel demand. This can be of particular importance in countries which do not have their own steelmaking industry. An accurate cost comparison with an equivalent steel lining will depend upon the current material costs, which are affected by considerable cyclical variations in steel prices, and also upon the geographic location of the site and the associated logistic conditions. Experience with projects so far carried out show that savings of the order of 10 to 30 per cent can be obtained with a prestressed concrete solution, by comparison with the costs of an equivalent steel lining;
- the order for the steel lining must be placed very early in the construction period, because of the long lead times for delivery. The prestressed concrete solution enables short-term decisions to be taken where circumstances decree, according to the quality of the rock encountered as the tunnel is excavated;
- excellent long-term strength is attained because of the absence of cracks in the concrete, which provides very good corrosion protection for the steel;
- there is no risk of buckling, as there is with a steel lining, because of sudden dewatering;
- prestressed concrete has the special ability to recover completely even after considerable overloading, without serious disadvantages remaining. Cracks which temporarily occur close up again completely.

The first practical experience with this system was obtained in the construction of the two surge chambers for the power stations of San Morino and SuvianaBrasimone in Italy. The first comprehensive application in tunnel construction was for the Piastra-Andonno plant for ENEL in Italy. Considerable operating difficulties were encountered in this plant in 1973, since the tunnel lining was cracked to such an extent that the water losses were unacceptable for a proper performance of the plant. The client decided to construct an additional lining for the 1 1.4 km-long tunnel, designed to be capable of withstanding an internal and also an external pressure of 0.8

Table I - Major dimensions and prestressed details of completed projects

Project	Country	Type of structure	Construction	Max. intern. service pressure (MN/m2)	Length or height (m)	Intern. diam. (m)	Thickness of lining (m)	Anchorages	Prestressing: Tendon and tendon spacing	Prestressing: Breaking load per tendon	Corrosion protection
Piastra-Andonno (1)	Italy	Pressure tunnel	1973/74	0.8	11 400	3.3	0.25	ZU 6-4	4@15mm a=0. 30 m	1000 kN	Cement grout
Taloro (2)	Italy	Pressure tunnel	1975/76	0.9	495	5.5	0.45	ZU 6-6	6@15 mm a=0. 30 m	1500 kN	Cement grout
Grimsel head-race tunnel (3)	Switzerland	Pressure tunnel	1977	0.75	197	6.8	0.40	ZU 6-6	6@15 mm a=0. 20 m	1547 kN	PE-covered strand
Grimsel tail-race tunnel (4)	Switzerland	Pressure tunnel	1977	1.4	60	6.8	0.60	Z 6-12	12@15 mm a=0.24 m	3094 kN	PE-covered strand
San Fiorino (5)	Italy	Surge shaft	1971/73	1.0	99	8.2	0.8÷0. 6	Z 6-12	12@15 mm a=0.25 ÷1 m	3000 kN	Cement grout
Brasimone (6)	Italy	Surge shaft	1973/74	0.6	61	26	0.70	EE 6-19 EE 6-12 (Buttresses)	19@15 mm 12@15 mm a=0.15-0.4m	4750 kN 3000 kN	Cement grout
Taloro (7)	Italy	Surge shaft	1975	0.9	90	14.9	0.80	EE 6-19 EE 6-12 EE 6-7 (Buttresses)	19@15 mm 12@15 mm 7@15 mm a=0.30-0.6m	4750 kN 3000 kN 1750 kN	Cement grout

Anchorage arrangements

MN/m² (8 bar) and to have a water loss not exceeding 7.5 Us/km. A comparison was made between a traditional solution using a steel sheet lining and a variant in prestressed concrete. The latter was chosen above all because it enabled the construction time to be reduced by almost 50 per cent, which of course was of decisive importance for an existing power station.

The entire length of 1 1.4 km was completely lined in 12 months and the readings obtained when it was commissioned again gave a water loss of only 1.25 1/s/km. (See photograph below and reference 3.)

Since that time, a number of other pressure tunnels and surge chambers have been successfully lined by this method or are at present in construction. Fig. 6 shows the pressure and diameter relationships for the projects so

Prestressed concrete lining as an additional lining for the 11.4 km long pressure tunnel of the Piastra-Andonno hydroelectric power plant in Italy.

far constructed, the hyperbolic series of curves indicating the hoop tensile force to be resisted by the prestress without any contribution from the rock. The dotted area shows the range of use of the concrete lining post-tensioned according to the VSL method, also on the assumption that there is no contribution from the rock, the upper boundary lines being determined by the space required for the anchorages. Table I contains the most important dimensions of these structures and of the prestressing.

For all the projects so far, with the exception of the pressure tunnel at Taloro, the prestressing has been designed for the full internal pressure. It is quite possible, if certain conditions are satisfied, to allow the rock to make a partial contribution, in some circumstances by combination with a passive, hydraulic prestress; this could result in a considerable extension of the range of application.

Design of a prestressed pressure tunnel

General. The pressure tunnel lining constitutes a shell, which is embedded in the rock. With the exception of freely placed pipes, therefore, the rock acts in all applications as a surrounding abutment and a component part of the total structure. It is, however, not possible for the static relationships in this composite structure to be established until an analysis and assessment of the loading influences has been carried out. It is therefore essential, before commencing calculations, to decide whether the nature of the rock and the amount of overburden allow it to participate as part of the tunnel structure.

In this connection, the VSL method proves to be very flexible. The hydraulic pressure may be either completely resisted by the prestressing action, or if the rock conditions are good, part of this pressure may be trans

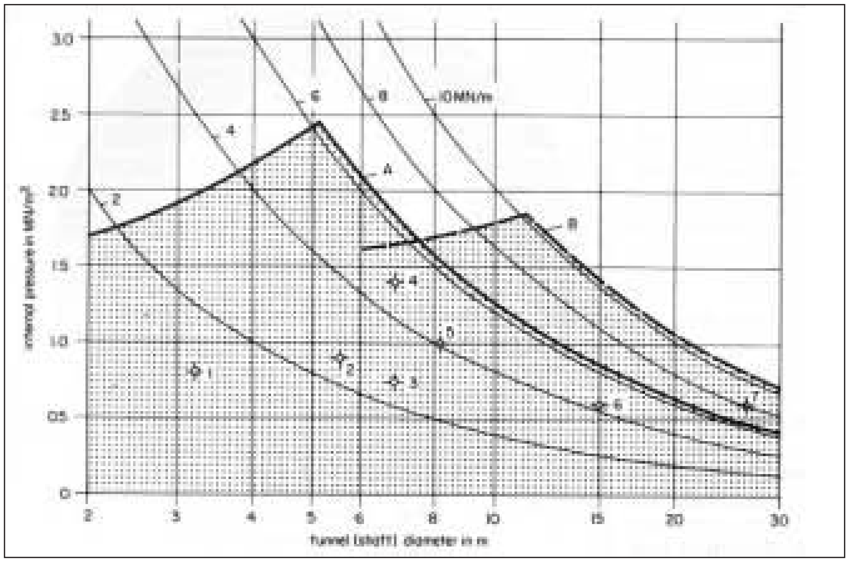


Fig. 6. Tunnel and shaft linings prestressed according to the VSL method, range of use. Curve A: without buttresses, maximum tendon breaking load 3600 kN. Curve B: with buttresses, maximum tendon breaking load 5700 kN.

mitted to the rock (although not to the same extent as with a steel plate lining). Normally, a circular crosssection of constant concrete thickness is assumed when making the design calculations. This assumption is evidently true only to a limited extent, since it is almost impossible to blast the tunnel through the rock as accurately as if it were drilled or cut.

Loads in tunnel lining

Hydraulic pressure in tunnel. The tunnel is in particular subjected to the hydrostatic water pressure, known as the operating pressure. This, the most important applied loading case, produces hoop tensile forces and deformations in the lining, which can be calculated on the assumption of a freely supported tube by known methods of analysis.

Other influences. This group comprises the following loading cases:

- rock water pressure (external pressure);
- rock pressure;
- changes of state in the prestressed concrete tube (creep, shrinkage);
- temperature; and,
- self-weight.

Extensive investigations have shown that these loading cases have only a secondary influence upon the design of the prestressed concrete tube, by comparison with the loading case for internal pressure. They must, however, be investigated from case to case.

Prestressing

General. As already mentioned, the tunnel lining is subject to the internal pressure p_i. It is the function of the prestressing to overcome..the hoop tensile forces produced by the operating hydraulic pressure. This is achieved if the prestressing force p is so chosen that the inward radial induced force $f=P/r_{ps}$ at every section of the pressure tunnel periphery is constant and thus equal and opposite to the operating hydraulic pressure pi (Fig. 7). A necessary condition for this, however, would be a frictionless, circular and concentric arrangement of tendons. This of course is not so for an individual tendon, because the prestressing force P varies on account of friction and moreover the circular arrangement cannot be maintained in the vicinity of the anchorage since at that point the cable must be deflected to the innerface, to enable it to be stressed. This results in varying inward radial induced forces (Fig. 8) and thus in bending moments.

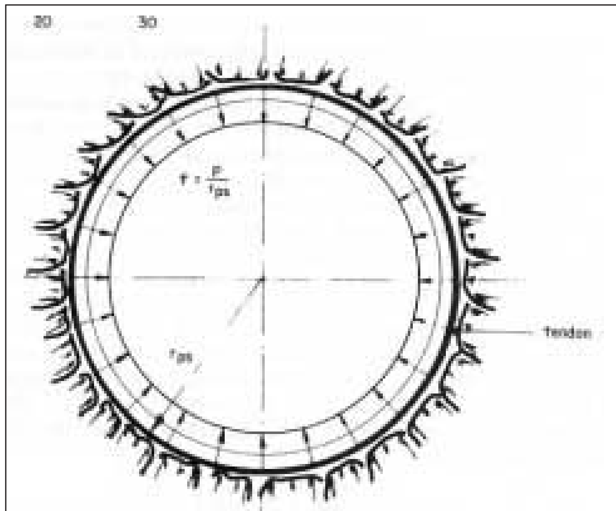


Fig. 7. Ideal tendon profile giving a constant inward radial induced force.

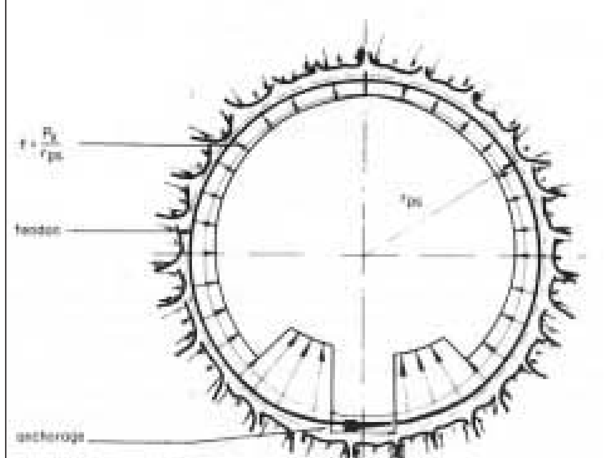


Fig. 8. Practical tendon profile showing diagrammatically the inward radial induced force.

Influences on cable force. The prestressing force initially applied at the anchorage falls off because of friction along the tendon according to the cable friction formula. This states that:

$$P_x = P_0 e^{-(am+Kx)}$$

Table II-Variation of friction coefficients				
	m		*K*	
	Range	Mean Value	Range	Mean Value
Steel duct	0.16-0.22	0.19	0.6-1.Ox 10^{-3}/m	0.8x 10^{-3}/m
Polyethylene covered strand	0.08-0.12	0.10	0.6-1.4x 10^{-3}/m	I.Ox 10^{-3}/m

Where:

P_x = prestressing force at a distance x from the stressing point,

m = coefficient of friction because of curvature of profile,

K = coefficient of friction because of wobble effect of cable,

a = total angular change of direction of cable in radians=x/r_{ps}

Experience shows that the coefficients of friction can vary from case to case, as shown in Table 11.

Using the mean values of m, the residual cable forces at the point on the cable diametrically opposite the anchorage are about 0.55 P_o for steel ducts and about 0.73 P_o for polyethylene-covered strand. Because of the large angular change of direction a in annular cables and the relatively short cable length, the influence of the wobble factor K can normally be neglected.

When anchoring the strands in the anchor head, the draw-in of the wedges results additionally in a strain loss of 6 mm, which leads to a local loss of force in the cable near to the anchorage. Assuming a linear force loss and an equal value for the friction coefficient for reverse movement of the cable of 3 mm on each side of the anchorage, the influence of wedge draw-in can be calculated as follows:

$$w = [(Dl_l E_s A_{ps})/D_p]$$

$$Dp = 2D_{pw}$$

where:

Dl_l = wedge draw-in (for Z- and ZUanchorages=0.003 m)

E_s = modulus of elasticity of prestressing steel=2x 10^5 MN/M^2

A_{ps} = area of steel

D_p =force loss/m=$(P_o-P_{x1})/x_1$

Fig. 9 shows the prestressing force diagram for an annular tendon allowing for friction and wedge draw-in. By carefully superimposing a number of curves of this type, that is to say by staggering the anchorages of adjacent cables, it is possible to obtain an approximately uniform prestressing force around the entire circumference. Possible arrangements of anchorage block-outs are illustrated in Fig. 10.

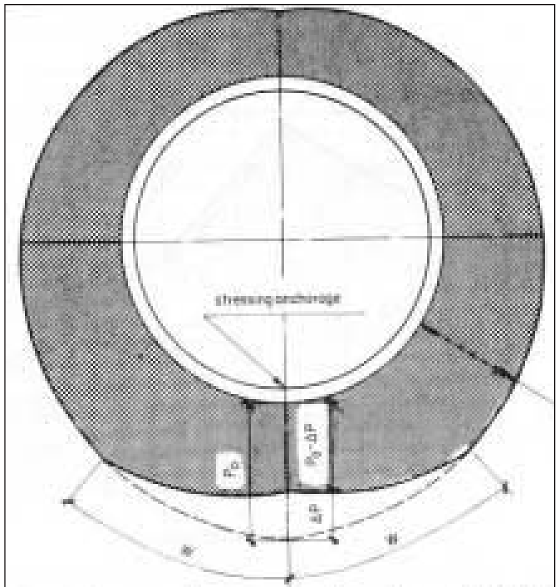
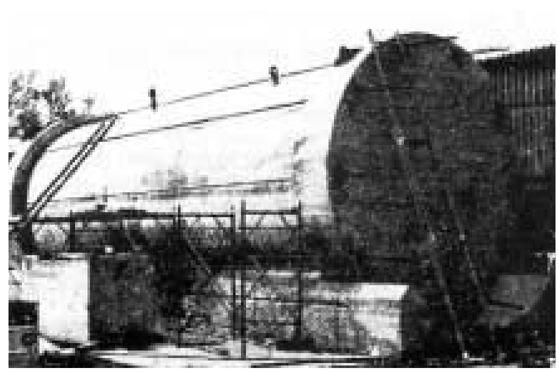


Fig 9. Diagram of prestressing force for a circular tendon taking into account friction and wedge draw-in

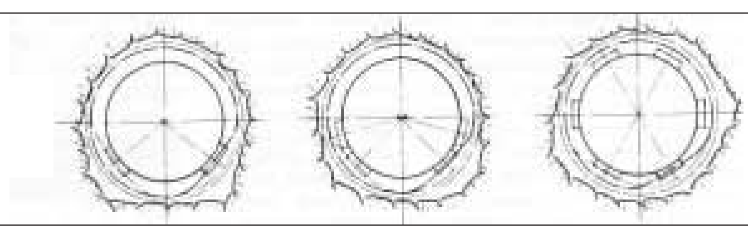
Infuence of cable eccentricities. As mentioned, the cable axis departs in the region of the anchorage from the concentric circular form and is brought towards the inside of the prestressed concrete tube. The deviation from the circular form results in bending moments, the influence of which cannot be neglected. The calculation of the internal forces and deformations for the free tube can be carried out, for example, by the method of the elastic centre of gravity[4] or by using known computer programmes.

In conjunction with the design of the new lining for the pressure tunnel of Piastra-Andonno, it was decided to check the calculations by a test pipe to a scale of l:1 (see picture below). The 10 m-long pipe was first prestressed and the concrete compressions which occurred were measured by deformation gauges and electrical resis



Test pipe the Piastra-Andonno project

Fig. 10. Possible practical arrangement of the block-outs within a group of adjacent cables (for pressure tunnels).



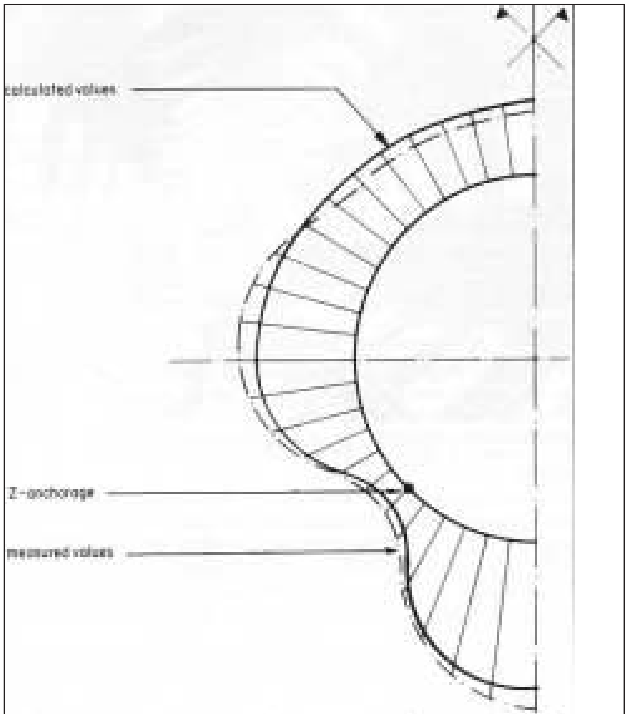


Fig. 11. Concrete compressive stresses at the outer edges in test tube as a result of prestress for the mean of two adjacent tendons. The effect on the concrete stresses of the cable profile in the region of the block-out is evident.

tance strain gauges. Fig. 16 shows a comparison between the concrete compressive stresses measured on the outer face of this test pipe and the calculated values. The good agreement between theory and practice is evident. The influence of the bending moment because of varying cable eccentricity is clearly visible. In the region of the block-out, it leads to a reduction in the concrete compressive stresses on the outside of the pipe and correspondingly to an increase on the inside of the pipe.

After the prestress loading case had been checked in this way, two heavy closure plates were fitted to the pipe and stressed onto it by prestressing tendons. To check the loading case for internal pressure, water was pumped into the pipe and raised to the operating level of 0.8 MN/m^2. The performance of the test pipe was absolutely satisfactory both in regard to the measured concrete compressions and in regard to watertightness.

Without going into the matter in greater detail here, it should be mentioned that this really undesirable influence of the bending moment can be reduced to an acceptable value by careful arrangement of the tendons and a favourable distribution of the stressing positions around the periphery (see Fig. 11).

Infuence of embedment in rock and varying wall thickness. The influences of embedment in the rock and a varying wall thickness should be taken into account for the following reasons:

• the prestress produces hoop forces and bending moments which deform the pipe in the transverse direction. Since the concrete is cast directly against the rock, the.pipe cannot deform freely, but is supported by the rock in certain zones. The design assumption of a free pipe is therefore not in accordance with reality;

• if the rock is blasted, an irregular tunnel profile results. For simplicity, however, the calculations are based upon the assumption of a circular pipe. It would be uneconomical to construct a, circular profile by preliminary concreting. In practice, a lining is therefore produced which departs from the ideal form.

In co-operation with the Institute for the Construction of Roads, Railways and Works in Rock of the ETH Zurich (Swiss Federal Institute of Technology of Zurich), an attempt has been made to allow for these two influences in the calculations. The results show that if embedding in the rock is assumed, the deformations and bending moments are reduced (Fig. 12), and that the influence of an irregular profile on the internal stresses remains within acceptable limits.

An important conclusion reached in this connection is that because of these two influences, it is not possible to determine accurately the state of stress in the prestressed concrete lining, since both the excavated profile and the rock properties can vary within the length of tunnel to be lined. It must, however, be emphasised that, after the prestress losses such as relaxation of the steel and creep and shrinkage of concrete have become attenuated, the prestressing force is permanently retained and "actively" counteracts the internal pressure. To this extent, therefore, a knowledge of the actual state of stress in the concrete is not of primary importance.

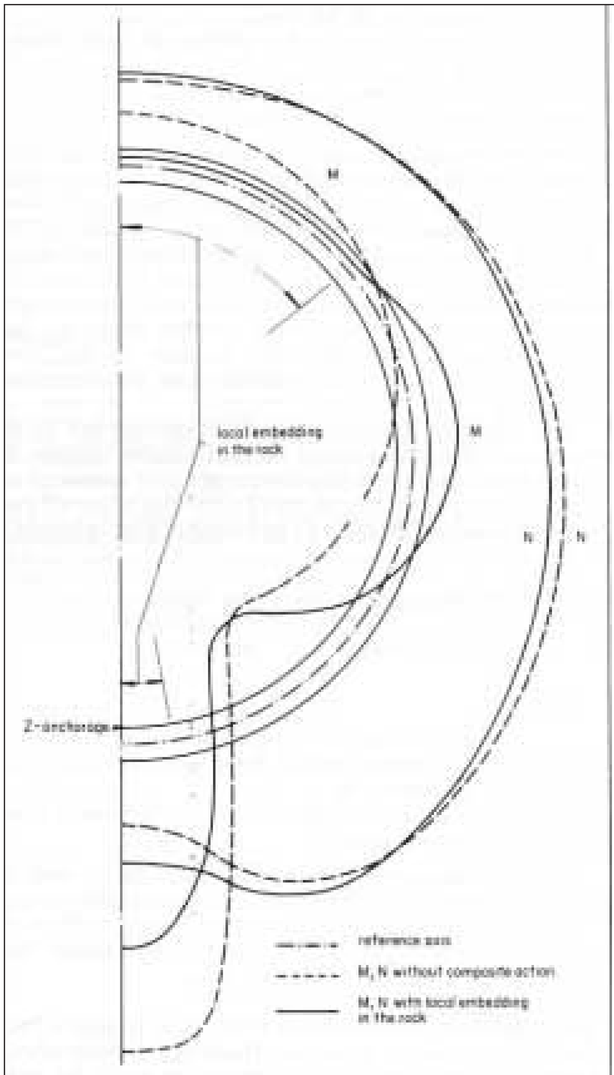


Fig. 12. Comparison of moments and axial loads without and with local embedding in the rock.

Other influences. For the sake of completeness, brief mention should be made of two further points, which must be considered in the design of a prestressed concrete pipe:

• prestressed pressure tunnels are usually of pronounced thick-walled type and it is therefore advisable at least to make an estimate of the influence of this thickwalled characters[5] ;
• the prestressed concrete tube is loaded in the longitudinal direction of the tunnel by the inward radial forces of the prestressing tendons. The resulting moments and shears can be calculated by the theory of the elastically embedded beams.[5]

Guidelines for construction

Concrete

General. The requirements for the lining concrete of prestressed pressure tunnels are very exacting. The concrete should possess the following properties:

• very high workability-the plastic to liquid consistency suitable for pumping concrete is most usually specified;
adequate strength, since relatively high compressive stresses can be expected as a result of prestressing (the minimum nominal cube strength at 28 days for prestressed concrete is 30 MN/m^2);
• watertightness, even under continual, high internal pressure; and,
• resistance to chemical and mechanical attack by the water flowing through the tunnel.

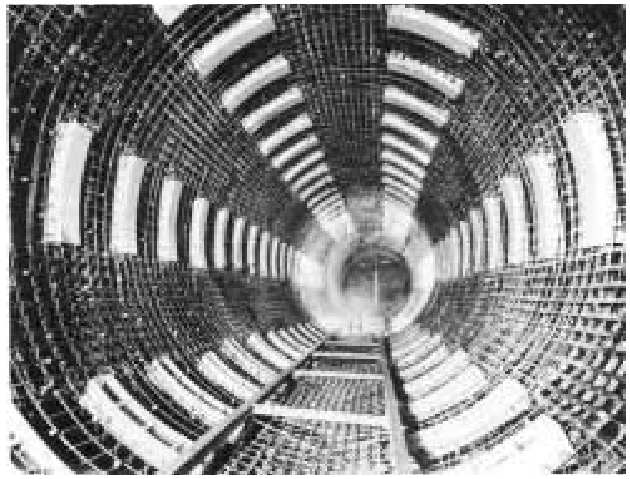
Workability and strength. A continuous grading of the aggregates, where possible of generally spherical shape, a large proportion of fines (sand and cement, and fillers) and high water content all favour workability. The strength, however, is inversely proportional to the watercement ratio. Any measures which tend to increase the water demand have an adverse effect upon the strength. Only an increase in the cement content improves both these properties together, but because of the other adverse influences, such increase must be undertaken with care.
The required concrete strength is determined by the load case for prestressing. If permissible values for stresses in cylindrical structures of small diameter are not given in the standards and codes, the required concrete strength at the time of stressing can be selected as follows:

f_{cu_0} ‡ $2.20\, f_{cp}$, max	the higher value should
f_{cu0} ‡ $1.65\, f_{pp}$ max	be selected

where:
f_{cu_0} = cube strength at time of stressing,
f_{cp} max = maximum central hoop compressive stress because of prestressing,
f_{pp} max = maximum peripheral hoop compressive stress because of prestressing.

The combination of the prestress after losses with the maximum stresses to be expected from external pressure (ie, rock water pressure and rock pressure), without the internal hydraulic pressure, should not exceed twothirds of the cube strength at 28 days.

Watertightness. Permeability to water is largely a function of the concrete structure. Hardened cement always contains pores; concrete can therefore never be absolutely watertight. It will always contain at least 10 per cent by volume of pores of varying origin, possessing differing properties.



Prestressing tendons, block-out formers and ordinary reinforcement in position. (Photo: Ingenieur Unternehmung AG, Berne.)

Watertightness is, however, very dependent upon a high standard of workmanship, avoiding honeycombing and the like. As a general rule, it may be stated that all measures which result in a dense concrete with few pores improve its watertightness. There is, however, no relationship between the strength and the water permeability.

If the aggregates are well graded and river sand is used as fines, a concrete containing 500 kg cement per m^3 of concrete and having a water/cement ratio of 0.4 is practically impermeable to water, even under high pressure (2.4 MN/m^2) and over a long test period.

A quantity of 350 kg of cement to 1 m^3 of concrete should be regarded as the minimum content. Cement having a high grain fineness can also be recommended. Special cements (eg, hydrophobic types) are, however, almost out of the question for economic reasons.

The risk of trouble resulting from a high cement content is not so great in tunnel construction as, for example, in building construction, since as a result of the high ambient humidity the shrinkage only reaches a fraction of the values which must be allowed for in structures constructed in the open. Nevertheless, proper curing is of importance in tunnels also. Among additives, plasticisers can in general be recommended, provided that their period of action is adequate.
To summarise, it may be stated that if suitable measures are adopted a virtually watertight prestressed concrete lining can be obtained. The freedom from cracks of prestressed concrete is itself a feature contributing to this result.

Durability. The long-term strength or durability is a term which should be used in relation to the type of attack, and desired life which the structure is intended to survive without damage. In pressure tunnels many of the normal influences such as large temperature fluctuations, frost attack and salt corrosion hardly apply, while the occurrence of other sources of damage, such as mechanical abrasion because of ice, gravel and the like is also very limited. On the other hand, durability against other types of attack, such as cavitation and chemical action by the water in the tunnel are very important.

The resistance to cavitation of the concrete can be increased by a factor of several times by good compaction, a low water content and high quality aggregates without micro-fines. An important aspect also is the painstaking elimination of all unevennesses and differences in smoothness (in particular, the block-outs must be carefully filled). Very careful attention should also be paid to the construction joints and junctions with lengths of steel-lined tunnel. In addition, concerning the durability of a prestressed concrete tunnel lining, the same precautions should be taken against chemical attack as for an unreinforced or normally reinforced lining.

Normal reinforcement

Although the main loads are carried by the prestressing tendons, construction reinforcement should be provided, as with other prestressed concrete (see picture on opposite page). This consists generally of a reticular pattern or reinforcement near the inner face of the tunnel and of anti-bursting reinforcement of the tendons. The quality of reinforcement in the longitudinal direction of the tunnel is determined by the loading during the posttensioning operation (by construction conditions). In the transverse direction, distribution reinforcement. of 0.10 per cent of the theoretical concrete cross-section is usually sufficient. Reinforcement to prevent bursting out of the prestressing tendons is usually only necessary in the vicinity of small radii of curvature close to the blockouts.

Grouting of concrete/rock contact joint

The contact joint between the concrete and rock, which opens slightly during prestressing, must be grouted. As with other contact grouting in tunnel construction, this must be provided for by the incorporation of grout pipes in the concrete at regular intervals.

The value of the grouting pressure should be limited so that the concrete compressive stresses because of active prestress and grouting pressure do not exceed 85 per cent of the cube strength.

Concreting of block-outs

Since the concrete filling of the block-outs cannot be introduced until after the cables have been stressed, it is subject only to slight prestress as a consequence of subsequent deformations in the tunnel. Particular attention must therefore be given to the clean and dense filling of the block-outs either with cement mortar, or concrete applied conventionally or by gun.

Permanent protection of the prestressing steel can be ensured, in this region also, by properly treating the contact surfaces to the filling concrete, the incorporation of normal reinforcement and the use of a filling concrete possessing minimal shrinkage.

Concluding comments

An attempt has been made here to describe the knowledge gained so far in a relatively new field of prestressing and thus to make this knowledge available to a wider public. The theoretical discussions given, the practical guidelines and the chosen specific examples serve to demonstrate that pressure tunnels which require an internal reinforcement can be economically constructed to a high engineering standard with a prestressed concrete lining.

The installations of the Kraftwerke Oberhasli in the Bernese Oberland are currently being extended by the Oberaar-Grimsel pumped-storage scheme with an output of 300 MW, for which a prestressed concrete lining is being constructed in various lengths of tunnel. It is intended that these works will be the subject of a future article, which will contain a more detailed discussion of the economic aspects and the solution of the practical problems.

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