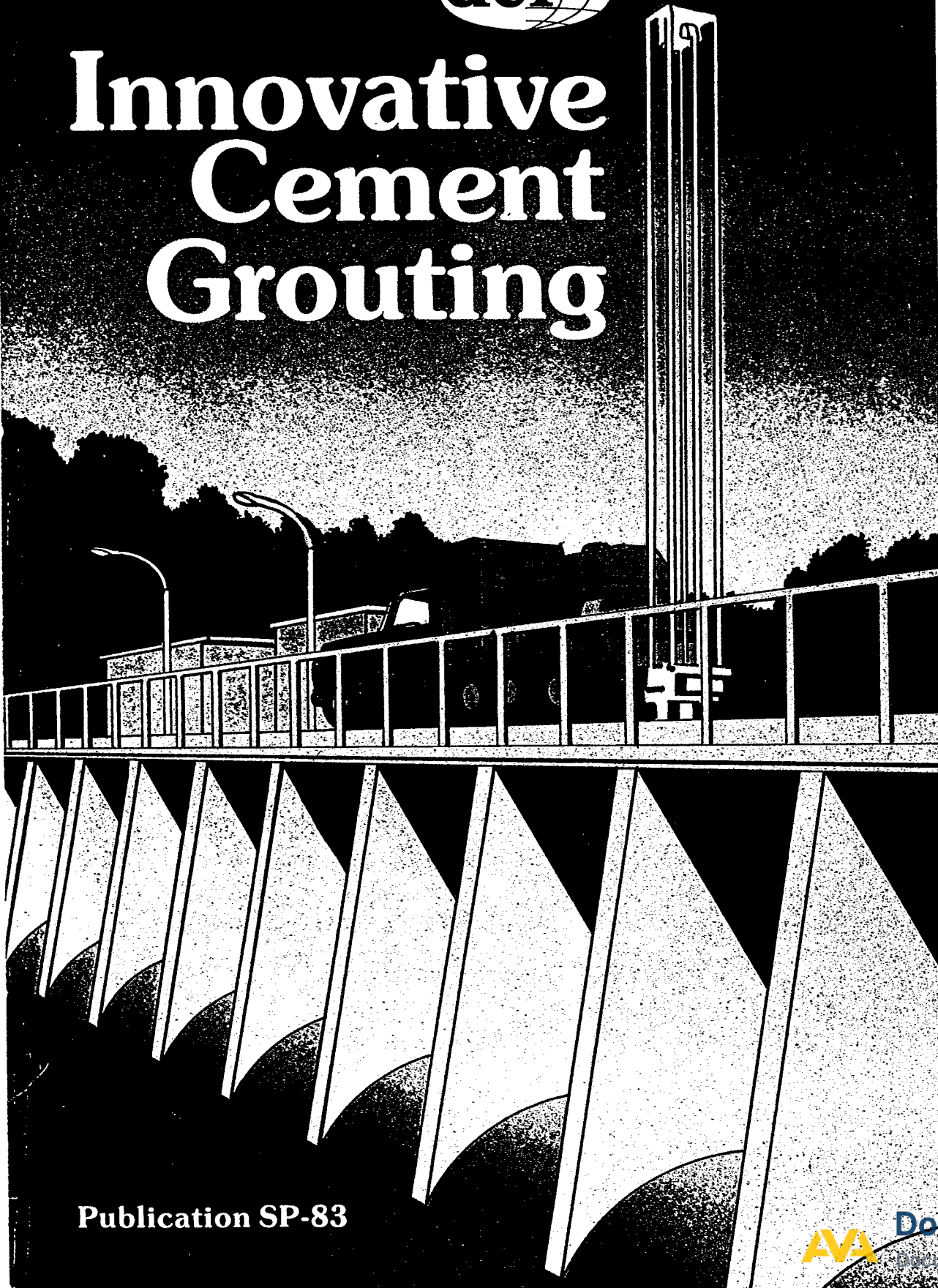




Innovative Cement Grouting



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PREFACE

ACI Committee 552--Cement Grouting--was organized in 1981 and given the mission to study and report on the application and use of cement grouting techniques for ground modification.

This publication includes a selection of the papers presented at the two-part technical session, "Innovative Materials and Techniques in Cement Grouting," during the 1983 Fall Convention in Kansas City.

While cement grouting has been used in construction projects for more than 100 years, only recently have industry, government, and academia formally recognized the need for cooperative research in this area.

Included in this volume are both academic research papers and construction-oriented papers.

The academic papers discuss potential new uses of existing materials, current research into a new nondestructive testing method, and properties of cement grouts.

The construction-oriented papers report on innovations performed on particular projects which could have broad application to the dam, tunneling, and rehabilitation industries.

This volume reflects the overall theme which permeated the sessions: the need for much more academic and applied research in the grouting industry.

With this in mind, Committee 552 has undertaken the difficult task of preparing a state-of-the-art report on cement grouting.

Committee 552 thanks the authors for sharing their experiences through their papers, committee members who reviewed the papers, Leon Glassgold of TAC who was instrumental in founding and steering the committee, and the ACI staff who helped organize the sessions and prepare this volume.

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Chairman, Committee 552

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SP 83-1**The Use of Condensed Silica Fume
in Grouts**

By Pierre-Claude Aitcin, G. Ballivy,
and R. Parizeau

Synopsis: Condensed silica fume is a by-product of the fabrication of silicon or ferrosilicon alloys. It is composed of very fine glassy spheres of quite pure silica having an average diameter of 0.1 μm . It is a very reactive pozzolan that has been found to increase drastically the compressive strength of concrete and to reduce significantly its permeability.

The unique properties of condensed silica fume can also be used advantageously in grouts if condensed silica fume is used in combination with a superplasticizer. The very fine silica particles give stability to the grout, there is no sedimentation of the cement particles and no bleeding. By adjusting the respective proportions of portland cement, of condensed silica fume, and of the superplasticizer a thixotropic grout can be made.

The hardened grout is stronger and less porous when it contains condensed silica fume because the lime liberated during the hydration of portland cement reacts with the very fine silica particles to form a compact secondary CSH. By this way the possibility of leaching out of the lime is practically suppressed and the resistance of the grout to chemical attack should be improved.

Keywords: bleeding (concrete); compressive strength; grout; plasticizers; porosity; pozzolans; shrinkage; silica; thixotropy; viscosity; water-cement ratio.

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P.-C. Aitcin is professor of civil engineering at the University of Sherbrooke, P.Q. Canada. His main research field is concrete technology. He has been working for the last sixteen years on the recycling of industrial by-products in cement and concrete (slags, fly ashes, condensed silica fume, mine tailings, etc.)

G. Ballivy is professor of civil engineering at the same University. His main research field is rock mechanics. He has been working on the different aspects of anchoring of rocks and cables in diams and underground constructions.

R. Parizeau is a graduate student at the same University. He is presently completing a master degree on the use of condensed silica fume on grouts.



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INTRODUCTION

Condensed silica fume is a new pozzolanic material that starts to be used currently in the cement and concrete industry (1, 2, 3, 4, 5). It is a by-product of the silicon and ferrosilicon industry. During the reduction of quartz by coal in the submerged electric arc furnace, SiO vapors are formed. Some of these vapors escape through the burden of the furnace, are oxidized when in contact with the oxygen of the air, and condensate in the form of very fine particles of amorphous silica. These particles have a mean diameter of about 0.10 to 0.15 μm (Fig. 1) that is fifty to hundred times finer than cement particles (6) (Fig. 2). The specific surface area of condensed silica fume measured by nitrogen adsorption is about 15 to 20 000 m^2/g .

Condensed silica fumes are currently used in addition to cement or in substitution of certain amount of cement in percentage in between 5 to 15 percent of the cement weight in concrete. This results in mixes where there is practically no bleeding or segregation due to the changes in the rheology of the mortar paste. When it is used in addition the compressive strength of such concrete is greatly increased. When it is used in substitution to produce a given 28-day compressive strength, condensed silica fume can replace 3 to 4 times its cement weight (7).

Condensed silica fume reacts quite rapidly for a pozzolanic material with the lime liberated during the hydration of portland cement to form an amorphous-like CSH (8). (Fig. 3). This pozzolanic reaction results in a decrease of the total porosity of concrete containing condensed silica fume as well as in a refinement of this porosity (9, 10) and a decrease of the permeability of the concrete (11).

In the present research work, we have tried to find if the unique properties of condensed silica fume could be used advantageously to overcome some problems that are normally encountered when dealing with grouts, i.e:

- stability of the grout (sedimentation of cement particles in the water)
- existence of an open porosity favouring the penetration of aggressive chemicals
- possibility of the leaching of the lime present in the hydrated grout.

We have also tried to find if a thixotropic grout could be made using cement, condensed silica fume and a superplasticizer, that is a grout which is fluid when agitated but sticky when at rest, in order to solve the problem of the grouting of roof anchor bolt holes.

EXPERIMENTAL PROCEDURE

All the grouts that contain condensed silica fume will be compared to a

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reference grout made with a Type 1 cement, having a water/cement ratio of 0.4 and that does not contain any additive.

The condensed silica fume used was a mixture of two condensed silica fumes coming from furnaces producing silicon and a ferrosilicon alloy containing 75 percent silicon. It had the following chemical composition.

SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	Na ₂ O	K ₂ O	C	L.O.I.
92.1	0.25	0.8	0.20	0.20	0.16	0.96	2.5	2.8

When dealing with grouts containing condensed silica fume, the water/cement ratio has to be changed for the water/cementitious ratio, that is the water to (cement + silica) ratio expressed in terms of mass. In all the cases the condensed silica fume dosage will be given as the percentage of the total mass of the cementitious particles (cement + silica).

Although the commercial superplasticizer that has been used was in a liquid form, all the superplasticizer dosages will be given as a percentage of their solid particles to the mass of cementitious particles. The superplasticizer used was a condensate of naphthalene formaldehyde.

INFLUENCE OF CONDENSED SILICA FUME ON THE INITIAL PROPERTIES OF THE FRESH GROUT

The influence of condensed silica fume in the initial properties of the fresh grout has been studied in terms of viscosity, thixotropy and bleeding.

Viscosity - - The viscosity of the grouts has been measured with a Synchro-Lectric type viscosimeter. (Fig. 4), and will be expressed as a percentage of the initial viscosity of the reference grout that was equal to 0.8 Pa.s (800 centipoises.) This method has been already used to study the rheological properties of cement mixes (12).

Initial viscosity - - It can be seen in (Fig. 5) that the introduction of condensed silica fume in a grout increases drastically its initial viscosity. The higher the condensed silica fume dosage the higher the increase. In the same Figure it can be seen that the use of a superplasticizer can decrease drastically the viscosity of a grout whether or not it contains condensed silica fume. However, the action of the superplasticizer is more effective on the reference grout than in the grouts containing condensed silica fume. There exists for each condensed silica fume dosage a superplasticizer dosage after which any addition of superplasticizer does not decrease anymore the viscosity of the grout.

For example, the initial viscosity of grout containing 10 percent condensed silica fume can be decreased to 30 percent of the viscosity of the reference grout by using 1 percent of superplasticizer. To reach the same

initial viscosity in the reference grout a superplasticizer dosage of 0.47 percent would have been sufficient and a dosage of 0.8 percent for a 5 percent condensed silica fume dosage. However, if a condensed silica fume dosage of 20 percent is used, the viscosity of the grout cannot be less than 60 percent of the initial viscosity.

Variation of the viscosity with time - - The variation of the viscosity of 5 different grouts has been observed for the first two hours. The results of this experiment have been plotted in (Fig. 6). In this Figure, it can be seen that during the first 10 minutes the viscosity of the reference grout increases rapidly (it practically doubles), then the viscosity stays unchanged for 1 hour after which it increases progressively for 2 hours.

In the two grouts containing 10 and 20 percent of condensed silica fume that have an initial viscosity about the same as the reference grout, it can be seen that there is no longer a plateau. The viscosity increases quite rapidly in spite of the presence of the superplasticizer. After a period of 1 hour the grouts are quite stiff.

However, if the initial viscosity of the grout containing condensed silica fume is adjusted to 30 percent of the initial viscosity of the reference grout, it can be seen in the same Figure that after 2 hours its viscosity is equal to that of the reference grout and no more higher than the one of the superplasticized reference grout of the same initial viscosity.

Thixotropy - - Grouts made by mixing cement and water are slightly thixotropic, but grouts containing condensed silica fume can be highly thixotropic. After a period of 20 minutes, such grouts seem to be stiff, but when agitated they exhibit the same viscosity as a reference grout at the same time. This thixotropy is illustrated in (Fig. 7). At time 0, the two grouts with and without condensed silica fume have the same viscosity. After 10 minutes the grout without condensed silica fume is still self leveling while the inclination of the becher containing condensed silica fume has to be increased slightly to reach the self leveling state. After 20 minutes, the grout containing condensed silica fume is stiff enough to stand in the beaker practically upside down. However, if agitated this grout become fluid again.

Sedimentation and bleeding - - The cement particles have a tendency to settle in normal grout and a film of pure water has a tendency to appear at the grout surface, that is the reason why grouts have to be agitated until they are injected. However, when condensed silica fume is present in the grout there is no more settling of the cement particles and no water separation. We have tried to visualize this effect in (Fig. 8).

INFLUENCE OF CONDENSED SILICA FUME ON THE PROPERTIES OF HARDENED GROUTS

Compressive strength - - The compressive strength of the grouts has been measured according to ASTM Standard C-109 "Test for Compressive Strength of Hydraulic Cement Mortar". The use of condensed silica fume does not increase

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significantly the grout compressive strength at 7 days. However at 28 days, the strength gain is more pronounced, and it increases with the amount of condensed silica fume. (Fig. 9).

Porosity measurement - - As the permeability of the grouts was too low (less than 10^{-12} m/s) it has not been possible to measure it. However we have proceeded to pore size measurement using a mercury porosimeter (12). The size of the cylindrical samples was quite big 20 x 100 mm (3/4 x 4"). They had a dry mass of about 50 g.

We have shown in Fig. 10 a and b the reproducibility of the test by running experiments on 3 different samples taken from the same reference grout. Figure 10 a gives the volume of intruded mercury whereas Figure 10 b gives the pore size distribution of the grout as a function of the pore radius.

We have shown in (Fig. 11 a) the average intrusion curve for the reference grout and a grout containing 10 percent of condensed silica fume at 7 and 28 days. It can be seen that the highest volume of intruded mercury is measured on the reference grout at 7 days and the lowest at 28 days in the grout containing silica fume. In this last case, the grout is practically non porous intrusion of (8 mm³/g of mercury instead of 120 mm³/g in the first case).

We have plotted in (Fig. 11 b) the pore size distribution at 7 and 28 days for the reference grout and 7 days for the grout containing condensed silica fume. It can be seen that the grout containing condensed silica fume has a finer pore size distribution than that of the reference grout at 28 days.

Shrinkage - - All the grouts that have been tested did not contain any additive at all. The shrinkage measurements have been made according to ASTM Standards C157-75 "Test for Length Change of Drilled or Saw Specimens of Cement Mortar and Concrete" with an apparatus complying with the ASTM Standard C490-77 "Apparatus for Use in Measurement of Length Change of Hardened Cement Paste, Mortar and Concrete".

The shrinkage of the reference grout at 60 days was equal to 0.21 percent while the shrinkage of the grouts containing respectively 10 and 20 percent of condensed silica fume was equal to 0.22 and 0.23 percent.

The presence of condensed silica fume in the grout does not change its shrinkage. This result has also been found true in concrete.

DISCUSSION OF THE RESULTS

The increase of the initial viscosity accompanying the replacement of a certain mass of cement by the same mass of condensed silica fume in grout can be overcome by using a superplasticizer. The superplasticizer dosage can be adjusted so that after 2 hours, the viscosity of the condensed silica fume grout be equal to that of a normal grout at the same time. The amount of

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superplasticizer necessary to decrease the viscosity depends of the condensed silica fume dosage. As there are more very fine particles in the grout, a higher amount of superplasticizer molecules are necessary to lubricate the particles present in the grout. However, when all the particles present in the grout have been lubricated, any addition of superplasticizer does not affect the viscosity of the grout.

It seems that the presence of condensed silica fume changes the kinetics of cement hydration because the viscosity of the grout increases constantly instead of being about the same from 10 minutes to 1 hour after the mixing of the grout.

Moreover, the thixotropic effect of condensed silica fume illustrates also the fact that there is an interaction between the very fine silica particles and the water molecules. This interaction can also explain why no bleeding is observed in grouts and concretes containing condensed silica fume.

Condensed silica fume increases slightly the compressive strength of a grout at early ages most probably due to its filler effect as it has been explained by H. Bache (13). However, the compressive strength increase observed at 28 days is a consequence of the reaction of the very fine silica particles with the lime liberated by the hydration of portland cement (2). This pozzolanic action can also explain the drastic decrease in the porosity of grouts containing condensed silica fume.

CONCLUSION

Condensed silica fume can be used in grouts if combined with superplasticizer in order to control the variation of viscosity with time. Such grouts are very stable, avoiding any water separation problems. The hardened grout exhibits a higher compressive strength and a lower porosity than a normal grout due to the filler and pozzolanic effects of condensed silica fume. Moreover the fixation of the lime liberated during the hydration of portland cement associated to the drastic reduction of the total porosity of the grout should make this type of grout less sensitive to chemical attack from an aggressive environment or to leaching by infiltrated water.

The authors are conscious that this research project is only a beginning in the development of the use of condensed silica fume in grouts, however they are confident that the unique properties of this new mineral additive could be used advantageously in the field of grouts. For example thixotropic compositions can be adjusted very easily without the need of bentonites only by selecting carefully the condensed silica fume and superplasticizer dosages.

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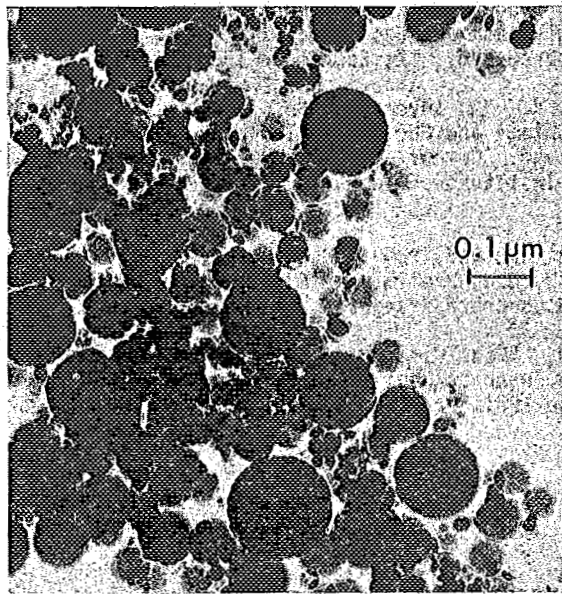


Fig. 1--Condensed silica fume grains

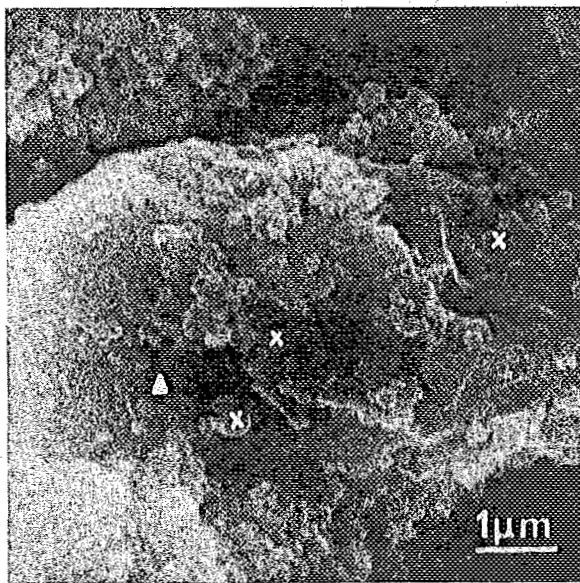


Fig. 2--Comparison of the size of condensed silica fume particles and portland cement clinker grain

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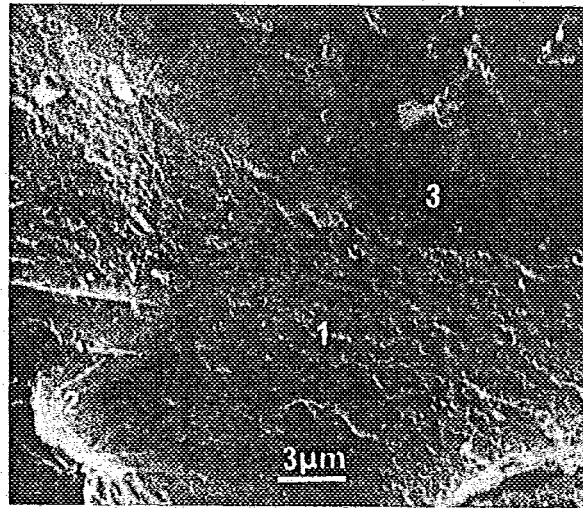


Fig. 3--CSH aspect in a concrete containing condensed silica fume

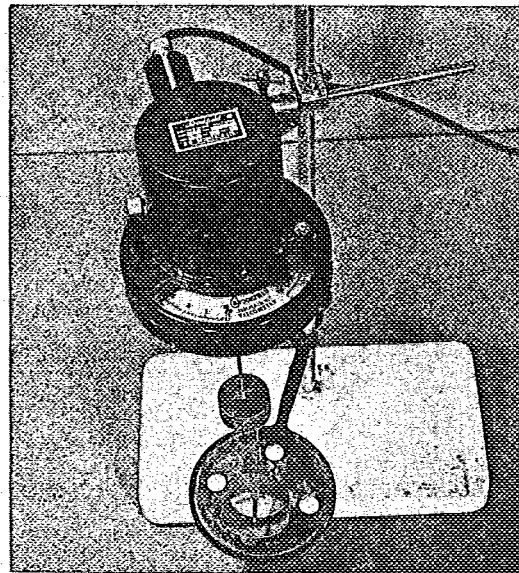


Fig. 4--The synchro-lectric viscosimeter with two different spindles

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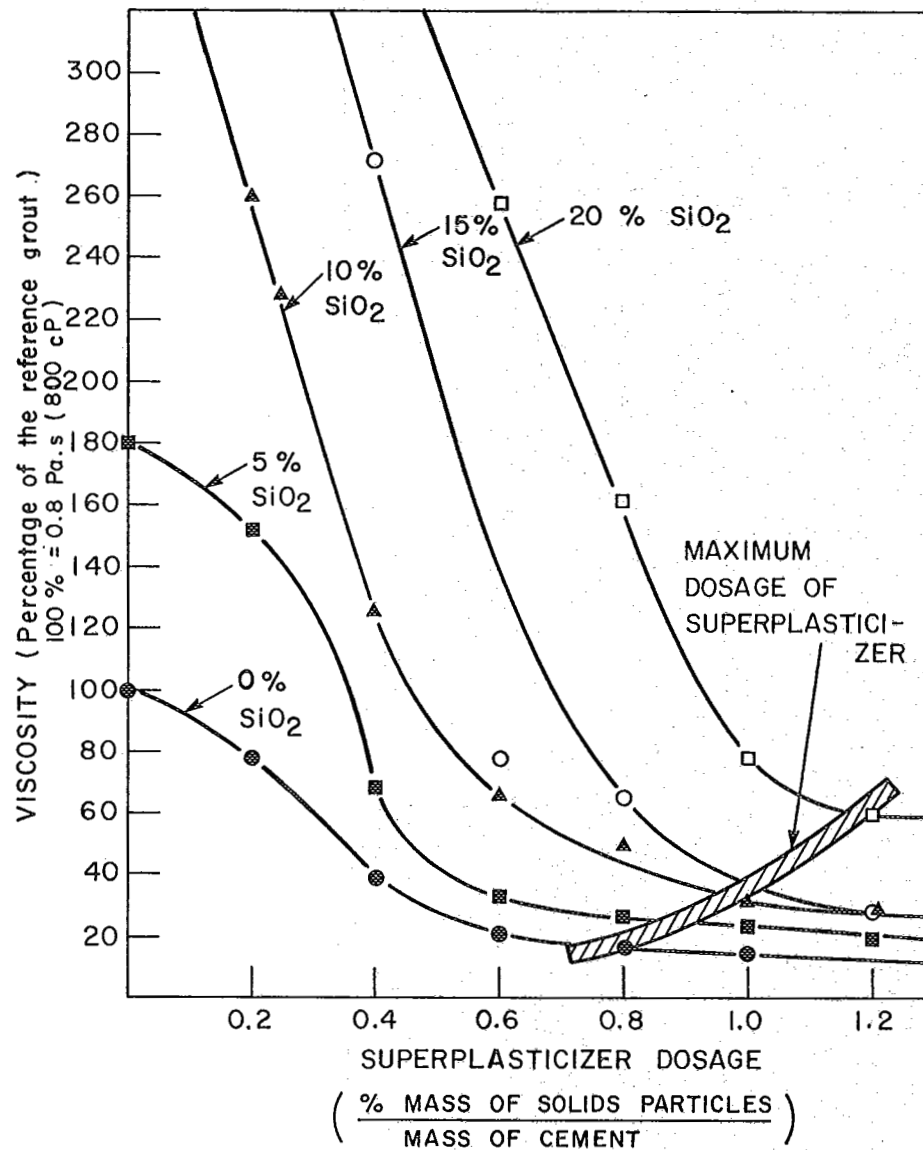


Fig. 5--Initial viscosity of grouts as a function of the superplasticizer and condensed silica fume dosages

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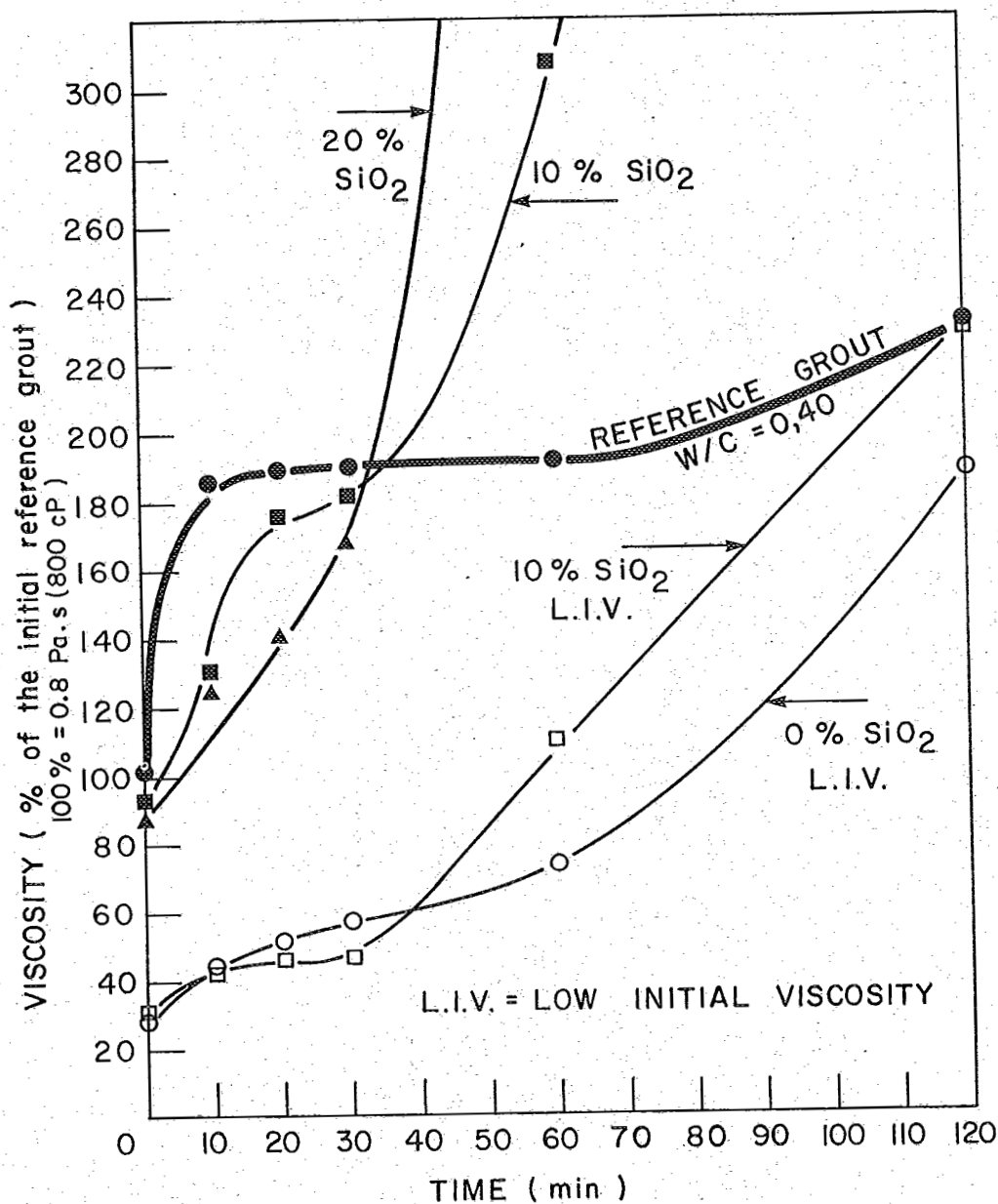


Fig. 6--Evolution of the viscosity of different grouts with time (in percentage of the initial viscosity of the reference grout W/C = 0.40)

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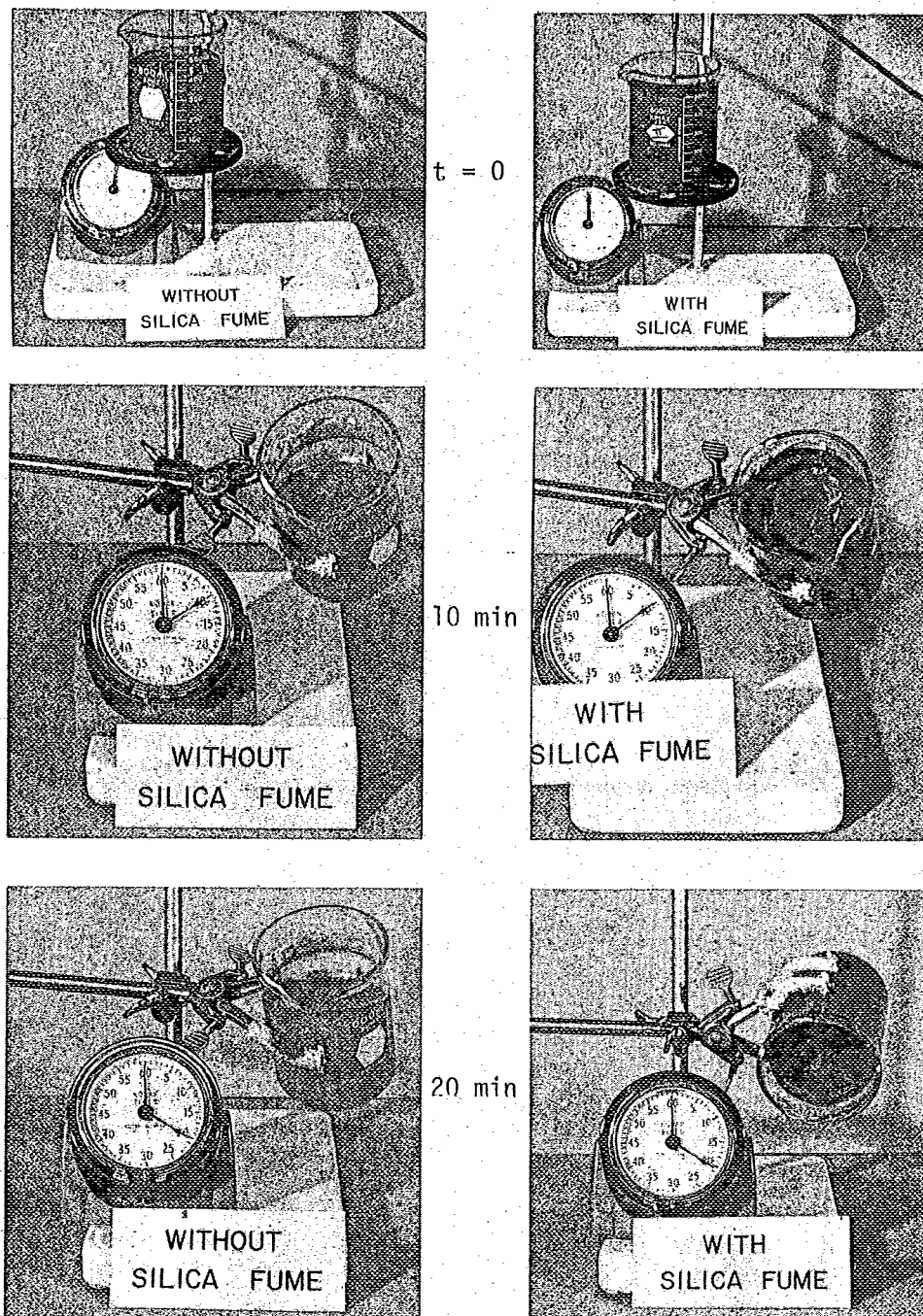
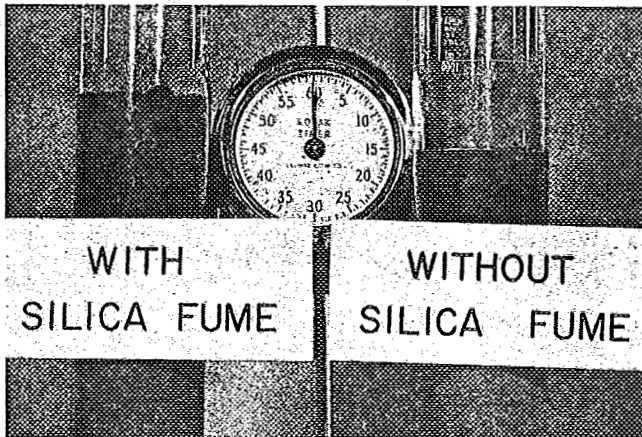
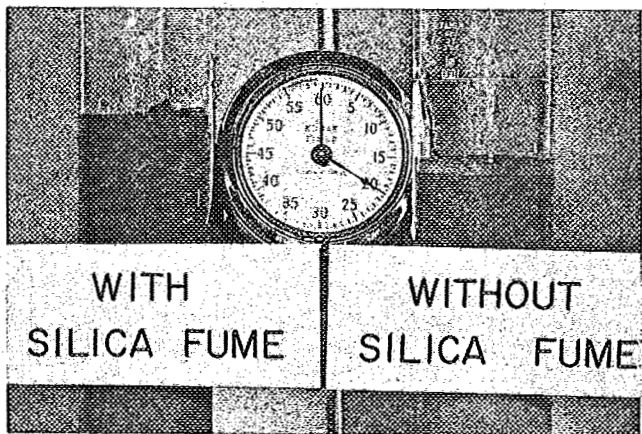


Fig. 7--Evolution of the thixotropy of two grouts having the same initial viscosity

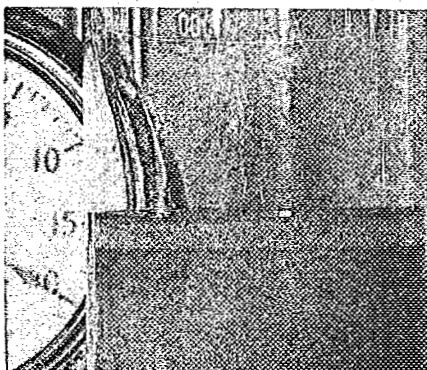
Condensed Silica Fume 15



$t = 0 \text{ min}$



$t = 20 \text{ min}$



Enlarged view
(without condensed
silica fume)

$t = 20 \text{ min}$

Fig. 8--Comparison of bleeding in grouts with and without condensed silica fume

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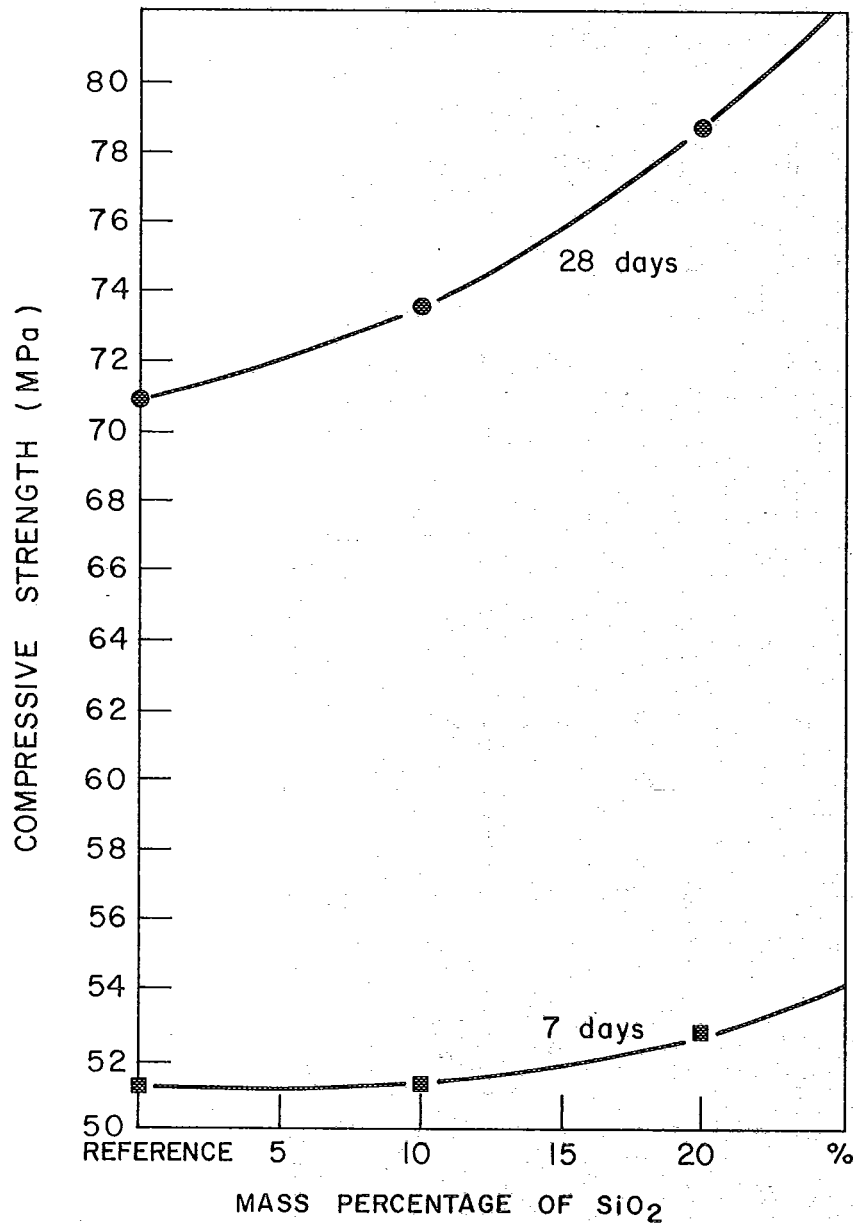


Fig. 9--Compressive strength of grouts containing different percentages of condensed silica fume

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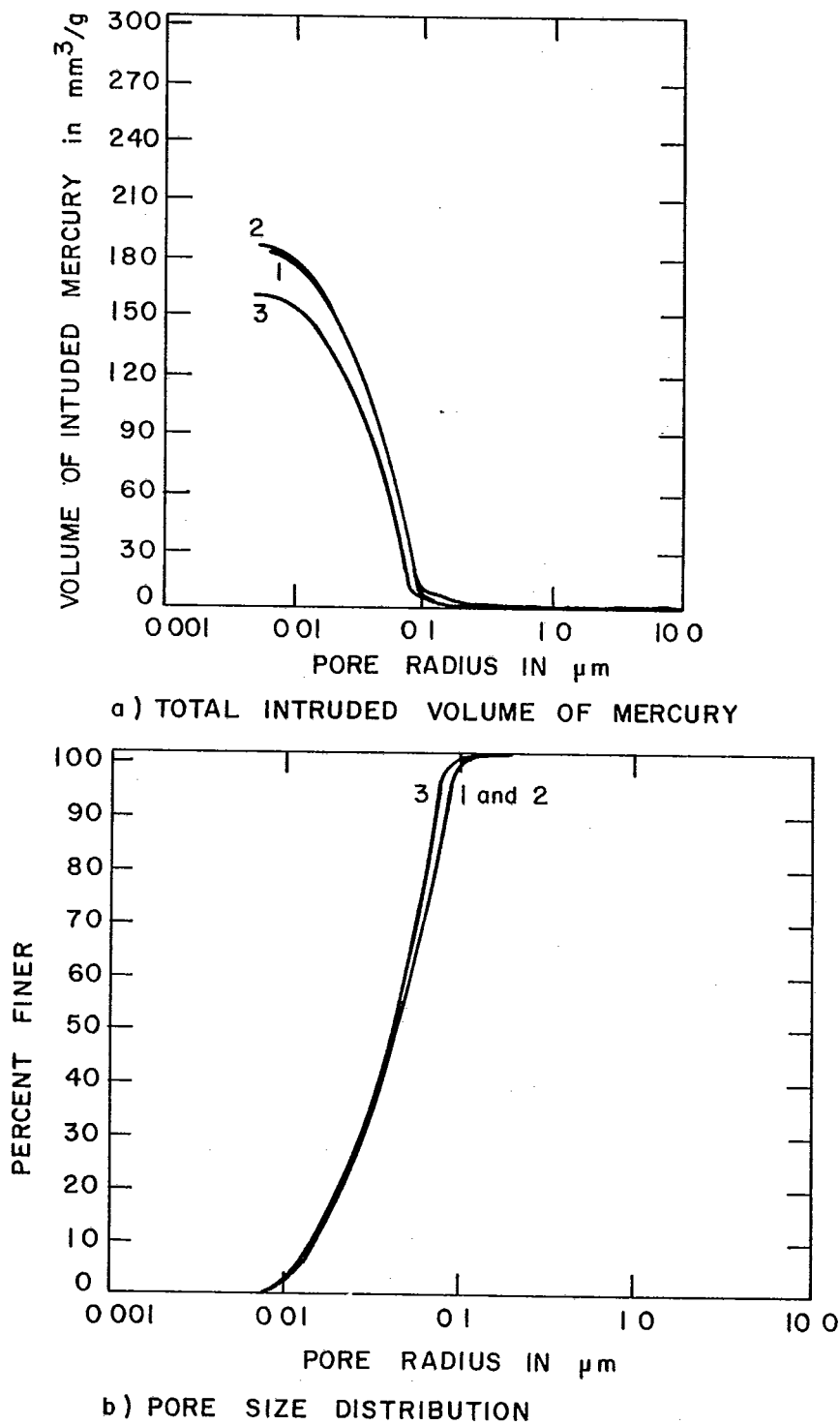
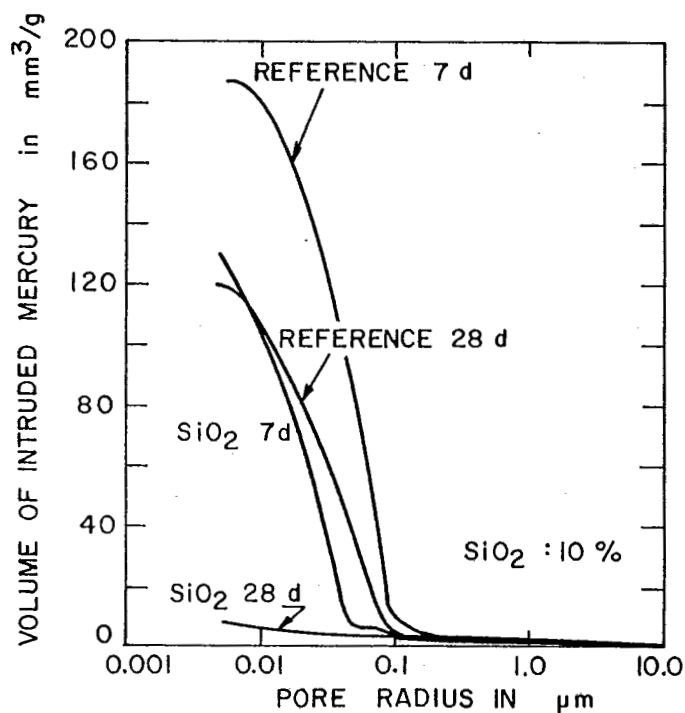
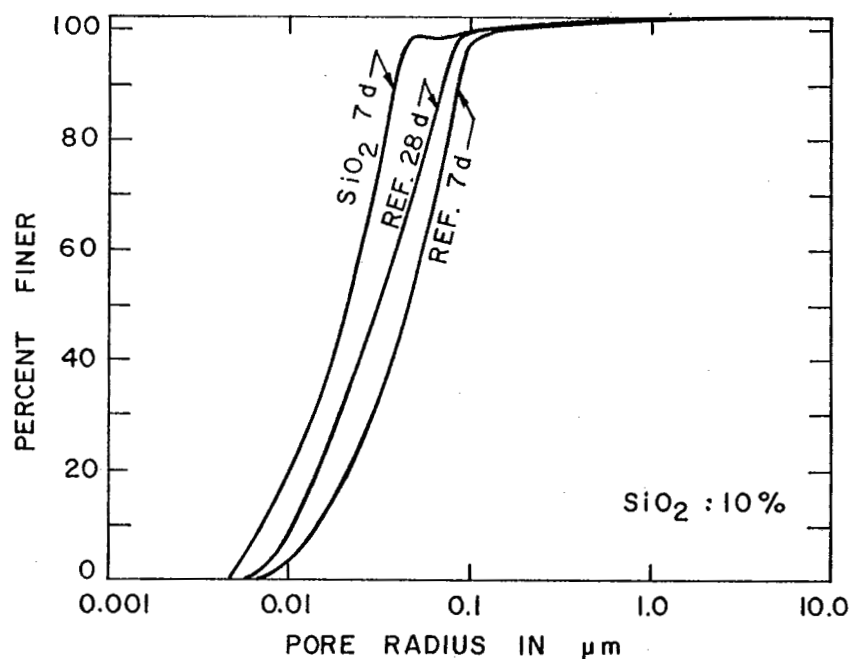


Fig. 10--Reproducibility of the intrusion of mercury in the reference grout; (a) total intruded volume of mercury, (b) pore size distribution

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A) TOTAL VOLUME OF INTRUDED MERCURY



B) PORE SIZE DISTRIBUTION

Fig. 11--Intrusion of mercury in two grouts, with and without condensed silica fume at 7 and 28 days; (a) total intruded volume of mercury, (b) pore size distribution

SP 83-2**Design and Prestress Grouting of a Concrete Lined High Pressure Tunnel at Drakensberg****By Luigi P. Gonano and John C. Sharp**

Synopsis: Innovative aspects of the design and construction of the concrete-lined high pressure tunnels of the Drakensberg Pumped Storage Project are described. These tunnels located in weak argillaceous rocks are prestressed and designed to remain crack-free at internal pressures up to 6.2 MPa. The extension of an innovative grouting technique developed in Europe provided an economical and technically acceptable alternative to steel liners. Design criteria for structural deformability, prestressing and creep behavior were developed using a prototype-scale test chamber. The design models and practical technology developed for interface grouting will allow the use of concrete linings in pressure tunnels with greater internal pressures and with poorer rock conditions than previously considered feasible.

Keywords: creep properties; grouting; pressure; prestressing; pumped storage; structural design; tests; tunnel linings; tunnels.

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L. P. Gonano: Obtained a Bachelor of Engineering in 1970 and a Doctorate (Rock Mechanics) in 1974 both at James Cook University, Australia. Experience is focused on rock engineering studies for large civil and mining projects, mainly underground. Currently a Senior Geotechnical Engineer with Golder Associates in Seattle.

J.C. Sharp: B.Sc., D.I.C. and Ph.D. in Rock Mechanics at Imperial College, London. From 1970-1980, held various positions up to Principal with Golder Associates in Vancouver, Melbourne and London. Currently, he is an Independent Consultant specializing in Rock Engineering.

1.0 INTRODUCTION

The recently completed Drakensberg Pumped Storage Project in South Africa is located in sedimentary rocks containing relatively weak argillaceous strata. The project comprises headrace tunnels, surge and pressure shafts, pressure tunnels/penstocks, an underground power station complex and a tailrace tunnel (Fig. 1). Details of the project are given by Pyzikowski (1).

At the downstream end of the pressure tunnels, the maximum internal pressure head varies from 500m (static) to more than 600m (static plus water hammer). This range of pressures together with the relatively high deformability and erosion/deterioration potential of the rocks necessitated special design and construction procedures. Only limited precedence exists for the use of high head concrete-lined tunnels in weak sedimentary rock.

The tunnel lining/grouting technology required to prestress concrete linings to accommodate the site pressure and rock conditions is described. Emphasis is placed on the basic design considerations, the special investigations conducted and the verification of the grouting operation.

2.0 BACKGROUND

Concrete linings have been widely used in the past as hydraulic surfaces for tunnels and shafts in hydroelectric and pumped storage schemes. As tunnels are constructed in more marginal conditions (low modulus, low quality or uncertain impermeability) and higher internal pressures are used, it becomes necessary to design the linings to contribute structurally to the stability of the conduit.

Prestressing of concrete linings has been used for almost a century to reduce cracking, to improve the efficiency of transfer of pressure to the rock and to render the lining impermeable. Early applications relied on the prestressing effect of tensioned wires located in an annulus. Although this method is still used



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today (2) the use of larger diameter conduits and higher hydraulic heads favors the use of grout injection methods. Talobre (3) cites numerous applications of both techniques in the first half of this century. More recently, special grout injection techniques have been developed and applied for use in the medium pressure range (100-250m head) e.g. the Kaiser (4) and TIWAG (5) methods.

Consolidation grouting of the rock often results in the development of nominal prestress in the lining. This side effect is beneficial in overcoming the main disadvantage of concrete, its low tensile strength. However, the contributions of prestressing have frequently not been explicitly factored into the design of the waterway. The benefits of prestressing noted above can only be relied upon in design, if it can be ensured that adequate prestress is maintained during the planned life of the structure. The uncertainty of estimating creep losses has been a significant factor and prestress requirements have been estimated using conservative and empirically based rules-of-thumb. For example, Jaeger (6) states that grouting pressures of at least twice those of the maximum internal pressures should be used.

An additional factor limiting the range of application of prestressed concrete linings has been the grouting technology itself. Borehole grouting results in highly variable lining prestress and furthermore the stress application factor (ratio of theoretical to actual prestress achieved based on pump pressure) is relatively low, of the order of 10 to 40%. The TIWAG grouting method developed by Lauffer and Seeber for the Tiroler Wasserkraftwerke AG in Austria is one grouting technique designed to overcome these disadvantages. This technique involves grouting of the interface between the lining and rock through embedded longitudinal or circumferential pipes. The rock surface is treated with a debonding agent or film prior to concreting to facilitate the formation of a thin wedge of grout which ideally "floats" the concrete lining away from the rock as a result of the elastic compression. With the aid of grout arrestors and the proper choice of grout mix, the grout annulus is slowly advanced along the tunnel in a continuous operation. The large areal exposure of the rock to the grout film also results in an efficient consolidation grouting of the rock and in some rocks, improved stiffness through preloading. This technique has been used in the past with considerable success in low pressure tunnels (less than 100m head) in Austria and has also been applied to the prestressing of the Gordon Power Tunnel in Australia (7).

The application of the technique at Drakensberg represents the first instance of the interface grouting of both shafts and high pressure tunnels. The principles and procedures for the prestress grouting of shafts are similar to those for the tunnels and only the design and construction of the high pressure tunnels are described here.

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3.0 PRESSURE TUNNEL LAYOUT AND GEOLOGY

The location of the pressure tunnels relative to overburden conditions and the layout of the project components is shown in Figure 1. The two concrete-lined pressure tunnels are each 5.5m diameter and approximately 850m long. A maximum grade of 1:10 from the base of the lower pressure shafts to the end of the steel lined penstocks was dictated by efficient excavation and lining methods.

The rocks through which the pressure tunnels pass form part of the Middle and Lower Beaufort Series of the Karoo System. They consist primarily of interbedded sandstones and siltstones (including muddy siltstones), with occasional thin seams of mudstone and carbonaceous materials (8).

Bedding is the dominant structure along the alignment and is also the most significant rock mass weakness, being sometimes slickensided or micaceous. Individual bed thicknesses vary from a few millimeters to several meters. The several faults, which have influenced the layout of the tunnels, typically comprise a brecciated zone some 0.1 to 0.5m in width and have associated sympathetic shears and jointing. These faults, together with occasional dykes of limited width constitute the more permeable features in an otherwise generally impermeable rock mass. Along the tunnel alignment, jointing is limited to specific zones, often in the vicinity of faults.

4.0 DESIGN CONSIDERATIONS

The permeability of the rocks traversed by the tunnels is sufficiently low to permit the economic use of a permeable (cracked) concrete lining designed primarily as a smooth hydraulic surface. However, several other factors influenced the design requirements at Drakensberg:

- o the high deformability of the rock could lead to excessive deformation and cracking of the lining under the high internal pressures
- o certain components of the rock appeared susceptible to hydraulic erosion and deterioration particularly under the transients resulting from pumped storage operation. Material transport through the lining cracks could lead to loss of support for and eventual breakup of the lining
- o the interbedded strata exhibited a low resistance to hydraulic jacking

It was concluded that to ensure the long term support capacity of the rock, the lining would need to be essentially crack-free. Thus, if a concrete lining was to be used, and this was preferred over steel because of the significant cost savings, considerable prestressing of both the lining and rock would be



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required. The need for an innovative prestressing approach can be seen from the comparisons in Figure 2.

Several factors influenced the choice of the TIWAG interface grouting technique:

- o the potential for high application factors and good uniformity of prestress
- o the capacity for simultaneously prestressing both the lining and rock
- o cost

The significance of the first factor can be seen in the following basic design calculation of the required initial prestress presented as an example. The geometry, pressure and creep values are also typical of the downstream section of the pressure tunnels (Fig. 3).

Internal Diameter/Thickness Ratio	= 7.33 (5500/750)
Modulus Ratio - Concrete/Rock	= 1.5 (30,000/20,000 MPa)
Maximum Internal Pressure	= 6.2 MPa
Concrete Strength at Grouting	= 30 MPa

For the above conditions, a change of internal pressure of 1 MPa results in a 1.3 MPa change in tangential lining stress. To prevent cracking, a minimum long term lining prestress of $6.2 \times 1.3 = 8.1$ MPa would be required to accommodate the elastic decompression caused by internal pressure. Because of variability in the prestress, the required average long term prestress is conservatively estimated to be at least 12 MPa (see Section 5). Creep, largely during 50 years of scheme operation is calculated to reduce the initial prestress by 50%. Thus, the average initial prestress required is about 24 MPa. Using thick cylinder theory, this translates into an effective radial grout pressure of 4.8 MPa.

Because of circumferential variability of prestress and bending effects, the maximum local tangential prestress will be greater than 24 MPa and in the same range as the concrete strength. This limitation of the useful application of crack-free prestressed concrete linings can be offset by designing linings with higher strengths and lower creep losses, and by improving the uniformity of prestressing.

Prestress application factors for borehole and for embedded piping interface grouting are typically 0.25 and 0.75 respectively. Thus, if borehole injection were to be used, grout pump pressures of the order of 20 MPa would be necessary. Such pressures could not be used in practice because of the problems of hydraulic jacking of the rock (in areas of low overburden cover) and the risk of local damage to the lining. Optimization to alleviate these particular constraints would involve, in addition to those methods noted in the previous paragraph, the use of thinner

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linings and higher prestress application factors.

The above design comparison of internal pressure, grout pressure, concrete strength and overburden stress underscores the need for accurate methods of predicting the losses in prestress arising from creep and for the optimization of the grouting technique. Developments in both areas will extend the practical range of prestress grouting to more demanding pressure head and rock conditions.

5.0 INVESTIGATIONS

The nature of the proposed grouting operation dictated the need for extensive investigations to determine reliable design criteria. Three types of design data were addressed in the investigation:

- o data on rock conditions, i.e., deformability, in situ stresses, hydrofracture potential, permeability
- o determination from insitu and laboratory tests, of the creep behavior of sound rock, disturbed rock, grout and concrete and the development of a pragmatic model for predicting prestress loss due to creep
- o investigation of the likely overall response of the lining/rock structure using a prototype field test section to verify the feasibility of using a prestressed concrete lining.

Details of the investigations are given by Sharp and Gonano (9). A full scale test section constructed to duplicate as closely as practicable the anticipated methods for the pressure tunnels was considered the only reliable means of evaluating the effects of rock disturbance, creep behavior of the concrete/rock/grout system and the structural interaction under internal pressures up to a failure condition. The test section was extensively instrumented to observe its stress - deformation response and thereby gain meaningful predictions of operating response for the variety of rock, head and overburden conditions along the alignment. The location of the test section is shown in Figure 1.

The test section was constructed prior to serious consideration of the TIWAG grouting method, and prestressing by borehole injection was therefore used. After exhaustive repetitions of grouting behind the lining, with pressures limited by hydrofracturing in the rock, a 40% average application factor was achieved. The tangential lining prestress obtained by this method was markedly non-uniform (Fig. 4). As an indicator of what can be achieved with borehole injection, these results provided the basis for the subsequent selection of the interface grouting method.



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The observations of the test section during internal pressure tests in the elastic range were used to derive deformability criteria for the lining/rock structure. Tests to failure established minimum overburden limits for the concrete-lined section. The tunnel deformability model developed incorporates the effects of modulus anisotropy, excavation, grouting and stress conditions.

Post-testing investigations of the test section over a period of 30 months consisted of measurements of lining deformations due to creep and loss of prestress, direct measurement of prestress loss by overcoring and creep tests on rock and concrete core. The test results were used to establish a creep model to predict prestress losses for a wide range of tunnel pressure and lining conditions (Fig. 5). This model relates, for the particular geometry and rock conditions of the pressure tunnel, material creep factors A and B (for rock and concrete, respectively) to the structural creep parameters V and Z. Knowing any two to these dimensionless parameters, the other two can be deduced. This model was calibrated using the results from the test section and subsequently checked for the constructed tunnels using instrumented sections located as shown in Figure 1.

This simple and practical algorithm of structural creep has two characteristics which enable the application of prestressed linings under exacting design requirements to be assessed:

- o The unknown and complex factors associated with grouting and creep are grouped into a limited number of parameters which can be deduced by back-analysis from the prototype structure.
- o Simplified methods of design, construction control and verification can be applied without attendant sacrifices in accuracy.

6.0 TUNNEL EXCAVATION AND LINING

The following details are provided to emphasize the importance of the tunnel excavation and lining procedures to the success of the prestress grouting operation.

The internal pressure and rock conditions necessitated that every effort be made to minimize the disturbance to the rock and hence the deformations of the tunnels in service. Even with extensive preparatory borehole grouting measures, it was recognized that low rock mass modulus resulting from poor excavation practice could not reliably be restored. Artificially fractured rock around the excavation profile, although grouted, would have a greater potential for creep and hence prestress loss.

Of special significance for the Drakensberg tunnels is the excavation of as circular a profile as possible resulting in a reasonably uniform thickness of concrete lining. The importance



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of the consequent uniformity of grouting prestress has been noted previously. The tunnels were excavated to a circular profile using conventional drill-and-blast methods and three-meter rounds drilled with a four-boom jumbo.

Both smoothwall and presplit blasting techniques were adopted, the latter being applied using reduced round lengths in the poorer areas. These techniques combined with accurate drilling and well balanced charges produced a high standard of excavation (Fig. 6). The additional cost of these measures were more than justified by the practical benefits to the grouting operation.

To minimize the extent of disturbance around the excavation, tensioned rock reinforcement using two-speed resin grouting was placed over the roof and haunch areas to within one meter of the advancing face. Supplementary reinforcement was specified in faulted/jointed areas. Shotcrete was applied where necessary to prevent the deterioration of the argillaceous strata. Tunnel convergence and borehole extensometer measurements indicated well controlled near-elastic stress relaxations.

The excavated profiles conformed reasonably well with the intended shape except in areas where major geological structures were encountered. After excavation of each tunnel was completed, the cross-sectional profiles were measured and upgraded to produce a more uniform lining thickness in order to improve the efficiency of the prestress grouting. The use of slipping and shotcrete backfill over some 40% of the tunnel circumference produced smooth and uniform profiles with no areas greater than 2m x 2m extending more than $\pm 100\text{mm}$ from the average tunnel radius of 3.3m.

Prior to placement of the concrete lining, a debonding film of slaked lime was applied over the rock and shotcrete surfaces. Circumferential grout injection tubes, split into invert and crown rings, equipped with manchetes at 1.5m intervals were spaced at 2.4m intervals (Fig. 7). The manchete sleeves aid the development of the initial grout wedge. Full circle fleece grout arrestors were also nailed to the surface of the excavation. The fleece allows bleed water to pass but retains the cement particles thus creating a barrier to grout travel and permitting some control of the grout front.

The reinforced concrete lining was some 700-800mm thick and cast in monolithic 12m bays. Previous experience with the TIWAG grouting system indicated that control of grout loss through construction joints and the associated drop in effective grouting pressure was extremely difficult. Special inflatable fleece-wrapped pipes were used to control leakage in the circumferential construction joints.

The concrete was designed to have a minimum cube strength of 30 MPa at 28 days. The short term loading modulus at the time of interface grouting was about 30,000 MPa. Full details for the lining design are given in Ref. 10.



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Prior to interface grouting, cavity and in effect, low pressure consolidation grouting were carried out over the length of the tunnels through arrays of six 2m long holes at 3m centers. Grouting pressures up to 1.5 MPa were used.

Over fault zones or zones where significant jointing had been observed, comprehensive water tests were performed to determine the degree of potential loosening at depth (a condition not covered by the test section or the resulting tunnel deformability model). Measured permeabilities were low and subsequent high pressure consolidation grouting using 3m long holes and 5 MPa nominal pressures recorded only limited grout takes.

In spite of these specific treatment measures, prior to grouting, non-uniform conditions remained, particularly in the vicinity of major faults as evidenced by the variable takes during the interface injections. For general consolidation of the rock adjacent to the lining, reliance was placed upon interface grouting.

7.0 CONTROL OF GROUTING OPERATIONS

As previously noted, it was considered necessary to maximize the level of prestress that could be practically achieved in the lining. This was undertaken by utilizing comprehensive diametral convergence measurements (calibrated using embedded instruments) and by varying the maximum required convergence according to the particular conditions at each point along the tunnel.

The process of determining the convergence required to be introduced by the grouting process is outlined in Figure 8. Both internal dynamic pressure and age of concrete vary significantly along the length of the tunnels. The age of the concrete at grouting had a marked effect on the total loss of prestress that is experienced over the life of the tunnels since the time of grouting. The selection of the "required initial prestress" is also tempered somewhat by the strength and hence age of the concrete since in the present instance, the potential problem of rupture of the lining during grouting is the main factor limiting the level of prestress that can be achieved.

Previous experience with the grouting of the lower pressure shafts and also with tunnels in Austria pointed to the need for careful control of convergence to reliably assess the deformational state of the lining. Although uniformity of prestress is inherently better than can normally be achieved by borehole grouting, there is a tendency for bulges to develop in the lining leading to marked non-uniformity in the prestress and a lack of control in the distribution of prestress circumferentially. For this reason, four convergence diameters were monitored at any one cross-section of the tunnel. Cross-sections were spaced at about 6m intervals along the tunnel. Special portable telescopic convergence rods were developed to measure 16 diameters on four cross-sections immediately ahead of the moving grout front (Fig. 9).



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To assist in the interpretation of the lining stresses, embedded vibrating wire strain gauges, gap width monitors, borehole extensometers and tangential stress cells were installed in the initial upstream sections. This afforded a direct and reliable correlation of lining stress with convergence and a better appreciation of variability of lining prestress based on the four convergence measurements.

Other controls include pressure monitoring of valve outlets ahead of the injection plane, clockwork recording of grout pump pressure, and stroke counters to monitor the volume of grout injected at any one section.

8.0 THE GROUTING OPERATION

The interface grouting operation was commenced at the upstream end and progressed continuously along the tunnel completing each 850m length in about 20 days. In this way, the initial difficulties could be sorted out and a feel developed for the operation so as to optimize prestressing in the more critical downstream end. The grout consisted of ordinary portland cement and water initially injected at a ratio of 1:2 (by weight) or wetter to facilitate the opening of the manchettes. It was then gradually thickened to about 1:1 as maturity was reached at each injection point to assist in the pressure buildup.

The grout was injected using four double-acting pumps mounted on a trolley. To maintain a continuity of injection, two or more circumferential rings were pressurized simultaneously using a leap-frog technique. Very high pressures up to 15 MPa were necessary to activate the manchette sleeves, after which pressures in the range 5 to 7 MPa were adequate to develop the required convergences. The average convergence actually induced at a section was varied, according to the required convergence as noted in Section 7 and the uniformity of prestress being achieved.

The grout front exhibited a tendency to progress in individual "fingers", with a concentrated flow developing in the invert. This was mitigated by boosting the pumping volume in the crown preset ring. Careful analysis of the data from the convergence measurements and the two instrumentation sections indicated that as many as 3 or 4 independent inward bulges were produced around the lining by these grout "fingers".

Typical circumferential and longitudinal distributions of tangential prestress at the lining neutral axis are given in Figures 10 and 11. Although prestress is more uniform than that shown in Figure 4, such a result was achieved only after some experience and meticulous control of the grouting and monitoring operations. Additional refinement in the design of the grouting system to improve uniformity of prestress may be possible although a practical limitation may be the intrinsic variability in rock conditions.

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Grout takes averaged 670 kg/m run of tunnel. Based on an in situ density of about 2400 kg/m³, this corresponds to an average total gap width of 14 mm. Six to 7 mm of this was achieved by lining convergence and about 3 mm from radial compression of the rock at an average grout pressure of 5 MPa. Thus the grout absorbed in filling fissures in the disturbed rock was equivalent to 3 - 5 mm of circumferential gap width, allowing for leakages.

The minimal problems experienced with leakages, mainly through stemmed injection boreholes and construction joints, are attributed to the high quality of the lining construction. The careful design and execution of the grouting program resulted in the stringent design requirements for initial prestress to be satisfactorily achieved in all but a few bays of the tunnel. These bays were upgraded by activating the "back-up" rings consecutively installed along the tunnel for such an eventuality.

9.0 POST-GROUTING BEHAVIOR

The post-grouting behavior of the lining was checked using a remotely monitored instrumentation section near the downstream end to validate the design criteria and short term performance predictions with regard to creep and deformability. The extremely linear variation of lining strain with internal pressure noted during watering-up indicated an overall elastic behavior of the lining/rock structure.

The very low rates of leakage (typically initial rates of 0.5 liters/second per tunnel and decreasing to 0.005 liters/second) are indicative of the efficacy of the grouting operation. The instrumentation section is being monitored during project operation to confirm the design predictions for creep and loss of prestress over the operational life of the tunnels.

10.0 CONCLUSIONS

The investigation, design and construction of high head concrete lined tunnels prestressed by interface grouting has been described. Reliable design criteria and construction requirements to achieve an essentially crack-free lining in low modulus rock were satisfactorily developed from a prototype-scale test section.

The major design issues which need to be addressed are the deformability of the concrete/rock/grout system, the maintenance of compressive and elastic stress conditions during operation of the tunnels and the development of a tunnel-specific creep model for the reliable and accurate prediction of long term prestress conditions. The preparation of a high quality lining and the careful execution and monitoring of the interface grouting are significant construction issues.



Documents

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This approach combined with the TIWAG interface grouting technique has extended the range of application of concrete linings for pressure tunnels in terms of internal pressure and rock conditions. The experience indicates however that there is considerable margin for improving the practical technology of interface grouting, particularly with regard to developing the most suitable lining and in the execution of the grouting.

11.0 ACKNOWLEDGEMENTS

The authors wish to thank the Electricity Supply Commission of South Africa for permission to publish this paper. Gibb Hawkins and Partners were responsible for the design of the tunnels and Golder Associates performed investigation, design and construction control of the geotechnical aspects of the tunnel excavation, lining and grouting. The grouting operation was carried out by Rogess, a Joint Venture of Rodio (South Africa) Pty. Ltd., Ground Engineering Ltd., and Shaft Sinkers Ltd. The extensive contributions of the authors' colleagues whose supporting work is reported here are also acknowledged.

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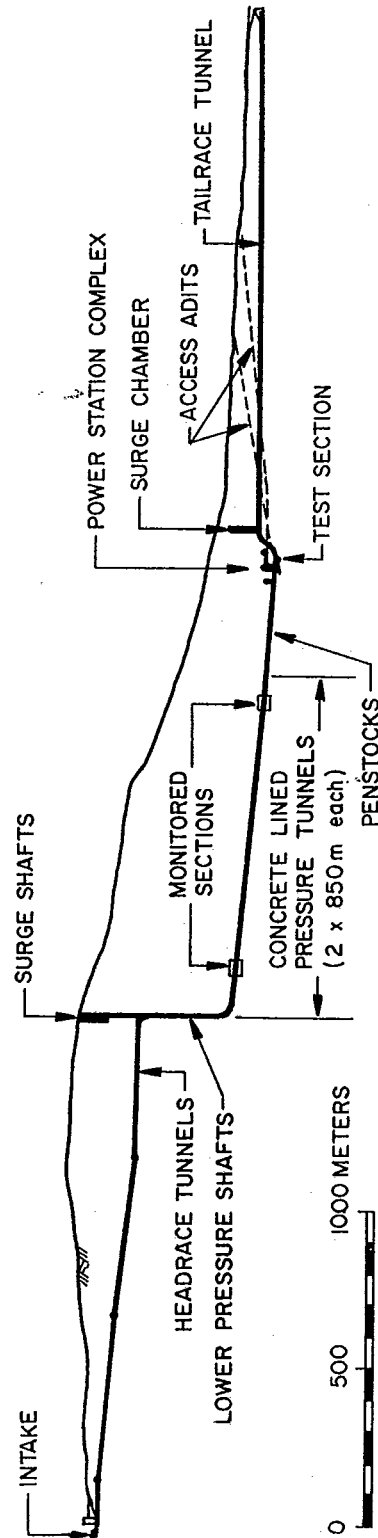


Fig. 1--Elevation along alignment showing major scheme components

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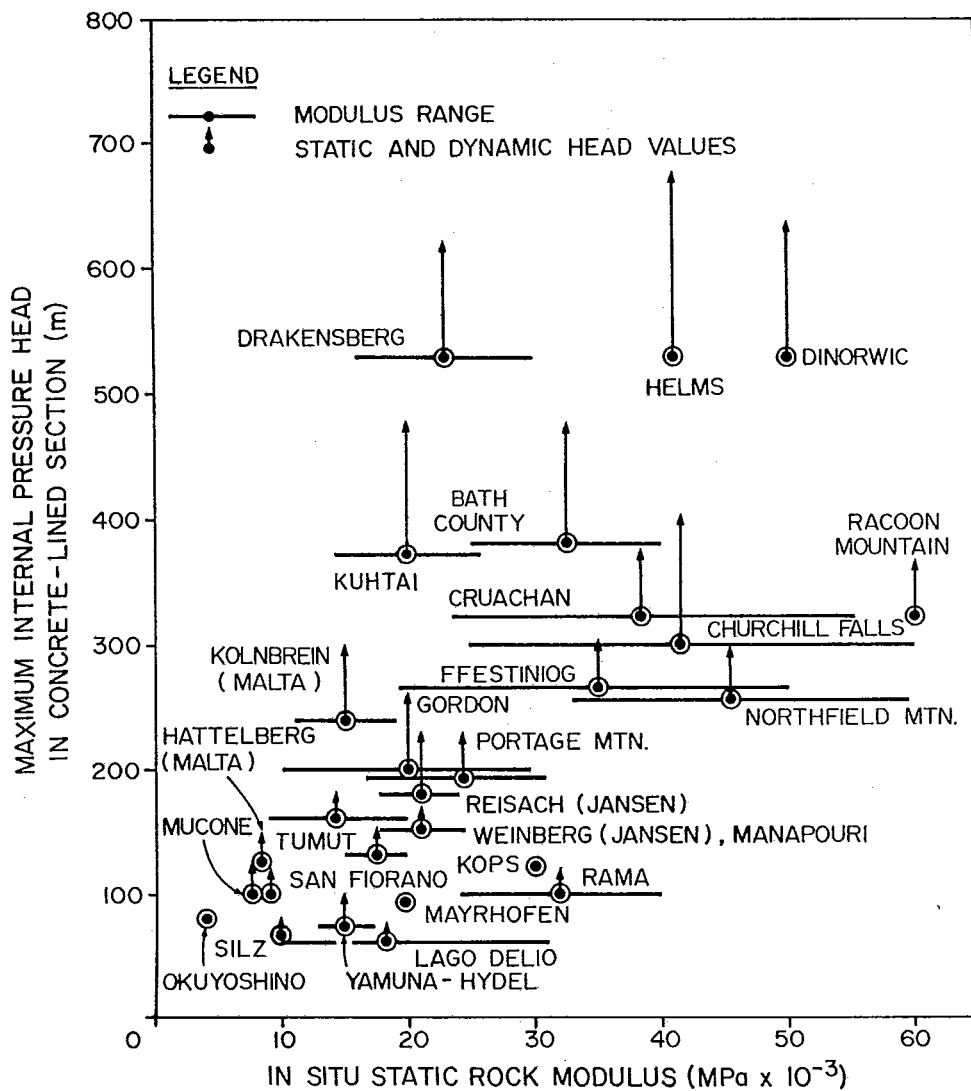


Fig. 2--Comparison of internal pressure and rock modulus for concrete-lined pressure tunnels

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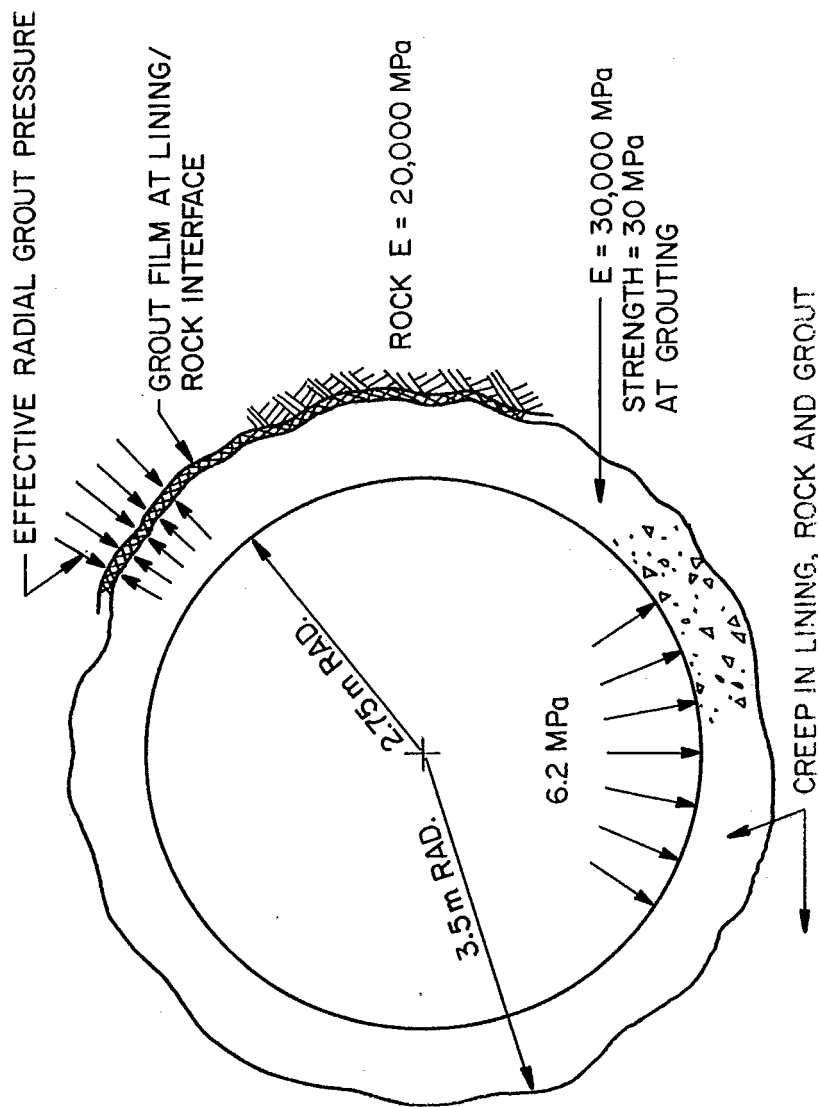


Fig. 3--Tunnel design model for calculation of prestress requirements

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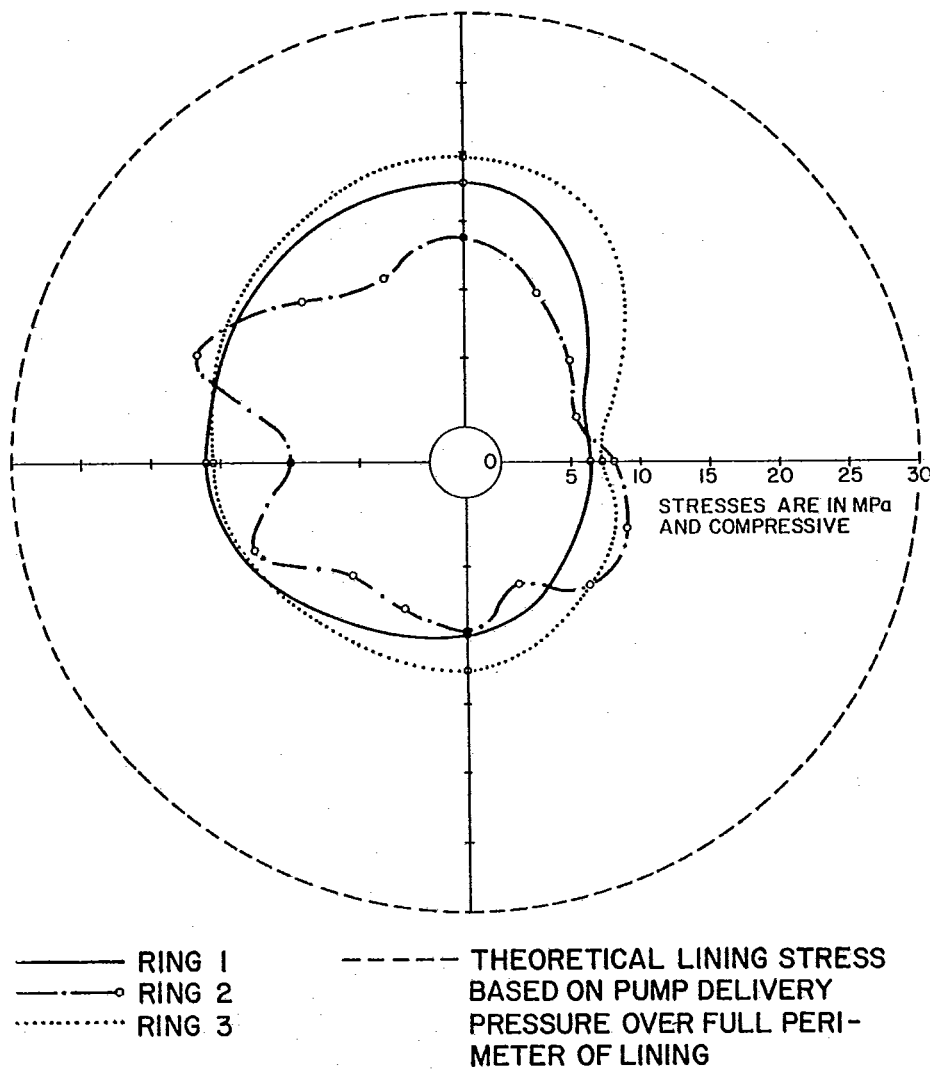


Fig. 4--Tangential lining prestress produced by borehole injection grouting in test section

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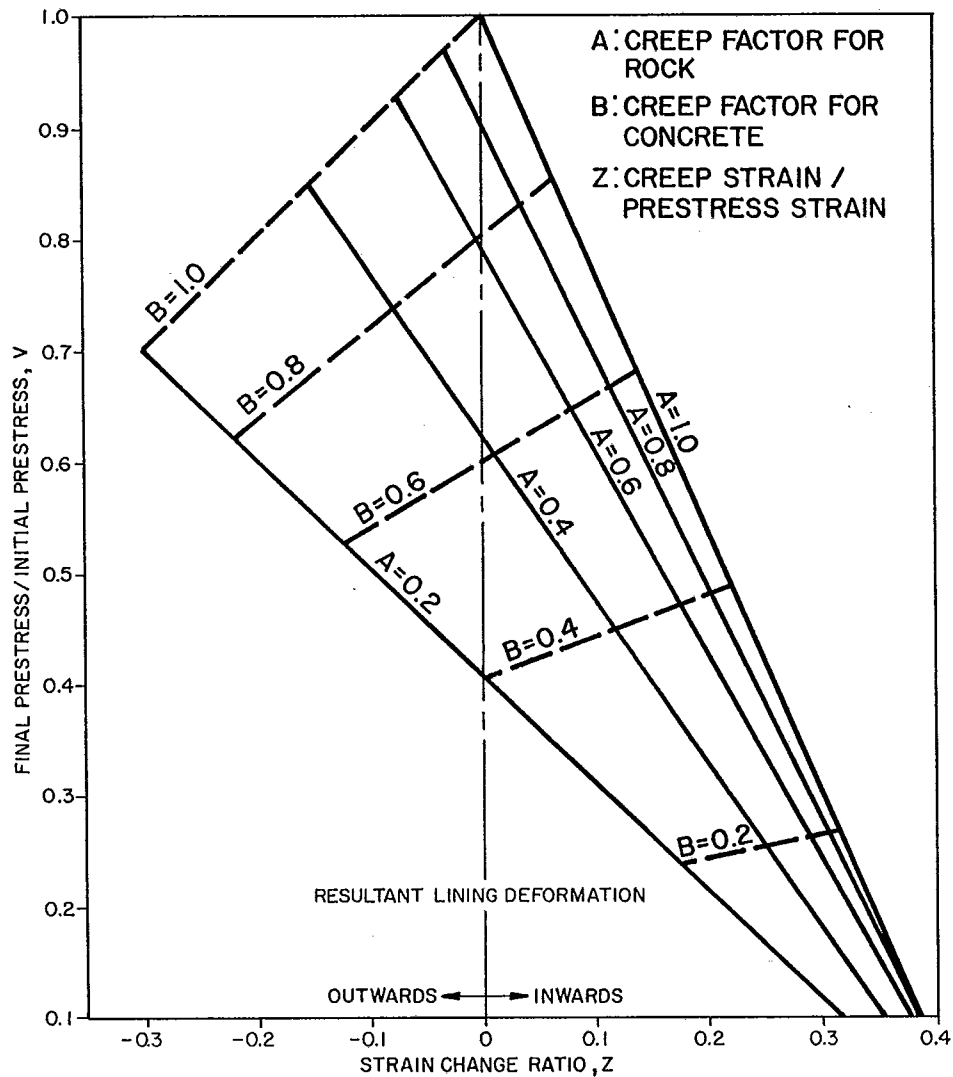


Fig. 5--Structural creep model of pressure tunnels

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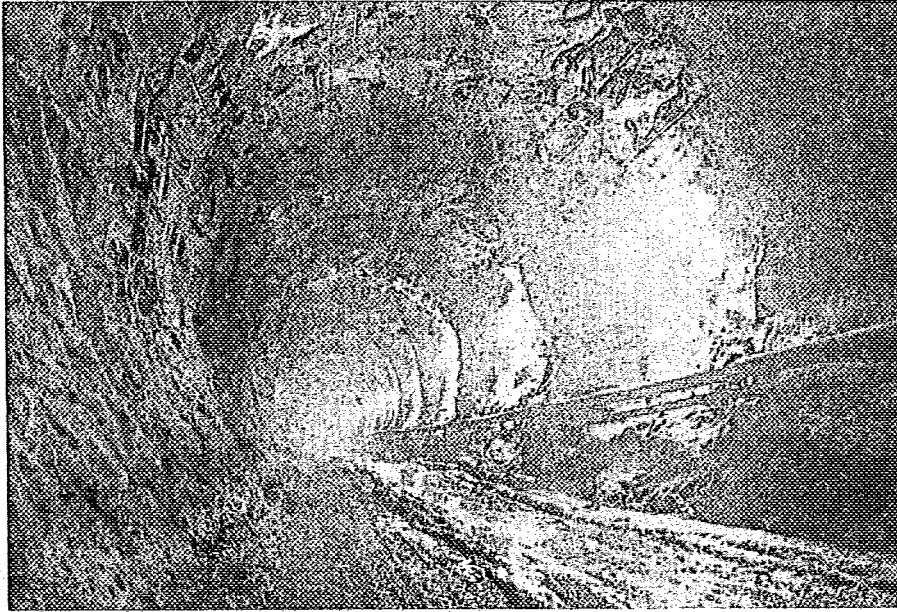


Fig. 6--Typical tunnel excavation profile

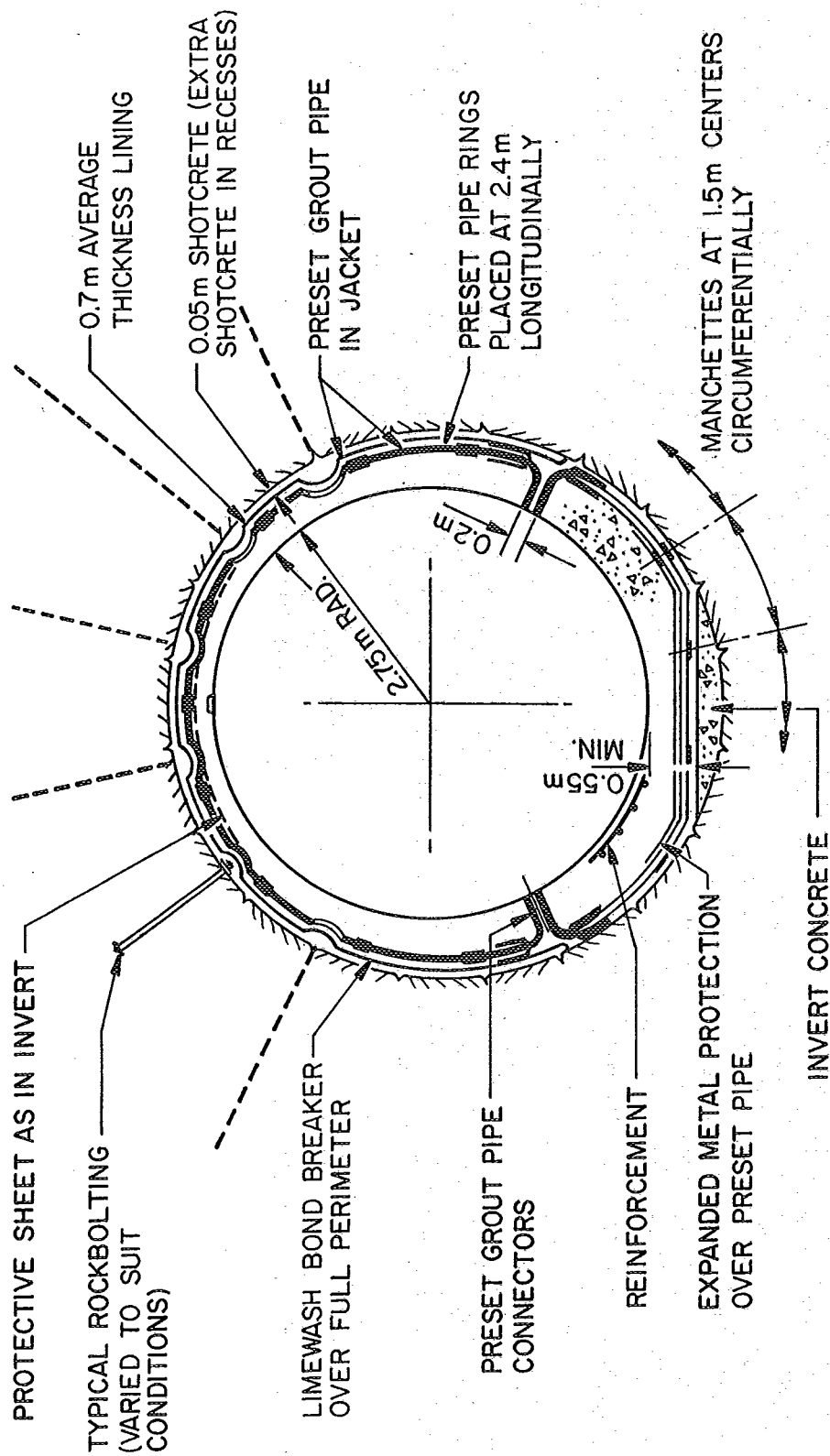


Fig. 7--Cross-sectional view of grouting fixtures

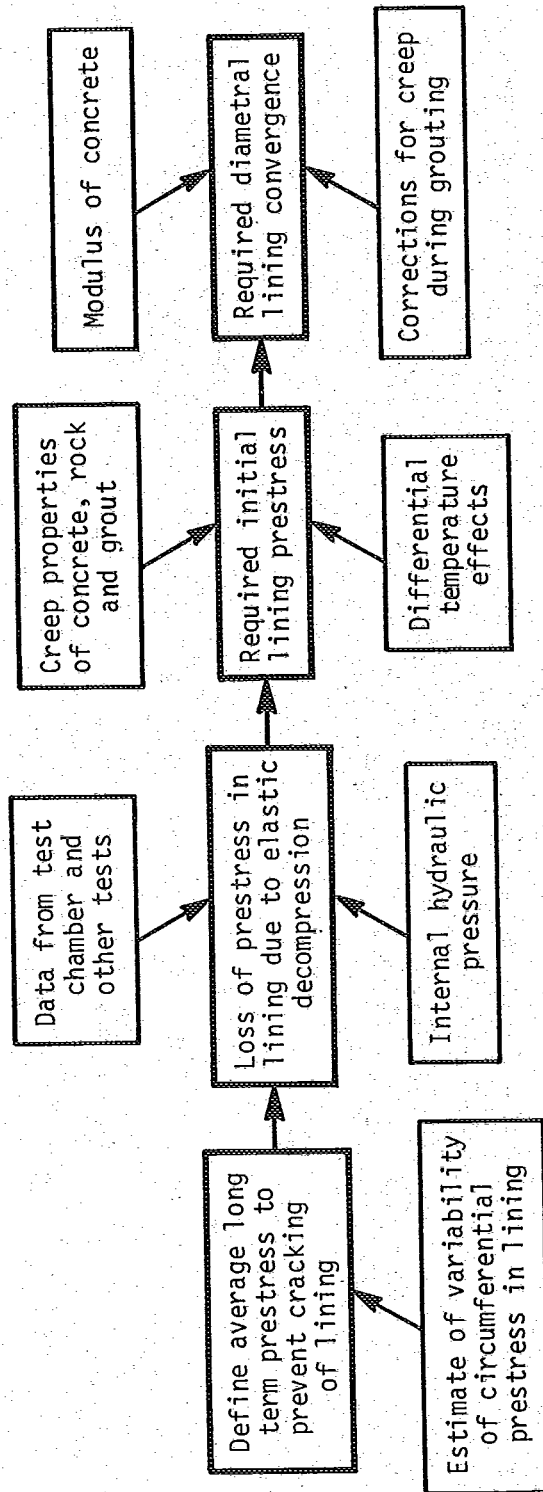


Fig. 8--Determination of required lining convergence for control of grouting

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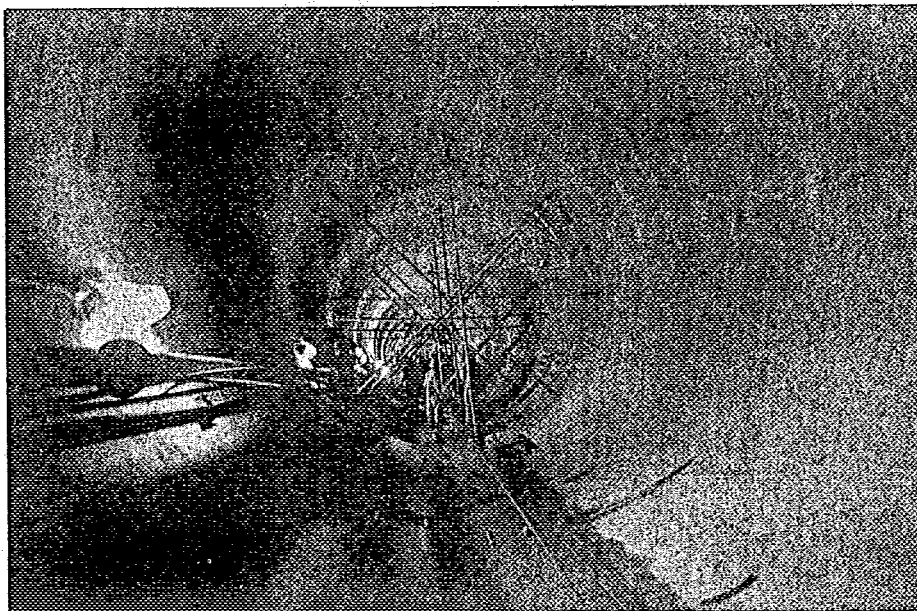


Fig. 9--Convergence monitoring of the grouting operation

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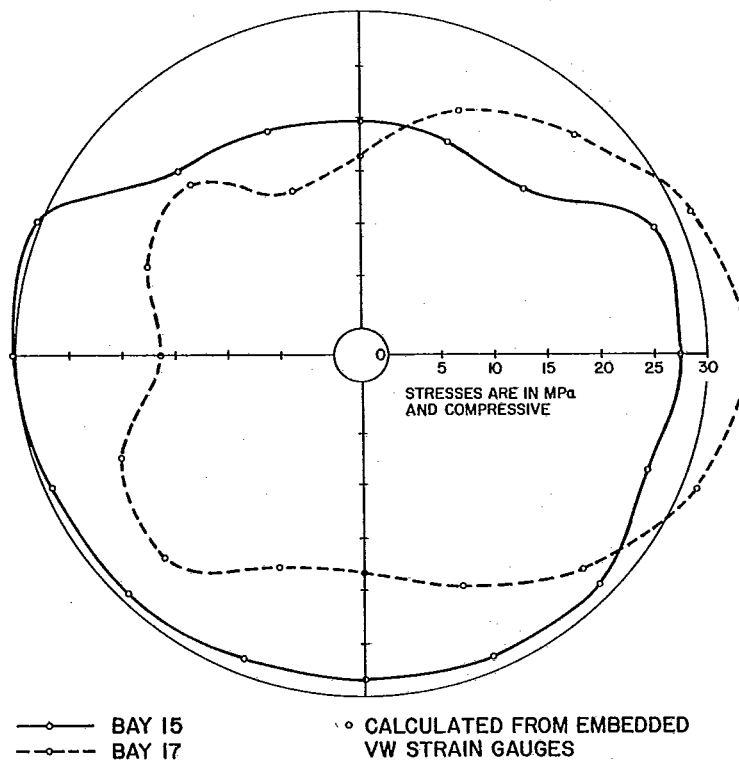


Fig. 10--Typical circumferential distribution of initial lining prestress

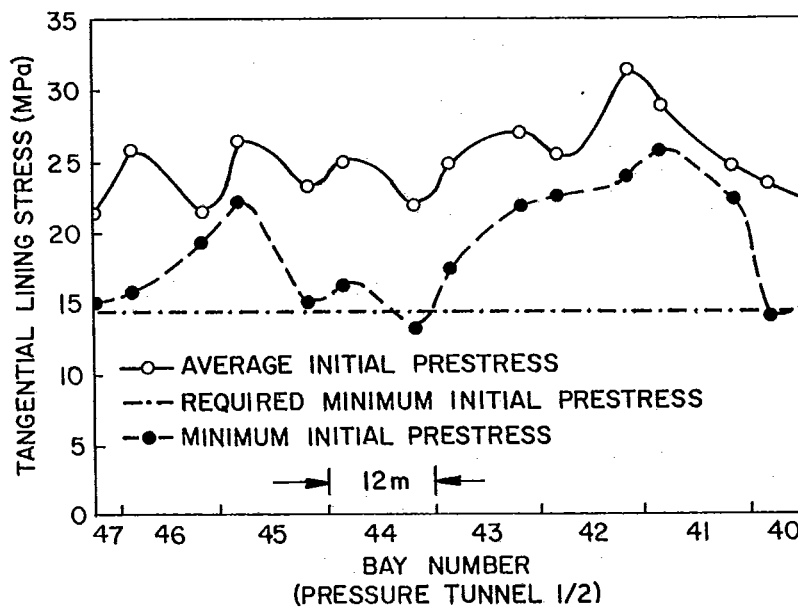


Fig. 11--Typical longitudinal distribution of average and minimum initial prestress

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Changes in Length and Volume of Cementitious Grouts

**By Della M. Roy, M. Perez,
B.E. Scheetz, and P.H. Licastro**

Synopsis: Changes in length and volume of cementitious grouts for use in nuclear waste isolation have been investigated, and compared with the behavior of other mixtures, neat cement pastes, fly-ash containing mixtures, and mixtures with high-range water reducers ('superplasticizers'). Expansive and non-expansive formulations are compared with respect to their changes in length and volume, both at atmospheric pressure and elevated curing conditions, including those simulating placement under pressures of up to 1600 psi hydrostatic.

Keywords: bleeding (concrete); drying shrinkage; ettringite; expansion; fly ash; grout; hydration; plasticizers; pressure; shrinkage; volume change; water-cement ratio; water reducing agents.

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B.E. Scheetz is an Associate Professor of Solid State Science, Materials Research Laboratory, The Pennsylvania State University. Dr. Scheetz has extensive experience in characterization of products resulting from the hydration of cements.

P.H. Licastro is Senior Research Associate at The Pennsylvania State University. He has worked on the development and characterization of materials for borehole plugging and mine shaft sealing, including the optimization of physical, mechanical and chemical properties for site-specific applications.

INTRODUCTION

Origin and Importance of Study

Cementitious mixtures are heterophase, polydisperse, poly-mineral systems. Therefore, the deformation of a microvolume is uneven and this gives rise to the occurrence of internal stresses. It is necessary to understand the interrelation of the physico-chemical processes characteristic of cement hydration, and the microstructural and mechanical phenomena.

The initial objective of this research was to investigate the length changes during hydration of grouts and other cementitious mixtures with different chemical composition, particle characteristics, and various additives and admixtures. It is important to determine length change at the early stage of hardening because structural features developed during this stage may be perpetuated as sources of later cracking and structural defects, particularly in higher water-cement ratio mixtures.

The main objectives were:

a) To compare the length change expressed as percent and the rate of change for cements with different chemical composition as a function of time; b) to find the relationship between volume change and the following parameters: water-cement ratio, specific surface, and bleeding water; c) to determine the influence of fly-



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ash and high-range water reducers ('superplasticizers'); and d) to compare the two techniques (length change and volume change) for the study of length changes, both at atmospheric and elevated pressure.

The ultimate concern of this study is to reduce these data to practical application for the successful completion of geological repositories, designed for radioactive waste isolation.

The two basic types of negative length change (shrinkage) are considered: chemical shrinkage and drying shrinkage. Both are involved with hydration of the cement and the hardening of the grout and both are associated with a continuous change in the internal humidity of cement paste. These must be understood in order to be able to compensate for shrinkage in the proportioning of grouts intended to be non-shrinking or expansive. This study restricts its concern primarily to the early stage (chemical) shrinking or expansion.

Chemical shrinkage is the reduction in total volume during hydration. The final volume of the system, residual unhydrated cement, residual water, hydrated cement, is usually smaller than the sum of the initial volumes of unhydrated cement and water before mixing (1,2). If the cement paste is protected against evaporation it shrinks due to the internal desiccation of the mass caused by hydration of the cement, reducing the internal humidity.

If water is not available from an external source, the system will tend to shrink and develop stress in the skeleton of the grout. The term autogenous shrinkage has been used to describe shrinkage occurring in sealed specimens to which no external moisture is available and from which no moisture is lost. The term plastic shrinkage is used (3,4) to describe the shrinkage observed during approximately the first five to seven hours after mixing. However, this usage is confusing since the term plastic shrinkage is much more widely used to refer to negative volume change before hardening due to excessive loss of water. Others have referred to it as first shrinkage (5,6). Buil and Baron (5) have studied the second shrinkage which appears after setting during the hardening of the cement paste. Other types of chemical shrinkage include carbonation shrinkage, but this will be of little importance in applications where there is no exposure to carbon dioxide.

Setter and Roy (1) described the difference between inner and bulk chemical shrinkage, the first being a consequence of the decrease in specific volume of the water. Bulk chemical shrinkage is a reduction in the overall volume of the mass and is a result of setting and the hydration process.

Added to the phenomenon in cement itself are the effects of various additives used for both technical and economic reasons.

The effects of water-reducing mixtures upon cement suspen-