The properties of cemented silicated backfill for use in narrow, hard-rock, tabular mines

by G.E. Blight* and A.J.S. Spearing†

Synopsis

Several aspects of the behaviour and properties of backfills consisting of cemented cyclone-underflow tailings are illustrated by means of examples, and guidelines are given on the optimization of the performance of cemented fills.

The stretch characteristics of the geofabric used to contain freshly placed backfill are described, as well as the relationship between these characteristics and the forces acting on retaining barricades. The properties required to achieve a self-supporting, stable free face in cemented backfill are explored, and a graphic relationship between stable height and shear strength is presented and compared with that for uncemented (classified) tailings.

After a description of the characteristics of Witwatersrand quartzite gold-tailings backfill and commonly used cements, the process of drainage and air entry into backfill is outlined. Following this, the early-age gain in strength and the shear-strength characteristics, as well as the compressibility characteristics of cemented backfill, are described.

Two phenomena that are potentially damaging to cemented backfill are seismic effects (or blasting acceleration) and the closure that takes place during the curing of cement. Investigations show that neither of these phenomena is actually significantly deleterious to gold-tailings backfill: liquefaction does not occur as a result of seismicity, and hydration of cemented fill is not significantly impeded by closure.

Introduction

The use of backfill in narrow, hard-rock, tabular mines in the Witwatersrand Basin has a number of advantages, which are either actual or realizable in the near future. Some of the advantages are that backfill

- ➤ reduces damage caused in stopes by rockbursts
- ➤ helps reduce the effective stoping width and, when used with good temporary face support, improves the stability of the stope face
- ➤ improves the stability of gullies, escape ways, and dip travelling ways
- ➤ increases the overall reef extraction by allowing a reduction in size, or the elimination, of reef pillars
- ➤ reduces labour for the installation of stope support
- ➤ reduces the fire hazard by eliminating timber as an underground support material
- ➤ reduces the entry of heat into the workings by sealing off most of the exposed rock surface, and thereby reducing the costs of ventilation and cooling.

Also, backfilling has some positive influence on the surface environment. The pressure on forest resources is reduced by the decreased demand for mining timber, and the volume of tailings requiring disposal in dams on surface is significantly reduced[1].

Backfilling has a long history in the Witwatersrand Basin, having been used intermittently from the 1890s. Until the early 1950s, backfilling, using mostly sand and hand-packed waste, was a widespread practice. However, because of the rising cost of backfilling and some unfortunate accidents, the practice was phased out. Research began again in the mid-1970s[2,3], and the use of backfill of many types (e.g. cemented and uncemented full plant tailings with and without dewatering, cemented and uncemented cyclone-underflow tailings) has increased progressively in the past twenty years[4,5].

This paper deals with the properties of backfills consisting of cemented cyclone-underflow tailings and of total tailings. The use of this material has many advantages, some of which are as follows.

- ➤ Cyclone-underflow tailings are significantly coarser than full plant tailings, and the fill therefore drains relatively quickly and gains strength quickly.

* *Department of Civil and Environmental Engineering, University of the Witwatersrand*

† *Gold and Uranium Division, Anglo American Corporation of South Africa Ltd, P.O. Box 61587, Mashalltown, 2107.* Present addess: *Rock Mechanics and Backfill Consultancy cc. P.O. Box 2141, Honeydew, 2040.*

 Paper received Jan. 1995; revised paper received Aug. 1995.

- Total tailings can also be used, thus further reducing the environmental impact of surface tailings dams.
- The addition of a binder results in an even more rapid gain in strength, and the fill therefore develops significant stiffness and carrying capacity at an early age.
- The addition of a gelling agent with the binder can greatly reduce run-off water, as well as solids losses. This significantly reduces the post-filling shrinkage and improves the development of early strength and stiffness. Reduced run-off also has the potential to reduce the contamination of mine water by cyanide.

Forces on geofabric barricades

The most critical phases of the life of a backfill from the point of view of safety are the period immediately after filling, when the fill is supported by a geofabric-covered barricade, and that at the time of the first blast after placing, when the barricade might be damaged and the backfill still has insufficient strength to be self-supporting. In addition, a seismic event can occur at any time. Hence, the sooner the free face of the backfill can develop sufficient shear strength to be self-supporting, the less chance there will be of an accident involving the stability of the fill.

Immediately after filling, the barricade must carry the load from the heavy fluid (the backfill slurry) retained, which has a unit weight of about 17 kN/m^3 on placement, depending on the relative density of the slurry. In a horizontal flat stope of width h(m), the pressure, p, at the base of the barricade will be ($17h$) kPa, and the thrust, P, on the barricade per unit length will be ($8.5h^2$) kN/m. In a dipping stope, the pressures and thrusts will be greater, but these values will be used illustratively, as in Table I.

For a horizontal spacing of supports of b, the load on each support will be

$$P = 8.5\,bh^2. \qquad [1]$$

The tension in the geofabric will depend on the extent to which it will stretch or bulge under pressure. The greater the bulge, the less the tension. Figure 1 shows measured relationships between the stretch of two geofabrics (one woven, one knitted) and the tension in the fabric. It will be seen that knitted geofabrics stretch considerably more than woven geofabrics, and also undergo very much more stretch at negligible tensions. Geofabrics also creep under sustained tension, as indicated by the small plateau at the top of each diagram of tension versus stretch. Knitted geofabrics also creep more than woven ones.

If a tension of 5 kN/m is taken as an illustrative value, woven geofabrics will stretch by as much as 10 per cent. If the geofabric is supported at a minimum horizontal interval, b, the outward bulge between supports will be given by

$$B - b\left(\frac{2\varepsilon}{5}\right)^{1/2}, \qquad [2]$$

where ε is the stretch of the geofabric. Table IB shows the bulges to be expected for various support spacings for woven and knitted geofabrics. It appears that a support spacing of more than 1 m will result in the bulging of knitted geofabrics that may be unacceptably large.

Excessive bulge can interfere with scraping operations and cause damage to the geofabric. An excessive bulge can dislodge packs on the down-dip gully side of the backfill structure, especially as closure occurs. To reduce the adverse effects of geofabric bulging, the supports should generally be placed no further than 1.0 m apart.

The geofabric tension, T per unit length, corresponding to a particular pressure, p, can be calculated from the expression

$$T = \frac{p(b^2 + 4B^2)}{16B}. \qquad [3]$$

Equations [2] and [3] have to be solved iteratively by use of the characteristic tension-versus-stretch (T versus ε) curve for the geofabric under consideration (e.g. Figure 1).

As an example, suppose a maximum pressure of 50 kPa has to be supported on a woven-geofabric barricade with a horizontal support spacing of 1 m. If ε is taken as 1 per cent, B will be 90 mm (from Table IB). Hence, from equation [2],

$T = 35.8$ kN/m.

Depending on the geofabric characteristic, this may be too high a value. To reduce T, the only option is to increase bulge B (equation [2]), which can be done either by increasing the support spacing, b, or allowing some slack in the geofabric. Neither route is a good one: an increase in b will result in greater loads on the barricade supports, and the allowance of slack in the geofabric will result in tensions that cannot be predicted. Hence, it may be preferable to use a knitted geofabric. In this case, with ε as 10 per cent, B becomes 200 mm, with b = 1 m and T = 18 kN/m.

A reduction in spacing b will reduce the thrust, P, on the supports, but will increase the tension in the geofabric, T. For a safe barricade design, all these factors must be considered in relation to the characteristics of the geofabric. Geofabrics

Table I

Pressures and thrusts in backfill

A			B		
Stope width, h m	Pressure at base of barricade, p kPa	Thrust on barricade per m kN/m	Spacing of supports, b m	Bulge of geofabric under pressure of 50 kPa, B mm	
				Woven	Knitted
1	17	8.5	1.0	90	200
2	34	34.0	1.5	135	300
3	51	76.5	2.0	180	400

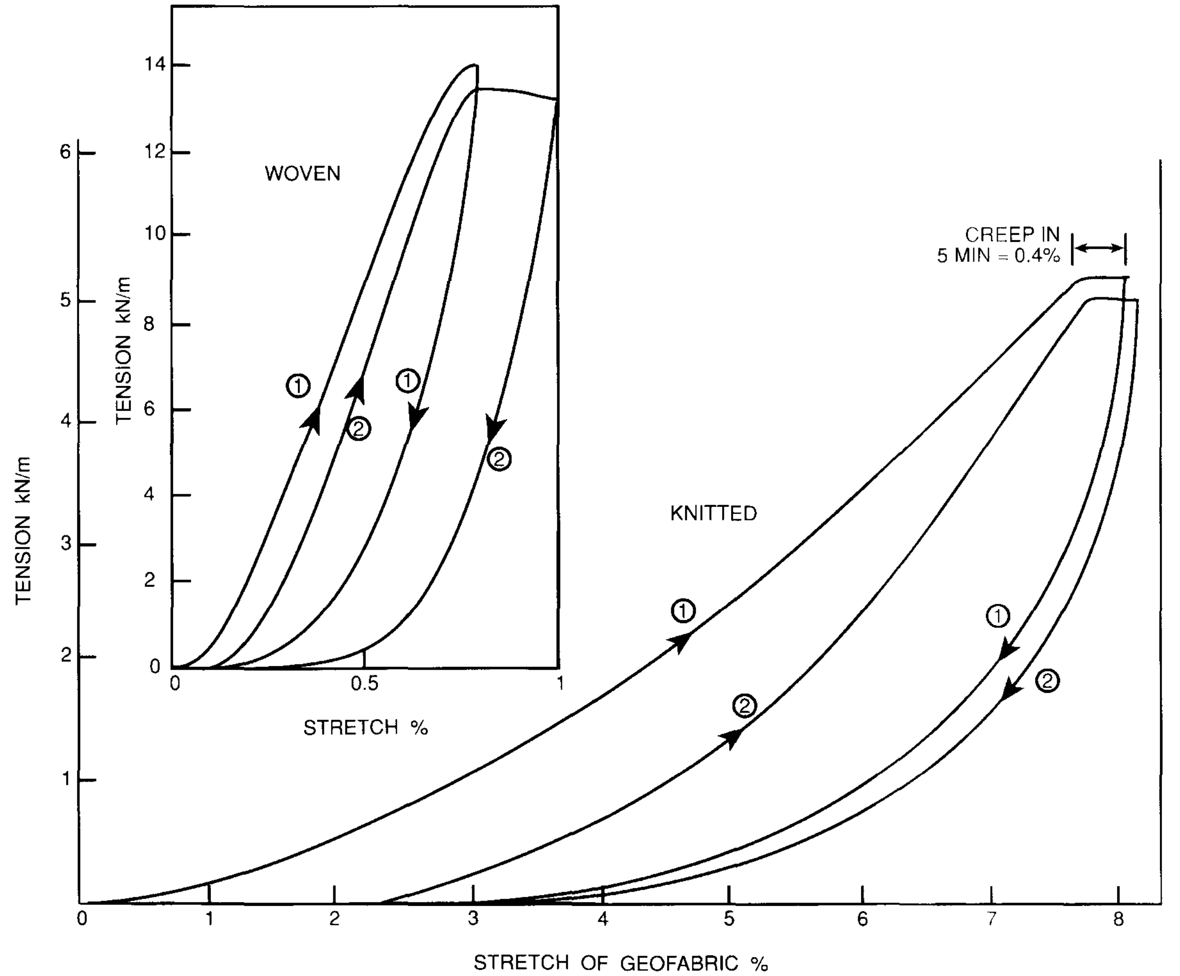

Figure 1—Typical tension-versus-stretch characteristics for geofabrics

should not be used without laboratory testing of their properties, and care should be taken that any seams do not weaken the geofabric.

Relationship between backfill early strength and height of a stable free face

If a free face fails, it will fail by the sliding out of a wedge of material, as indicated in Figure 2. By consideration of the equilibrium of this wedge, the height of a stable free face can be shown to be given by

$$h = \frac{2c'}{F\sin\alpha\,[\rho\, a\ \cos\alpha\left(1-\cot\alpha\,\dfrac{\tan\phi'}{F}\right)}. \qquad [4]$$

In equation [4], c' and ϕ' are the shear strength parameters in the Coulomb strength equation

$$s = c' + (\sigma\text{-}u)\tan\phi', \qquad [5]$$

where s is the shear strength
σ is the normal stress on the plane of shearing
u is the pore-water pressure.

F is the factor of safety of the face against failure, ρ is the density of the backfill in kg/m^3, a is a vertical acceleration that has a minimum value of g, the acceleration due to gravity (10 m/s^2), but could be larger as a result of the effects of blasting and seismic action. If ϕ' is taken conservatively as 30 degrees, α as 60 degrees, F as 1.2, and ρ as 1800 kg/m^3, then equation [4] becomes

$$h = 2.96\ \frac{c'}{a} \doteq \frac{3c'}{a}. \qquad [4a]$$

If equation [4] is modified for strengths measured by means of the vane shear test, [4] becomes

$$h = \frac{4s}{F\,a\rho\ \sin\ 2\alpha}, \qquad [4b]$$

which, for α = 60 degrees and F = 1.2, becomes

$$h = \frac{2.14s}{a} \doteq \frac{2s}{a}. \qquad [4c]$$

The relationship between vane and triaxial strengths has been established by comparative tests.

Equations [4a] and [4c] can be plotted for various values of a, as shown in Figure 2. Figure 2 can be used in the design of adequate backfill performance as a prerequisite to trials in a mine.

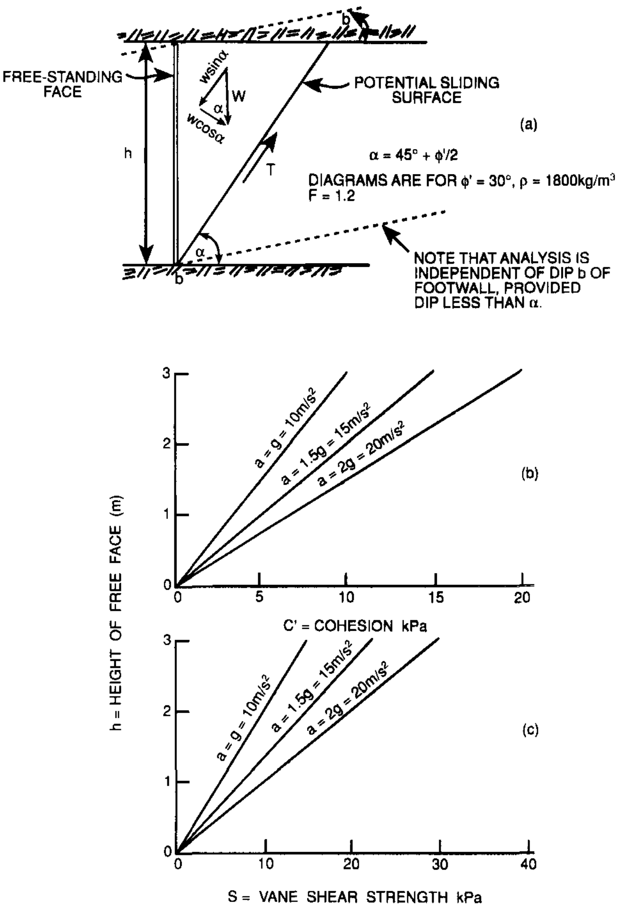

Figure 2—Relationship between the height of a free face and the required shear strength

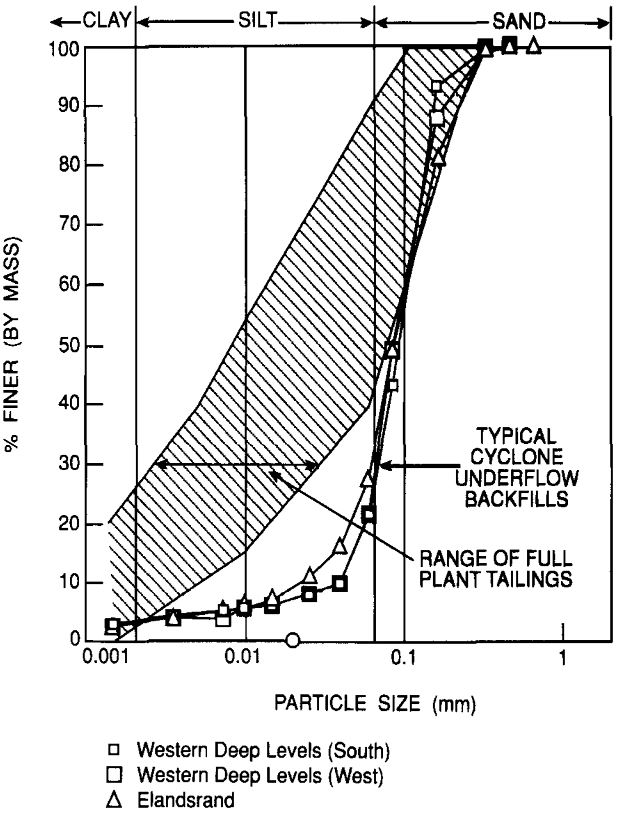

Figure 3—Typical particle-size ranges for backfills consisting of full plant tailings[6] and cyclone underflow

Backfill material and cements or binders

Backfill may consist of full plant (or total) tailings or the coarser fraction of the tailings separated as underflow by cycloning (classifying). Cyclone-underflow backfill has the advantage of draining more quickly, and therefore of gaining strength and stability in a shorter time after being placed (if the finer cementitious materials do not drain out preferentially and block the pores of the geofabric). Figure 3 shows the range of particle-size distributions for full plant tailings recorded by Blight and Steffen[6], as well as the particle-size distributions of three typical cyclone-underflow fill materials. Whereas most full gold-plant tailings consist predominantly of silt-size particles, cyclone-underflow fills consist predominantly of sands. Mineralogically, the gold tailings consist mainly of milled quartz with a minor content of micas (such as chlorite) and clays (such as pyrophyllite).

The binders available and commonly used in South Africa for cementing backfill are as follows:

- ordinary Portland cement (PC)
- ground granulated blastfurnace slag (GGBS), trade named Slagment
- fly ash (FA).

Of the three listed here, PC is a so-called hydraulic cement that sets and hardens on the addition of water. GGBS and FA are pozzolans, i.e. materials that will not harden by themselves but, when activated by certain chemicals, will harden to form cementitious products. The commonest activator is PC itself, but hydrated lime is also used to activate GGBS; 15 per cent GGBS and FA are often used as cement extenders in commercially available blended cements. Portland blastfurnace cement (PBFC), which is a 50:50 mix of PC and GGBS, is also available commercially. In the case of PC and GGBS blends, the GGBS is activated by the calcium hydroxide (hydrated lime) released by the PC as it hydrates and hardens. FA is activated by the addition to PC in a similar way. Another substance that can be used to activate GGBS and FA is sodium hydroxide (caustic soda). (However, this is not often used since it is dangerous to handle.) Whereas GGBS activated by PC or lime hardens more slowly than PC on its own, GGBS activated by lime and PC can gain strength more rapidly than PC. This blend has therefore become a widely used binder in cemented backfill. Many of the data presented in this paper relate to backfill cemented by means of PC and lime-activated GGBS. Sodium silicate or water glass (Fillset system*) can also be added to cemented backfill since it is able to take up and retain excess water in the pores of the fill, acting as a gelling agent and reducing the quantity of water lost from the fill in the hours after placement.

All of the binders mentioned have chemical constitutions that are related. Figure 4 compares the chemistry of PC, GGBS, and FA. In the diagram, C_2S and C_3S (dicalcium and tricalcium silicate respectively) are basic components of PC.

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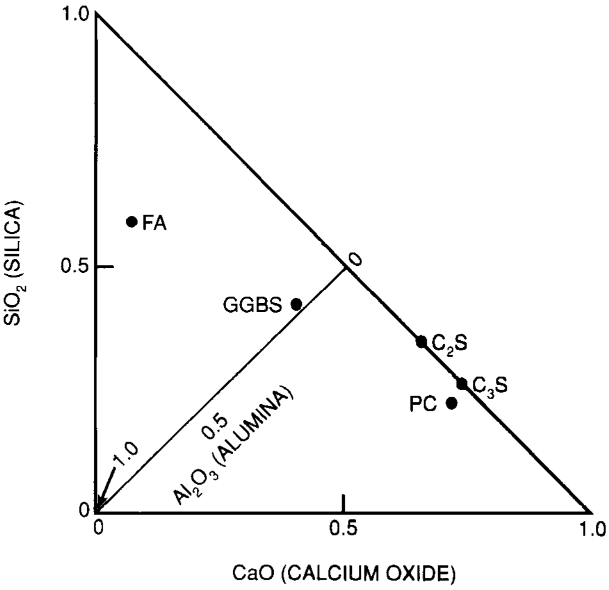

Figure 4—Proportions of lime, silica, and alumina in PC (di- and tricalcium silicate), GGBS, and FA (after Addis[7])

To provide as coherent a picture as possible of the properties of cemented backfill, the illustrative examples as far as possible refer to fill from the same mine (Western Deep Levels). However, data obtained for cycloned fill material from other mines are consistent with those for Western Deep Levels, and results for fills from other mines are also introduced in the paper.

The drainage of backfill

Backfill is usually placed (gravity being used as the transportation mechanism) as a viscous slurry having a relative density (RD) of 1,7 to 1,8 (unit weight of 17 to 18 kN/m^3). Excess water has to drain from the slurry for it to become a frictional solid, with the solid particles in contact and interlocked. If a binder or cement has been used, then, as it hydrates and develops chemical bonds, the interlocked particles become cemented together and the strength can increase considerably.

The resistance to compression of a fill arises largely from its shear strength (which is described by equation [5]). The cohesion, c', is the strength that arises from bonds at interparticle contacts. Except at very low stresses (e.g. in the vicinity of a free face), the term [$(\sigma\text{-}u)$ tan ϕ'] will be far larger than c'. The effective stress, $(\sigma\text{-}u)$, takes account of the strength-reducing effect of the pore-water pressure. In a just-placed fill, u at any point will be positive and equal to a head of water of the same height as the height of the fill above the point. In a fully drained fill, in which a free water surface exists at the level of the footwall, u will be negative and have a value equal to a head of water that is the same height as the height of any point in the fill above the footwall. Figure 5 illustrates the distributions of pore pressure, u, total stress, σ, and effective stress, $(\sigma\text{-}u)$, shortly after placement and after the completion of drainage of a fill. It is important that drainage be completed as soon as possible since it is only in a drained fill that resistance to the stresses caused by blasting and seismic events will be established. Cementitious bonds will grow rapidly in strength only in drained fill.

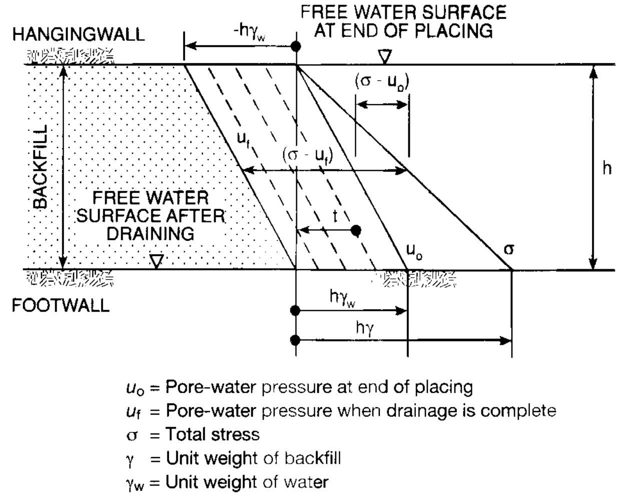

Figure 5—Diagram showing stresses in a backfill after placement and during draining

The drainage process can be divided into two phases. In the first, excess water leaves the fill by draining from free surfaces, and the particles move closer together and into contact. However, the pore spaces in the fill remain fully saturated with water at this stage. The second phase starts once a threshold value of negative pore pressure has been reached, and air starts to enter the pores of the fill, displacing more water. This threshold negative pore pressure is known as the air entry pressure because it is numerically equal to the air pressure that would start to displace water from the pores if the pore-water pressure were atmospheric.

Figure 6 shows drainage-time curves measured in the laboratory for a specimen of cyclone-underflow backfill from Western Deep Levels. On the horizontal axis (time scale) of the figure, the actual times are given in minutes for the specimen (38 mm in diameter by 76 mm long), as well as time factors (in days per m^2) to allow for the dimensions of the specimen. The values of $\Delta\sigma$ shown in Figure 6 represent the increments of all-round stress, σ, applied to the specimen to cause the drainage to take place. It should be noted that drainage occurs more slowly and with a reduced volume as the applied stress is progressively increased and the solid particles move closer together, thus reducing the void space in the fill.

On the basis of Figure 6, the drainage of water from a panel of backfill 2 m long (on strike) with a stope width of 1.5 m can be calculated to take about 1 day to occur. As the pore pressure moves into the negative range (Figure 5) and becomes numerically equal to the air entry pressure, air will start to enter the pores of the fill, and more water will drain out, thus prolonging the period of initial drainage.

Figure 7 illustrates the principle of air entry and drainage by means of a test on a specimen of full plant tailings taken as an undisturbed sample from the beach of a tailings dam. The air entry pressure is the pressure at which the air permeability of the specimen ceases to be zero. In this example, it will be seen that the air entry pressure for flow across the layers of the specimen (i.e. for the vertical flow of air *in situ*) is more than twice that for flow along the layers. Figure 7 also indicates that the air entry pressure is related to

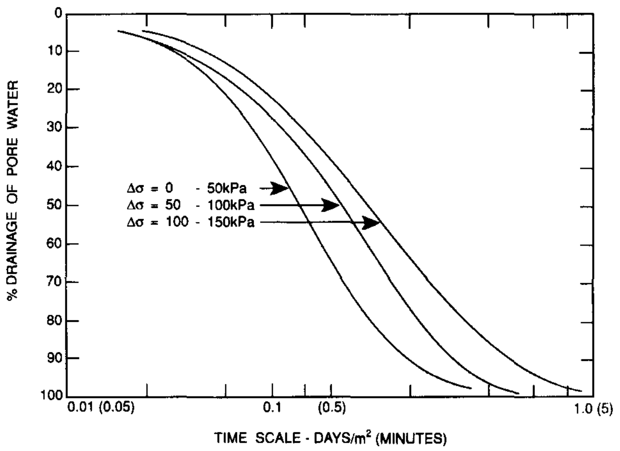

Figure 6—Laboratory drainage curves for a cyclone-underflow backfill from Western Deep Levels

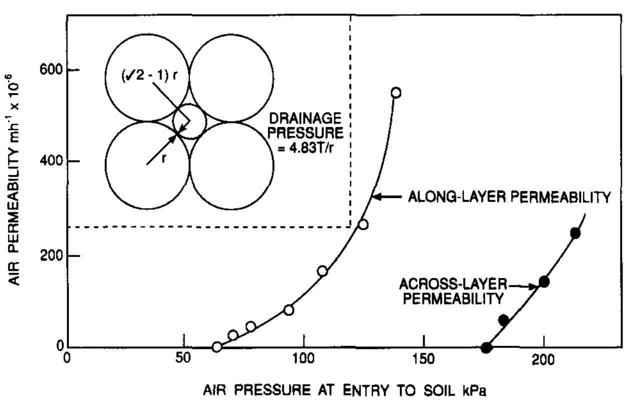

Figure 7—Air entry into a layered specimen of full plant tailings (after Blight[8])

the sizes of the coarsest pores in the material. (In Figure 7, T is the surface tension of water.) Water will be expelled first from the coarsest pores at the free surface of the backfill. Thereafter, successively smaller pores will drain as the pore-water pressure becomes more negative.

Figure 8 shows drainage curves for air entering a specimen of backfill consisting of cyclone underflow from Western Deep Levels. As the figure shows, the air entry pressure for a cyclone-underflow backfill is less than 10 kPa (it is actually about 5 kPa), as compared with about 50 kPa for full plant tailings (Figure 7). Drainage occurs quickly at first, at about the same rate as that for the saturated material (Figure 6). The rate of drainage then drops, but continues slowly for a long period as the air gradually penetrates the pores of the fill. Figure 8 also shows that, if the air pressure is subsequently increased (which is equivalent to making the pore-water pressure more negative), there will be a fresh surge of drainage as the pores that were too fine to drain under an air pressure of 10 kPa are now unable to resist drainage at 20 kPa. In an actual fill, the increase in air pressure shown in Figure 8 would be equivalent to the movement of the pore-water pressure to a more negative value.

As noted earlier, full consolidation of a panel of cycloned fill takes about 24 hours to occur, and thereafter slow

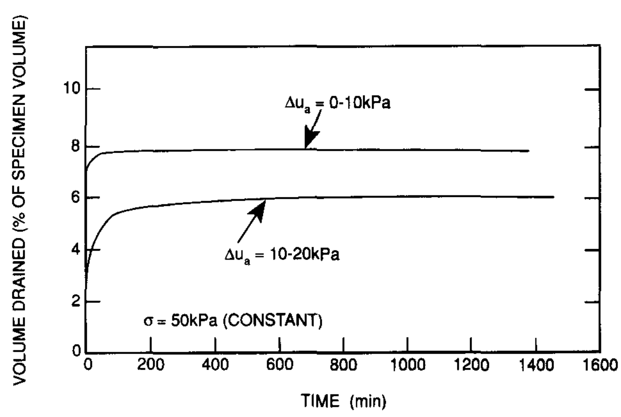

Figure 8—Drainage curves for air entering a cyclone-underflow backfill from Western Deep Levels

drainage, as a result of the displacement of water by air entry, will continue. From Figure 5, for a stope width of 3 m, the mean vertical effective stress after consolidation will be

$$18 \times 1.5 + 1.5 \times 10 = 42 \text{ kPa}.$$

The effective stress on the potential shear plane of the sliding wedge would be about half of that, or 20 kPa. Hence, the shear strength required on this surface would be about 12 kPa (if $\phi' = 30$ degrees). Hence, such a face would have a low margin of safety if the barricade were removed. Because the height of a free-standing face, h, in equation [4c], is linearly related to s, and s in turn is linearly related to h, no free face in a consolidated uncemented cycloned fill would be very stable under this condition. In practice it has been found that stope faces of up to 1.5 m in height are sufficiently stable at the completion of consolidation. For heights above this, the strength must be augmented by cementation of the fill.

Early strength of cemented backfill

At early times after placements, before chemical bonds from the cement have developed appreciably, the strength of backfill will arise entirely from the term

$$(\sigma\text{-}u) \tan \phi'.$$

Because the total stress in the fill, σ, will be small until some compression has occurred as a result of closure, the negative pore pressure ($-u$) can be expected to provide a significant component of the early strength. For a fully drained fill (Figure 5) when $h = 1.5$ m, the average value of u would be -7.5 kPa and the average value of σ would be 9 kPa. Hence,

$$\sigma\text{-}u = 9+7.5 = 16.5 \text{ kPa}.$$

The average shear strength would be about 12 kPa (i.e. 16.5 tan ϕ') if $\phi' = 36$ degrees. Evaporation of water from the free surfaces of the fill can result in more highly negative values for the pore-water pressure than the 7.5 kPa quoted here.

Figure 9(a) illustrates the growth of shear strength with time for a specimen of uncemented cycloned backfill. For the strength measurements, each specimen of the backfill slurry was placed in a standard soil-compaction mould (CBR mould) of 150 mm diameter by 150 mm high. The moulds have perforated bases, which were lined with a disc of geofabric to allow water to drain from the backfill. Once the excess water had drained from the specimens, evaporation from their upper surfaces caused the pore-water pressure to slowly become more negative. The shear strengths were measured by means of a hand-held shear vane. As Figure 9(a) shows, little strength developed during the first 24 hours after the placement. Thereafter, the strength developed more rapidly, although two of the specimens did not reach the expected fully drained strength of 12 kPa (see earlier) within 96 hours of placement. In contrast, Figure 9(b) shows the growth of vane shear strength for the same backfill cemented with various percentages of GGBS activated with lime and PC. (This particular binder is tradenamed Fillcem.) With the addition of a binder, the benchmark strength of 12 kPa can be reached within 5 hours of the placement.

Shear strength of cemented backfill

Figure 10 shows the results of a series of shear-strength measurements on a cycloned fill from Western Deep Levels cemented with various proportions of GGBS activated with lime and PC. The strengths were measured by means of a shear-box apparatus at 24 hours after placement, and are in the form given by equation [5], i.e.

$$s = c' + (\sigma - u)\tan\phi'. \quad [5]$$

It will be noted that the angle of shearing resistance, ϕ', is constant for all binder contents, and that the cohesion increases with increasing binder content. However, it should also be noted that results reported previously by Blight[9] showed that, with certain materials, the angle of shearing resistance also increases with increasing binder content. The inset on Figure 11 shows how the cohesion increases with increasing binder content.

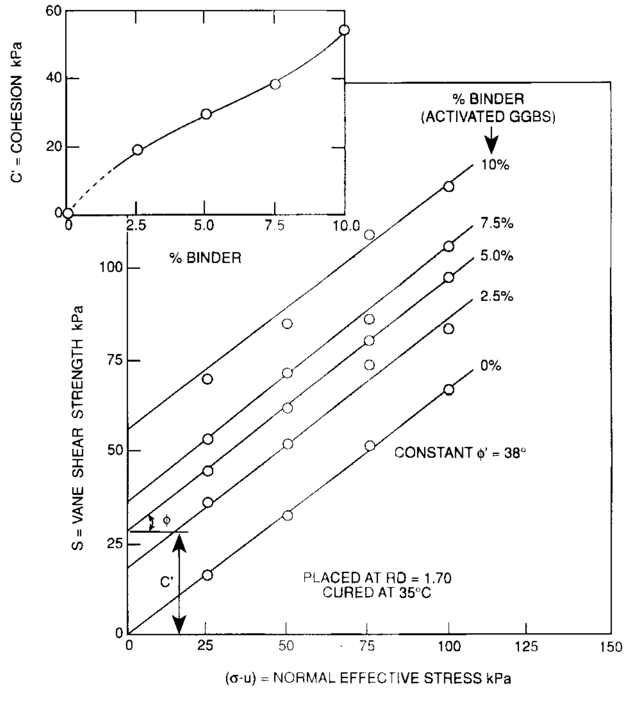

Figure 10—Strength parameters of cyclone-underflow fill stabilized with various percentages of activated GGBS at 24 hours after placement (Western Deep Levels)

Figure 11 compares the results shown in Figure 10 with the results of similar tests on different cycloned tailings and using different types of binder.

Figure 11 shows that the source of the fill can have a significant influence on the shear strength (e.g. the results for backfills from Western Deep Levels and Vaal Reefs cemented with lime-activated GGBS). The type of binder, however, appears to have less effect on the relationship between c' and binder content (as shown by a comparison of GGBS and PBFC with cycloned fills from Vaal Reefs and Western Deep Levels). This difference is probably due to the differences in particle-size distribution, particle sharpness (angularity), mineralogical characteristics, and water quality. However, the results obtained after 24 hours are superior for PC to those for GGBS. Experience shows that the strength of backfill cemented with GGBS will gradually catch up that of backfill cemented with PC. It should, however, be noted that PC is significantly more costly than GGBS.

An examination of the numerical values of c' in Figure 11 shows that the benchmark strength of 12 kPa (required for stability at a free-face height of 3.0 m) can easily be reached within 24 hours with 5 per cent PC.

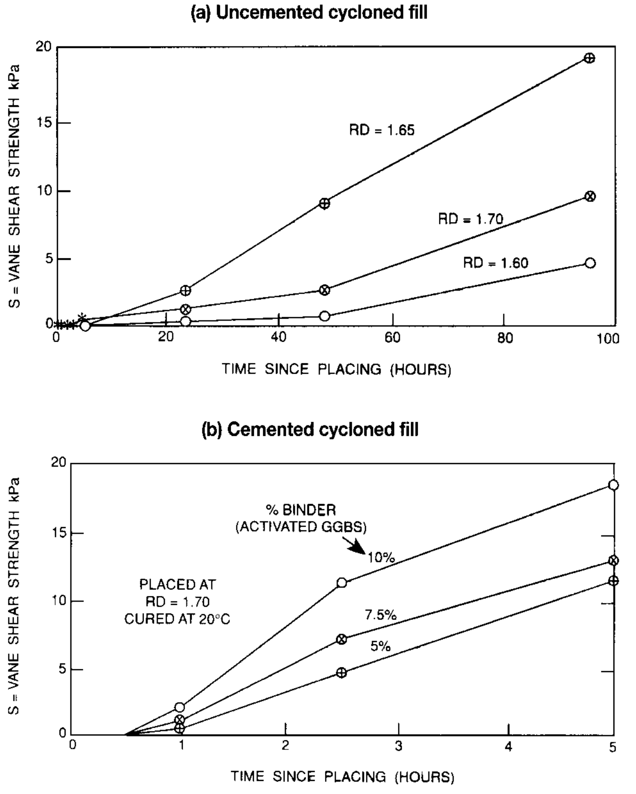

Figure 9—Growth of vane shear strength with time for (a) uncemented cycloned fill and (b) cemented cycloned fill, in a CBR mould and allowed to drain (Western Deep Levels)

Compressibility of cemented backfill

The compressibility of backfill, which controls the closure of a backfilled stope, is usually measured by means of the confined compression test, in which the fill specimen contained laterally in a metal ring is subjected to vertical compression. Figure 12 shows the results of a series of

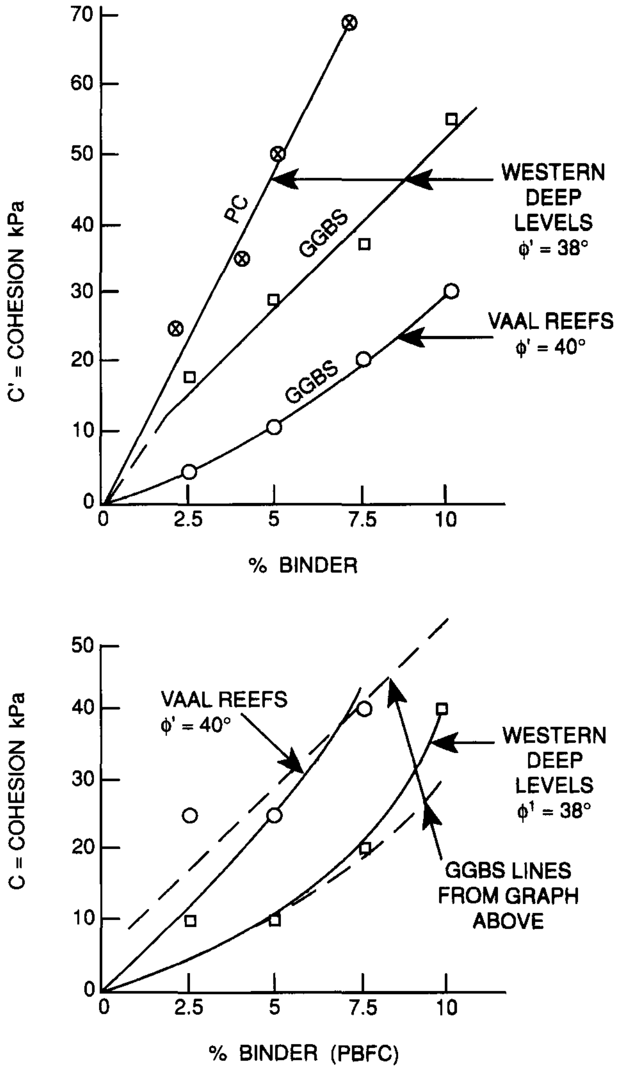

Figure 11—Strength parameters for cyclone-underflow fill from two mines stabilized with PC, PBFC (50:50 PC:GGBS), and activated GGBS at 24 hours after placement (placed at RD = 1.70, cured at 35 degrees)

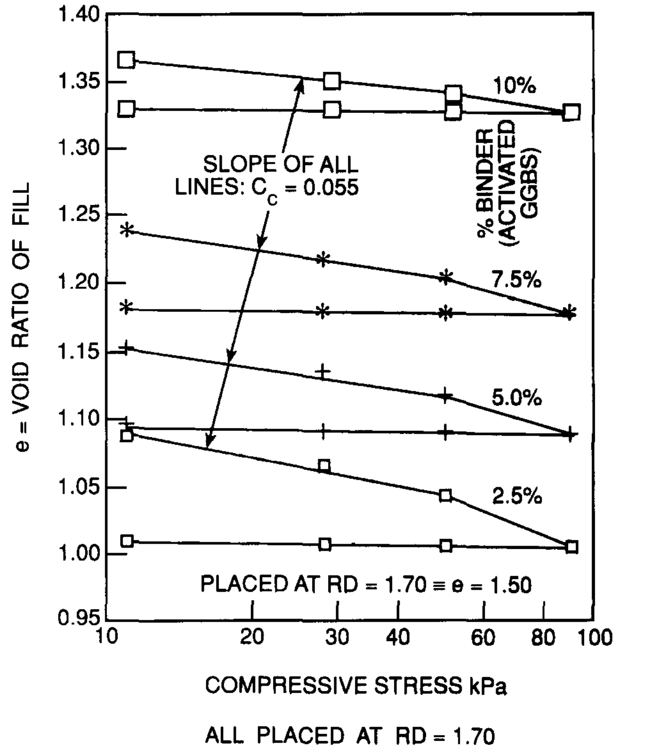

Figure 12—Compression of cemented cyclone-underflow fill from Western Deep Levels at low stresses (with activated GGBS as binder)

confined compression tests at low pressures on a cycloned fill from Western Deep Levels, cemented with various percentages of GGBS activated with lime and PC. The compression curves are given in terms of void ratio (e) versus the logarithm of pressure, where

$$e = \frac{\text{Volume of voids in fill}}{\text{Volume of solids}}. \quad [6]$$

The relative density at placement was 1.70, which corresponds to a void ratio of 1.5. It will be noted that the compression curves have the same slope (C_C) of 0.055 for all binder contents up to a compressive stress of about 50 kPa. Furthermore, all the specimens yielded (i.e. the slope of the compression curve steepened) at the same stress of about 50 kPa. The most notable difference between the curves is the initial void ratio, which increases with increasing binder content. This demonstrates the capability of the sodium silicate in the binder to immobilize water in the pores of the fill, which may be critical if liquefaction is to be avoided. The tests were carried out starting at 24 hours after placement and with 24 hours between each load increment. Reference to Figure 9(b) shows that the cementitious bonds between particles must have been well developed during these tests. Hence, in the low stress range, the compressibility of the fill was controlled by the cementitious interparticle bonds and was not affected by the initial void ratio.

Data for the compression of cemented fills at higher stresses are shown in Figure 13. In this figure, the percentages of compression in a confined compression test at stresses of 10 and 50 MPa were plotted against the relative density at placement of cycloned fill from Western Deep Levels. Two binders and binder contents were used: 3 and 5 per cent PBFC, and the same percentages of GGBS activated with lime and PC.

As shown in Figure 12, the percentage binder had little effect on the compressibility. However, the type of binder had an appreciable influence. The percentage compression at both 10 and 50 MPa was independent of the RD at placement when PBFC was used. This is because the PBFC contained no gelling agent, and the fill samples therefore, regardless of the RD at placement, all settled to the same initial void ratio of about 0.95 before being loaded.

The gelling action of the sodium silicate causes water to be retained in the pores. Hence, the initial void ratios were higher for the activated GGBS (Figure 12). Although this gelled water is retained in the fill under relatively low stresses, it is forced out as the stress is increased from the low stress (kPa) range to the high stress (MPa) range. The result is that the compressibility of fill containing gelled water is more than that of fill containing only free water after initial drainage. The effect of the delayed expulsion of water decreases as the RD at placement increases, as seen in Figure 13.

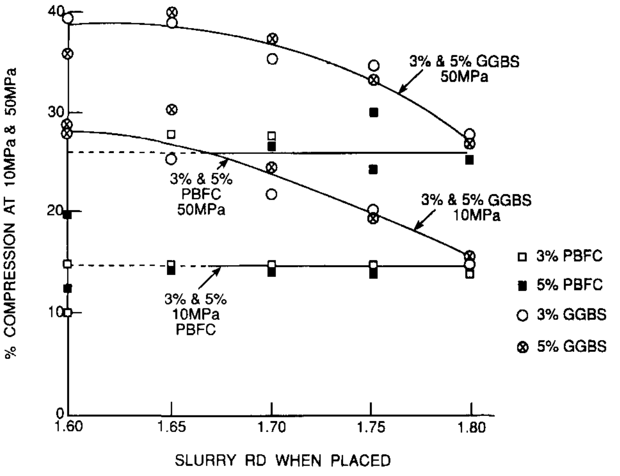

Figure 13—Relationship between compression and slurry RD (when placed) at compressive stresses of 10 and 50 MPa in confined compression tests on cycloned fill from Western Deep Levels

The object of a gelling agent is to reduce the initial water drainage from a fill, and hence shrinkage. Ultimately the gelled water will be forced out under stress, and the advantage of reduced initial drainage of water from the fill must be traded off against the disadvantage of greater fill compressibility at high compressive stresses.

Susceptibility of cycloned backfill to seismic loading or the effects of blasting

From the safety aspect, a backfill placed as a slurry is in its most dangerous state shortly after placement, while the pore-water pressure is high (Figure 5) and the effective stress and shear strength are low (equation [5]). In this condition, while the fill is still essentially a viscous liquid, damage to or collapse of a barricade, or the splitting of a bag retaining the fill, may result in a life-endangering mud flow. Once the fill has gained sufficient strength to resist flowing out of the stope, there is still a possibility that accelerations imposed on the fill by blasting or a seismic event may cause the fill to re-liquefy and flow out if the retaining barricade or bag is damaged.

The liquefaction of fine, sandy materials as a result of seismic accelerations is well-known in relation to damage caused on surface during natural earthquakes. The following three conditions have to be present before liquefaction of a previously solid fill will occur.

- The pores of the fill must be completely saturated with water.
- The void ratio must exceed a certain critical value.
- The disturbing accelerations must exceed a threshold value.

All surface soils that liquefy have individual particles that are either rounded or sub-rounded. Hence a fourth, unstated condition is that the particles should have a suitably rounded shape before liquefaction will occur.

An analysis by Blight[10] shows that, even very severe lateral accelerations (up to 10 g), will induce only moderate shear stresses (about 100 kPa) in the backfill contained in a narrow, tabular stope.

The process of liquefaction is illustrated in Figure 14, which shows the effects when a cyclic shear stress is applied to a specimen of saturated natural sandy silt. The diagram is a plot of the stresses

$$q' = \frac{1}{2}(\sigma_1' - \sigma_3') \text{ (maximum shear stress)} \quad [7a]$$

versus

$$p' = \frac{1}{2}(\sigma_1' - \sigma_3') \text{ (mean principal effective stress),} \quad [7b]$$

where σ'_1 and σ'_3 are respectively the major and minor principal effective stresses in the soil, q' is the maximum shear stress applied to the soil, and p' is the mean principal effective stress. In this case, q' was cycled from zero to 300 kPa to zero, as indicated by the numbers 1,2,3, etc., for the cycles. When a cycle of load is applied to the soil, the pore-water pressure, u, rises, and hence, because the effective stress is given by $(\sigma - u)$, the effective stresses are reduced. If the increase in u exceeds the increase in $1/2\ (\sigma_1 + \sigma_3)$, the stress path moves towards the origin. After 13 cycles, the stress condition in the soil had progressed to the failure envelope (or K_f line), and the silt failed in shear. On a subsequent cycle of load, the specimen was able to resist only a lesser shear stress of 240 kPa, and the test was terminated. Many investigators (e.g. Seed and Lee[11]) have shown that, if the cyclic loading had been continued, the shear resistance would have continued to decrease with each cycle (as indicated by the broken lines) and the specimen would simultaneously have undergone very large deformations.

Figure 15 shows a plot similar to Figure 14 for tests on a series of specimens of uncemented cycloned fill. Each experimental point on this diagram represents the end-point stresses after the application of 25 cycles of shear stress. For example, point A represents the vertical effective stress in a sample consolidated to a mean effective stress of 175 kPa, point B represents the stress state after 25 applications of a shear stress of 40 kPa, etc.

It will be seen that all the stress paths for whatever form of cyclic loading go onto the K_f-line and then proceed up it (to the right). In other words, the mean effective stress in the specimen, as well as the shearing resistance, increases progressively in every case. This applies regardless of the value of the initial effective stress (or, at least, for initial effective stresses as low as 5 kPa). Similar tests have been carried out for other cyclone-underflow tailings, for full plant tailings, and for cyclone overflow, and the result has always been similar. Hence, it can be concluded that Witwatersrand quartzite gold tailings (probably because of the angular and irregular shape and the harsh surface texture of the particles) will not liquefy under the application of cyclic shear stresses.

Although it is unlikely that cemented backfill will liquefy if uncemented backfill will not, tests were nevertheless undertaken to check this. The results of one series of these tests are shown in Figure 16. It will be seen that the results of the tests on cemented fill were very similar to those on uncemented fill, although the changes in pore pressure were less. There was no tendency to liquefy under cyclic stresses.

These laboratory results have frequently been confirmed *in situ* when placed fill has been affected by nearby large-energy seismic events that caused significant stope closure but no tendency for the fill to liquefy.

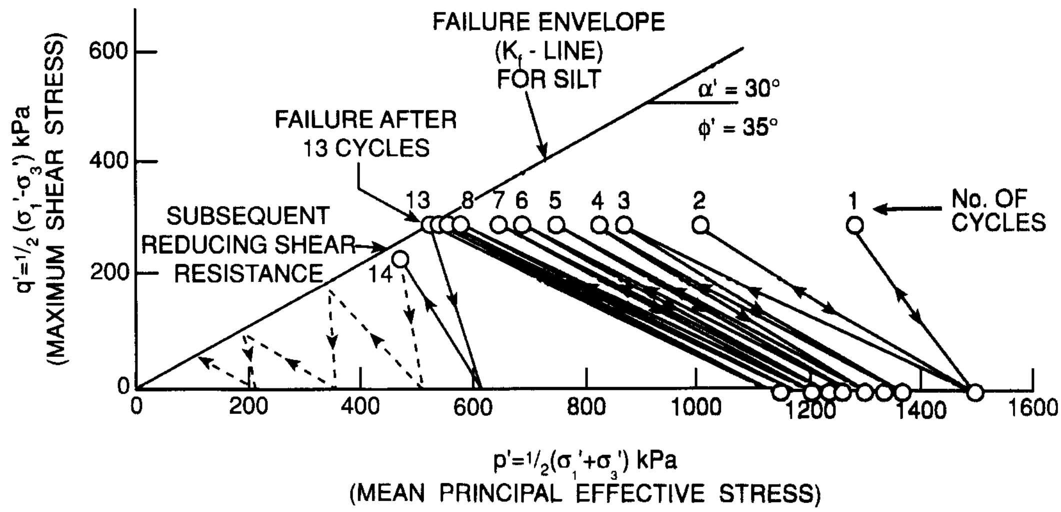

Figure 14—Stress path in a dynamic shear test on loose, saturated, natural sandy silt

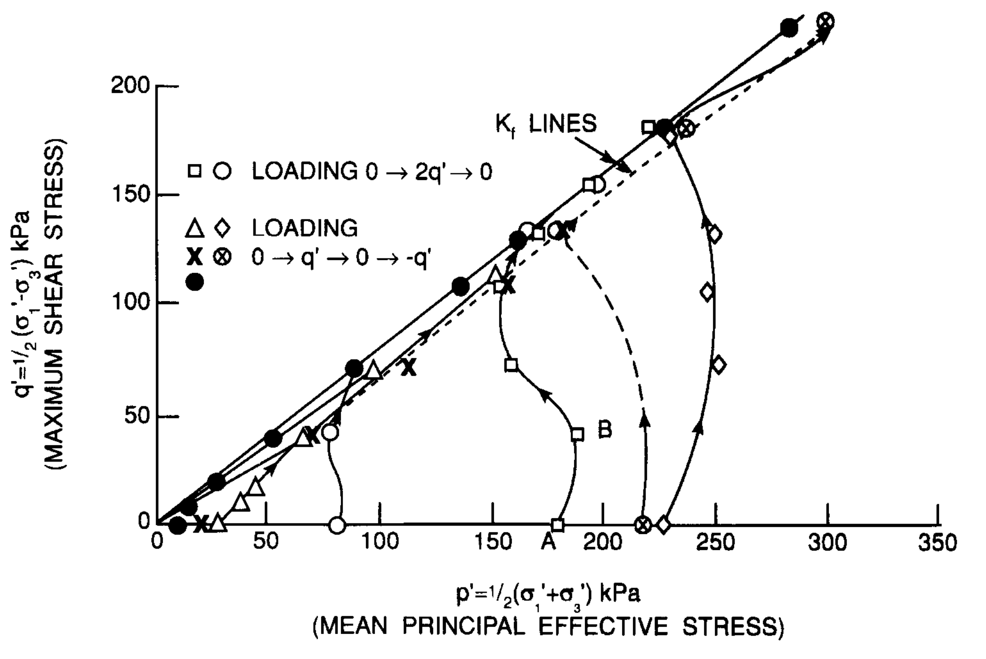

Figure 15—Stress paths for the undrained shearing of uncemented cycloned fill

Properties of cemented fill subjected to closure during curing

Fills will usually be subjected to closure from an early age after their placement. A cemented fill may therefore be subjected to appreciable compressive strains while the binder is curing and the fill is gaining strength. The question that arises is whether the closure strains damage the developing cementitious bonds and, therefore, whether laboratory test results, such as those shown in Figures 10 to 13, can possibly be representative of the properties of fill *in situ*.

To investigate the effects of closure strains on shear strength and compressibility, a series of tests was undertaken. The tests were of two types: drained triaxial compression tests, and compression tests on discs of cemented fill with initial dimensions of 300 mm in diameter and 30 mm in thickness. The binder was 6 per cent GGBS activated by lime and PC for all the tests in this series, and the results reported here are for cycloned fill from Elandsrand gold mine.

In each case, a sub-series of control tests was carried out in which the specimens were tested at the ages of 1, 2, 5, 7, and 14 days. These specimens were regarded as having been subject to zero closure during curing. In the second and third sub-series of tests, the specimens were subjected to compressive strain rates of 2.5 per cent and 5 per cent per day, starting 4 hours after casting, immediately after the initial set of the binder had occurred. When these specimens reached their designated test ages, they were tested to failure in the case of the shear tests. In the case of the compression tests for ages up to 2 days, the specified closure strain rate was applied and, on the designated test day, the strain was increased to

10 per cent. For the tests at 5 days, the closure straining was stopped when it reached 9 per cent and was then taken to 10 per cent on the designated day.

Figure 17 shows the results of the shear tests, which were plotted as the developed angle α (in the plot of q' versus p') versus the age at test. The results show very clearly that closure during curing had a beneficial effect on the shear strength of cemented fill.

Figure 18 shows the results of the compression tests plotted as the stress at 10 per cent closure versus age at test. Here, again, closure during curing had a beneficial effect in decreasing compressibility.

The way in which the beneficial effect arises is thought to be as follows.

➤ As the specimens are strained, their density increases, causing the binder-to-water ratio in the pores to increase and the binder to become more effective.

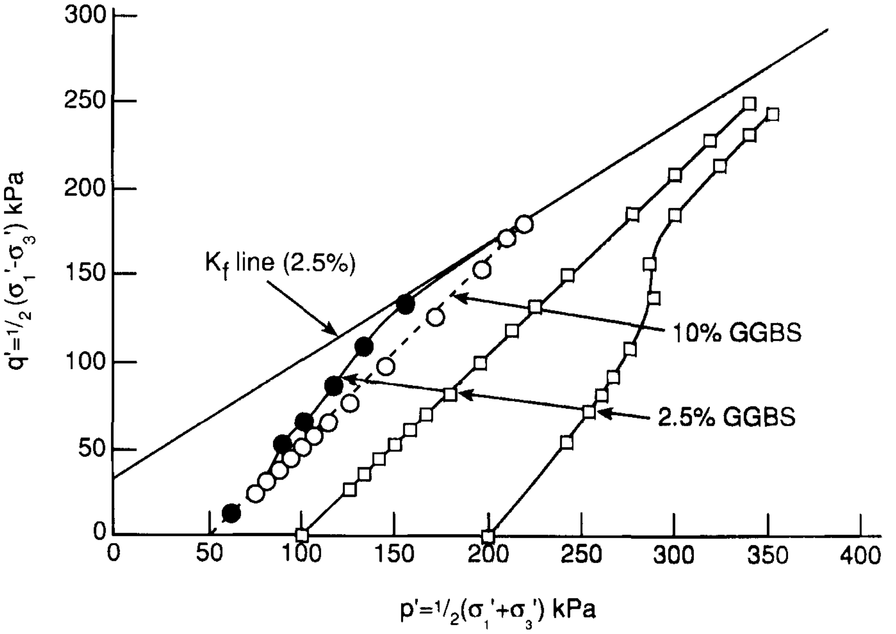

Figure 16—Stress paths for cycloned fill cemented with 2.5 and 10 per cent activated GGBS (Vaal Reefs, 24 hours after placement)

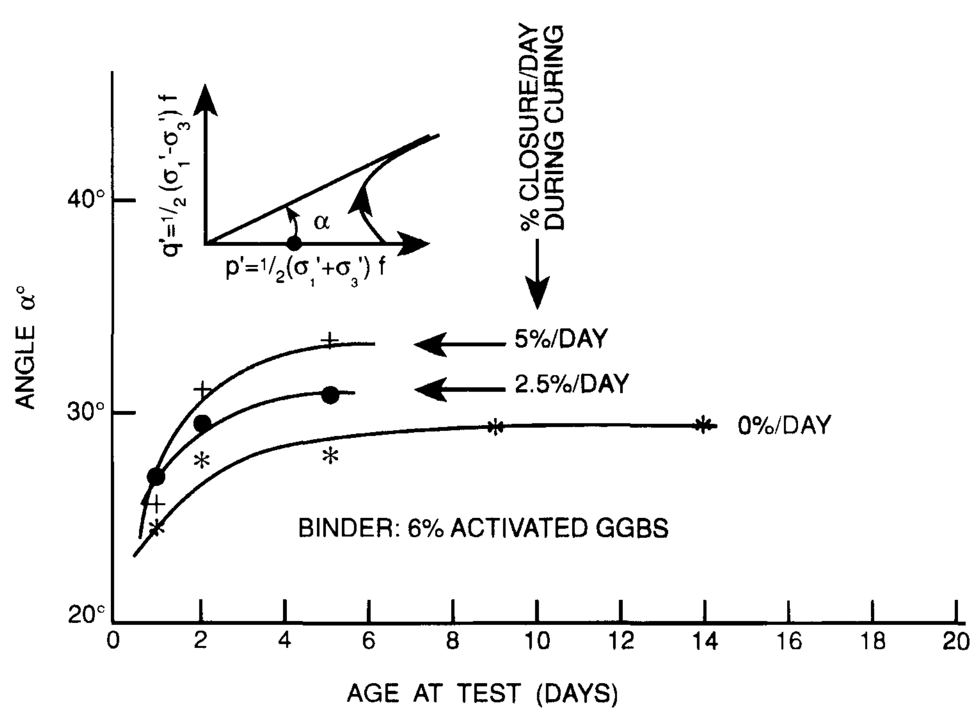

Figure 17—Results of tests on the effect of closure during curing on the strength of a cemented cycloned fill from Elandsrand

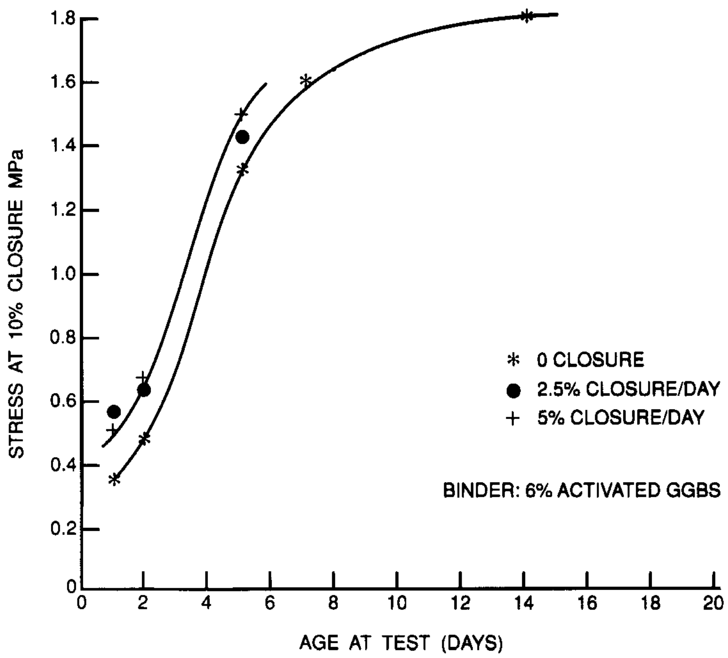

Figure 18—Results of tests on the effect of closure during curing on the compressibility of a cemented cycloned fill from Elandsrand

- Because the binder is in the process of hydrating and forming chemical bonds as the closure occurs, any damage to these developing bonds can heal as it occurs.

When a fill is subjected to a closure of up to 5 per cent per day during curing, the overall end result is a fill that is denser and more effectively and strongly bonded by the cement.

Conclusions

The following general conclusions can be drawn concerning the properties of cemented and silicated backfill.

(i) The stress–stretch characteristics of a geofabric used to retain backfill during placement should be considered in the assessment of the loads imposed on geofabric-covered barricades. Supports for the barricade should, in general, be placed not more than 1.0 m apart.

(ii) The relationship between the height of a stable vertical free face of backfill and the shear strength of the fill can be presented in chart form for various levels of acceleration, and should be used in the establishment of the early-strength requirements of the fill for a site-specific application.

(iii) Cemented backfill can gain early strength at a far greater rate than uncemented backfill, even though the rates of drainage and air entry may be similar.

(iv) The principal effect of cement addition on the strength of cycloned backfill is to increase the cohesion component of shear strength. Cemented fills have a similar void ratio and compressibility for a range of cement contents. If a gelling agent is used to reduce water run-off by immobilizing pore water in the fill, then the initial void ratio of the fill increases with increasing binder content. Backfills cemented with cement and gelling agents are ultimately more compressible than those bound by cement only.

(v) Backfills of Witwatersrand quartzite tailings, whether cemented or not, will not liquefy under the action of seismic or blasting acceleration. The closure (up to 5 per cent per day) taking place during the curing of the cement appears to have beneficial, rather than deleterious, effects on both strength and compressibility.

(vi) Cemented or silicated fill can be used most beneficially when the stoping width exceeds 1.5 m.

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