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Failure of a Transport Tunnel Below a Dolomite Stockpile

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Synopsis

The failure of a 3 m x 4 m transport tunnel constructed below a dolomite stockpile has been investigated. It was found that the cracking of the reinforced concrete lining had most likely been caused by lateral distortion (sidesway) of the tunnel. The transverse shears across the tunnel roof and in the residual soil below the stockpile caused by the sloping stockpile were very high. The average shear stress even exceeded the shear strength of the weathered material around the tunnel. It is thus important to consider the transverse shear forces across the roof slab and the horizontal shear stresses in the surrounding soil in the design of a tunnel lining. In addition to strengthening the tunnel section by means of steel frames, an embankment was constructed at the toe of the stockpile in order to increase the stability. The embankment also reduced the average slope of the stockpile.

Introduction

About six months after the construction of a 3 m x 4 m transport tunnel below a dolomite stockpile, it was discovered that the reinforced concrete lining had cracked and spalled along the edge of the roof slab for about two-thirds of the length of the tunnel. There was also a wide longitudinal crack in the tunnel floor.

The tunnel had been constructed in a trench at the bottom of a deep cut along the side of a hill as shown in Fig 1. Part of the hill had previously been excavated and benches had been cut in the slope to reduce erosion. The crushed dolomite was transported to the stockpile by a conveyor, 44 m above the tunnel roof. The discharge point was offset 36.6 m from the centreline of the tunnel.

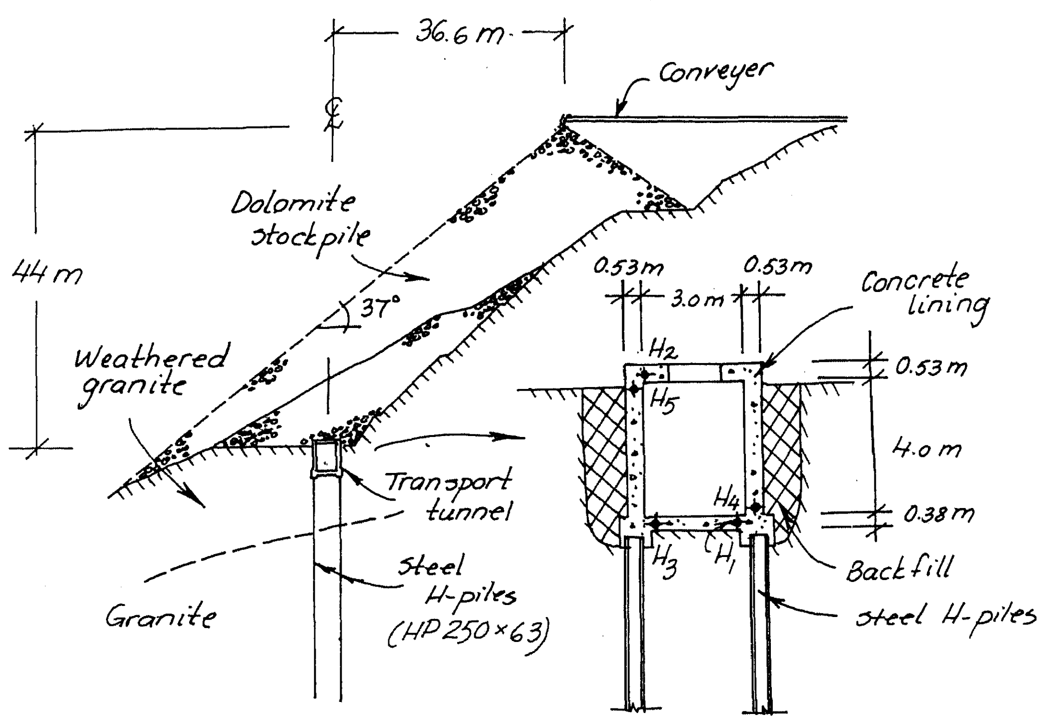

Fig 1 Location of tunnel

The tunnel was supported on two rows of steel H-piles (HP 250 x 63). The spacing of the H-piles varied from 0.92 m to 2.44 m depending on the height of the stockpile. The piles were driven to refusal.

The thickness of the roof slab and of the walls was 0.53 m while the bottom slab was 0.38 m thick. The roof tunnel had three openings at regular intervals so that the crushed dolomite could be drawn from different parts of the stockpile. Three steel hoppers located below the roof openings fed the stone from the stockpile to a conveyor in the tunnel which led to a crushing plant. The 60 m long tunnel extended only partly through the stockpile, as shown in Fig 2.

Soil Conditions

The soil conditions around and below the tunnel were explored after the failure with five boreholes (Fig 2). At Borehole 2 (BH 2) the soil consisted, as can be seen from Fig 3, of 10 m of firm to very stiff sandy clay (residual soil), 4.5 m of hard sandy silt, 3.4 m of very dense clayey, silty sand (weathered granite), 0.7 m of decomposed granite and of granite at 18.5 m depth. The penetration resistance of the firm to stiff sandy clay as determined from standard penetration tests (SPT) varied between 8 and 18 blows/0.3 m down to 10 m depth. In the underlying silt and sand, the penetration resistance (N) exceeded 100 blows/0.3 m.

Pressuremeter tests in the sandy clay (BH 4) at 1 m and 2 m depths below the tunnel indicated an average shear strength of 73 kPa. (The shear strength was estimated from the empirical relationship $s_u = p_\ell/9$ where p_ℓ is the limit pressure.) This shear strength is consistent with the penetration resistance determined from SPT. The average shear modulus (G_M) as evaluated from the pressuremeter tests was 39.4 MPa.

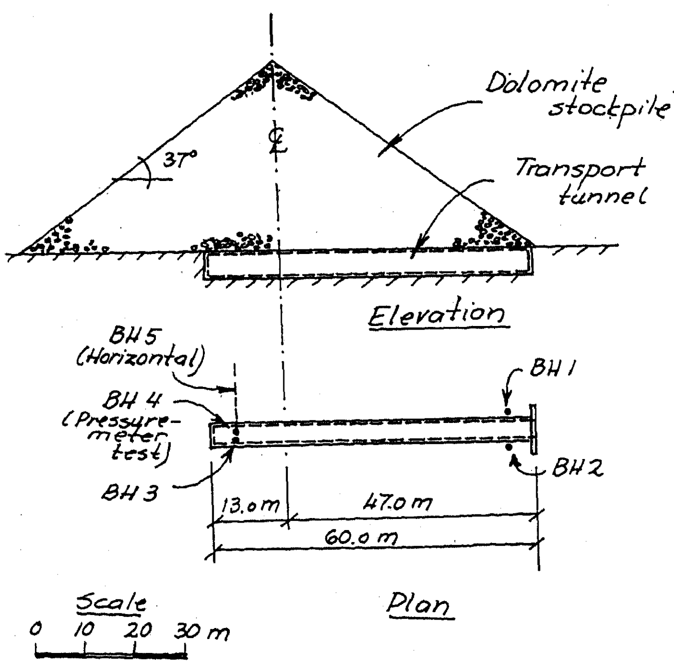

Fig 2 Location of tunnel and of boreholes

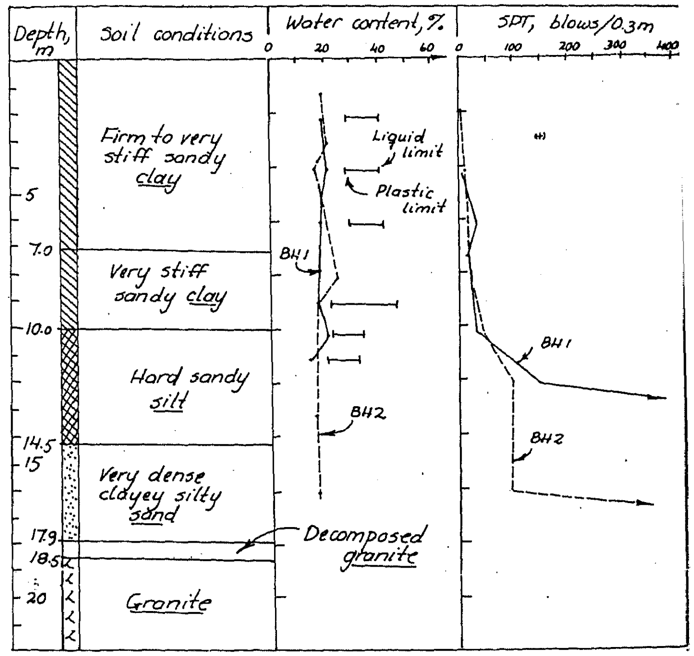

Fig 3 Soil conditions (BH1 & BH2)

The shear strength as determined by triaxial tests (UU-tests) increased with depth. At a confining pressure of 280 kPa, the approximate lateral pressure at the level of the tunnel, the average shear strength (77.4 kPa) was close to that (73 kPa) obtained from the pressuremeter tests. It should be noted, however, that it had rained almost continuously for more than two weeks when the cracking of the tunnel was discovered (Fig 4). The total rainfall for the month was 497 mm. The resulting increase of the ground water level, especially in the silty clay below the stockpile, could have reduced the shear strength of the soil.

Description of the Failure

The height of the stockpile had gradually been increased since the completion of the tunnel for almost seven months, when a large longitudinal crack was observed in the floor slab. The crack was located at H1 near the inner wall of the tunnel (Fig 1), close to the centre of the stockpile. The total height of the stockpile was 37.3 m above the tunnel roof when the crack was discovered. Subsequently, the height was reduced by 4 m. The height of the stockpile above the centre of the tunnel was 11.1 m at the time of the failure.

The concrete floor slab had been displaced upwards by up to 75 mm at H1 for about two-thirds of the length of the tunnel along the inner face of the pile cap beam below the inner tunnel wall. The top half of the concrete slab had separated horizontally at mid-depth. The separation extended at least 500 mm inwards from the wall. The top steel reinforcement was kinked across the separation.

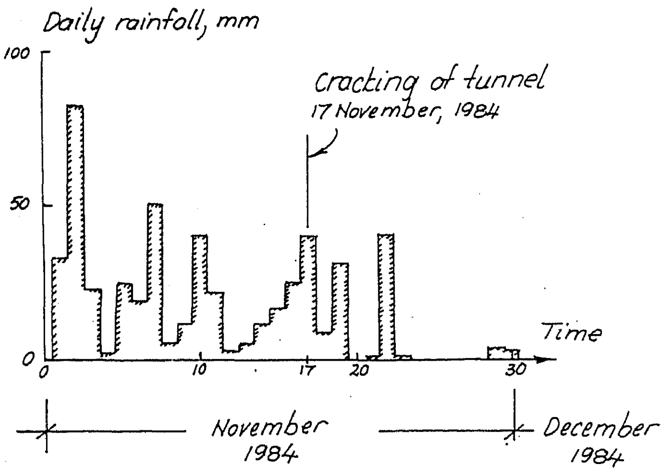

Fig 4 Daily rainfall in November 1984

Numerous cracks were also noticed in the tunnel roof along the opposite top corner of the tunnel at location H2 (Fig 1) where the concrete had spalled. The cracks in the roof were extensive in both directions. The cracks along the main reinforcement extended almost the full width of the roof slab. Between the second and the third hoppers at the far end of the tunnel the spacing of the transverse cracks was approximately 500 mm. The cracking was insignificant for about the first third of the tunnel from the exit end. The conveyer system in the tunnel was still operational when the cracking was observed.

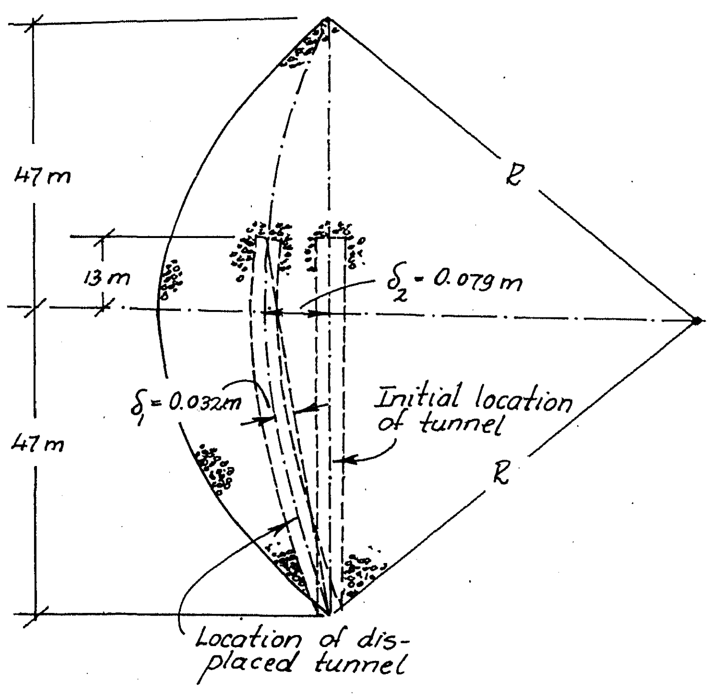

Fig 5 Lateral displacement of tunnel

Movements of the Tunnel

A survey carried out in February 1985 indicated that the tunnel had been bodily displaced horizontally by up to 79 mm (Fig 5). The horizontal deviation of the bottom slab from a straight line joining the two ends of the tunnel was 32 mm i.e. about 1 in 2,000. This was consistent with the height of the stockpile above the tunnel which reached a maximum at the middle hopper. The horizontal curvature along the tunnel axis was structurally negligible.

The horizontal angular distortion (sidesway) of the tunnel walls was about 16 mm in 60 m or 1/4.000 from the theoretical alignment. The sidesway, which was about 60 mm in 4 m or 1 in 67, was high. The resulting deformations due to sidesway must have been in the inelastic range.

The settlements of the tunnel increased by about 10 mm from the exit towards the loaded end. The relative settlements across the tunnel were negligible. It should be noted that settlement of the soil with respect to the tunnel wall alters the direction of the vertical shear force along the wall and therefore the direction and the magnitude of the lateral earth pressure. This changes the pile loads substantially while the stress distribution in the tunnel lining is only affected marginally. The settlements along and across the tunnel axis were small. Their effect on the tunnel stresses was therefore insignificant.

Design of the Tunnel Lining

It was assumed in the design of the tunnel and subsequently checked in the field that the unit weight of the crushed dolomite was 16.5 kN/m^3. The angle of internal friction (ϕ') was 37^o which corresponded to the angle of repose and the vertical load (325 kPa) on the tunnel corresponded to the average height of the dolomite stockpile (17.2 m) above the tunnel roof. The actual vertical load on the tunnel roof at the time of the failure was only 65% of the design load. The tunnel had been designed for a dragdown force f_a ($K\sigma'_v$ tan ϕ') which corresponded to a coefficient of lateral earth pressure, $K = \cos^2 \phi'$. The lateral earth pressures on both sides of the tunnel were assumed to be the same. The difference in the lateral earth pressures acting on the two tunnel walls as well as the horizontal shear force along the roof slab had thus been neglected.

The negative skin friction on the steel H-piles below the inside wall of the tunnel where the height of the stock pile was the greatest apparently did not affect the tunnel since the differential settlements across the tunnel were insignificant. The completely decomposed material behaved as a heavily over-consolidated clay.

Stress Distribution in the Dolomite Stockpile

The stress distribution in the crushed dolomite can be evaluated directly, since the average slope of the stockpile corresponds to the angle of repose (ϕ_r) which is equal to the angle of internal friction (ϕ' = 37^o) of the loose crushed dolomite.

The stress conditions in a slope with an inclination equal to the angle of repose can be calculated as shown in Fig 6. The normal pressure σ_{n2} acting on a plane

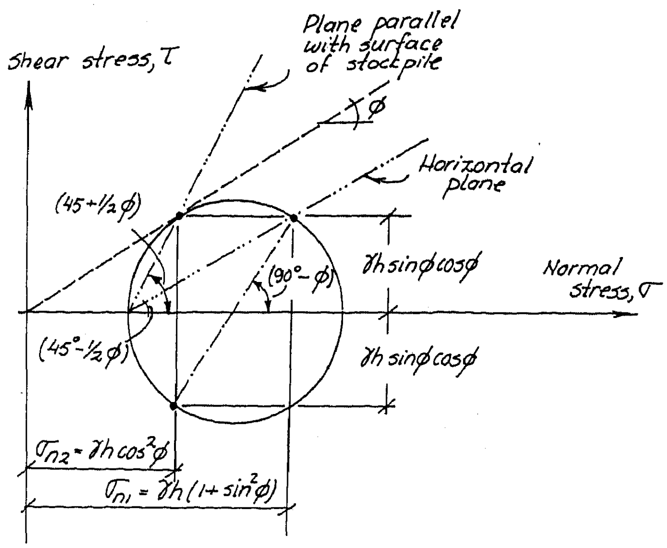

Fig 6 Mohr's stress circle

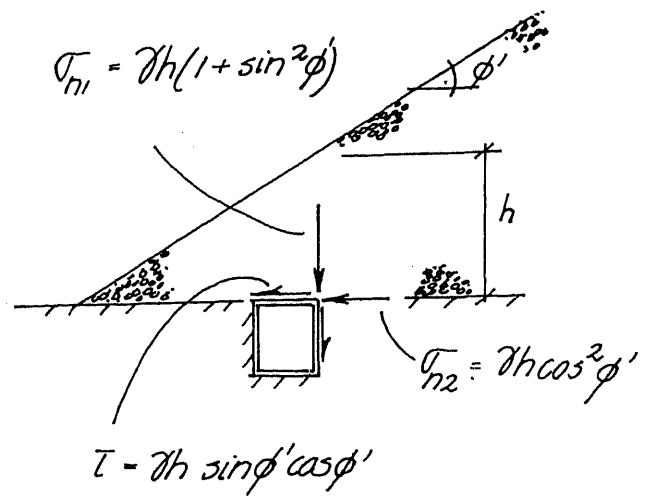

Fig 7 Stress distribution around a tunnel below a stockpile

parallel with the surface of the stockpile is thus $\rho h(1 + \sin^2\phi)$ where ρ is the unit weight of the crushed dolomite and h is the height above the tunnel. The same normal pressure occurs also on a vertical plane through the stockpile. The corresponding normal pressure (σ_{n1}) on a horizontal plane, the roof of the tunnel, is equal to

$$\sigma_{n1} = \rho h(1 + \sin^2\phi') \qquad (1)$$

This normal pressure is higher than that which corresponds to the height of the stockpile above the tunnel. At $\phi' = 37^o$, σ_{n1} will be equal to 1.36 ρh while at $\phi' = 45^o$, $\sigma_{n1} = 1.50\rho h$. The resulting stress distribution around the tunnel with $\phi' = 37^o$, $\rho = 19 kN/m^3$ and h = 11.1 m, the height of the stockpile just above the tunnel at the time of the failure, is shown in Fig 7. The normal pressure on the slab at the center of the tunnel is 287 kPa.

High horizontal shear stresses also act across the roof slab and in the surrounding soil. The shear stress (τ) along a horizontal plane through the stockpile can be evaluated from

$$\tau = \rho h \sin\phi' \cos\phi' \qquad (2)$$

At h = 11.1 m and $\rho = 19\ kN/m^3$ the average shear stress across the tunnel roof is 101 kPa, which for the 4.06 m wide tunnel corresponds to a total horizontal force (T) of 410 kN/m.

High horizontal shear stresses exist also in the weathered material below the stockpile, which increase linearly from the toe towards the centre of the stockpile. The maximum shear stress below the centre of the 37.3 m high stockpile has been estimated to be 341 kPa. The average shear stress along the base of the stockpile, 170 kPa, thus exceeds the estimated undrained shear strength of the soil (about 75 kPa).

The difference in lateral earth pressure between the two outside walls has contributed to the sidesway of the tunnel. This pressure difference can be calculated from

$$(\sigma_h' - \sigma_h'') = B\rho \cos\phi' \sin\phi' \qquad (3)$$

At B = 4.06 m, $(\sigma_h' - \sigma_h'')$ = 37 kPa which for the 4.91 m high tunnel corresponds to an estimated net lateral force (F) of 182 kN/m across the tunnel roof slab at a friction angle of 37^o for the dolomite. At $\phi' = 45^o$, this net force increases to 189 kN/m. The effect of the angle of internal friction on the stress difference is thus small.

<u>Lateral Displacements of the Tunnel</u>

The high shear stresses along the surface of the residual soil (the stiff clay) below the dolomite stockpile cause a shear distortion (γ) of the soil (G) as illustrated in Fig 8, which depends on the shear modulus. The shear distortion can be evaluated from the general relationship $\gamma = \tau/G$, neglecting the restraint by the piles. At τ = 101 kPa and G = 3940 kPa, the distortion is 0.026 radians. The lateral displacement of the tunnel at the bottom slab depends on the thickness (H) of the residual material below the tunnel. At H = 5.1 m the lateral displacement of the floor slab is estimated to be 133 mm (0.026 x 5100). The corresponding lateral displacement of the soil at the level of the roof slab just below the stockpile is 260 mm. The real lateral displacement was likely to have been less because the actual shear modulus was probably higher than that assumed in the calculations (3940 kPa).

The measured tilt (sidesway) of the tunnel has been plotted in Fig 9 as a function of the height of the stockpile above the centreline of the tunnel. It can be seen that the shear distortion of the tunnel increases rapidly as the height increases. The distortion is reduced when the height is reduced.

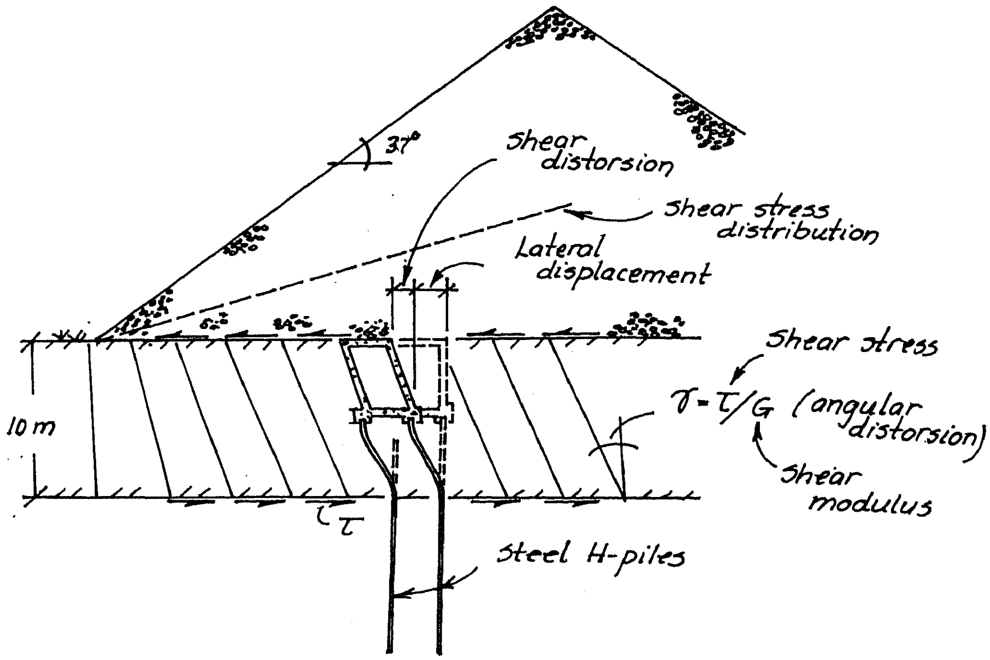

Fig 8 Displacement of tunnel

t is interesting to compare the calculated maximum ateral displacement of the tunnel floor (133 mm) with hat which has been observed. The maximum lateral isplacement of the bottom slab of the 60 m long tunnel as approximately 32 mm from a straight line joining he two ends. This corresponds to a total lateral isplacement of the tunnel at the centerline of the tockpile of 79 mm, which is 60% of the estimated isplacement (133 mm). The lateral displacements in he hard silt and the very dense sand has been eglected because of the high SPT values (> 100 lows/0.3 m). It should be noted that the measured hear modulus from the pressuremeter tests is often too ow due to the disturbance of the soil during the rilling of the boreholes. Therefore the calculated ateral displacement of the tunnel, neglecting the estraint by the piles, is too high.

tress Distribution in the Tunnel Lining

n elastic analysis indicated that the tunnel lining ad been overloaded along the outer edge H2 both in hear (Fig 11) and in bending (Fig 12) according to ritish Standard CP110. At the inner edge H3 the tress conditions were not critical. The observed rushing and spalling of the concrete at H2 are onsistent with the excessively high bending moment at his section. Even after moment redistribution and the levelopment of plastic hinges, the high bending moment t H2 must have reduced the shear capacity also at this ritical section. The calculated shear force was close o the shear resistance of the critical section at the ime of the failure.

lithout transfer of the loads from the piles to the loor slab, sections H1 and H3 at the inner and outer dges of the floor slab respectively would both be arginally overstressed in shear. Also, the bending oments were high at these two sections. However, the oil reaction beneath the floor slab increased the hear force at H1 but reduced it at H3. The shear capacity at H1 had undoubtedly been lowered by the high bending moment. Any weakness in the concrete arising from e.g. improper curing would cause a load redistribution from the unconfined concrete near the top of the floor slab at H1 to the reinforcement which

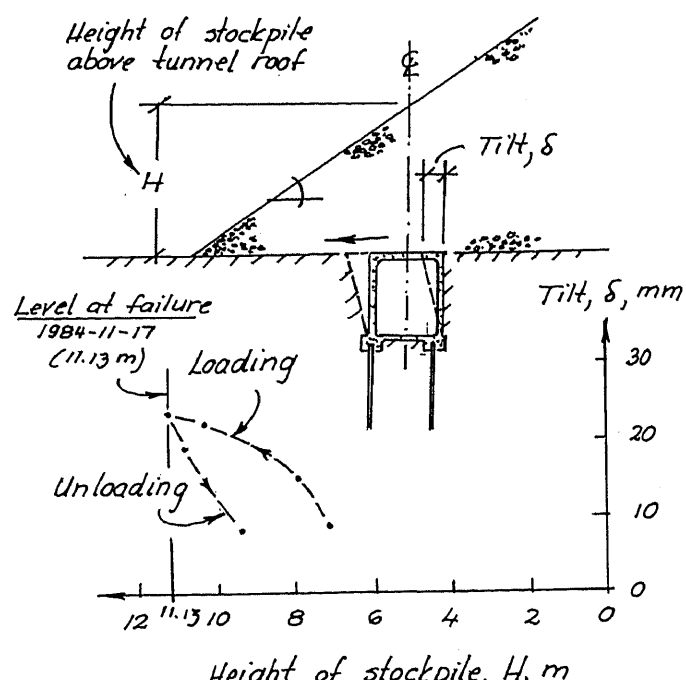

Fig 9 Tilting of tunnel

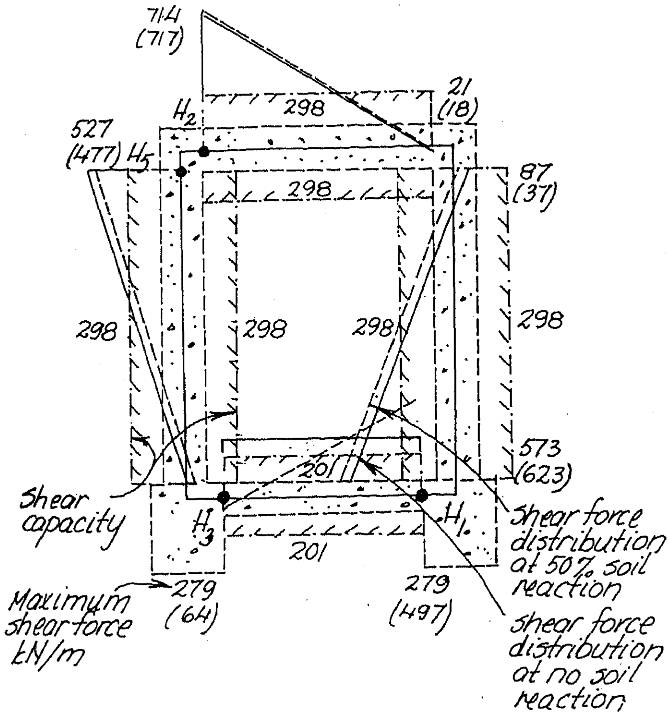

Fig 10 Shear force distribution in the tunnel section

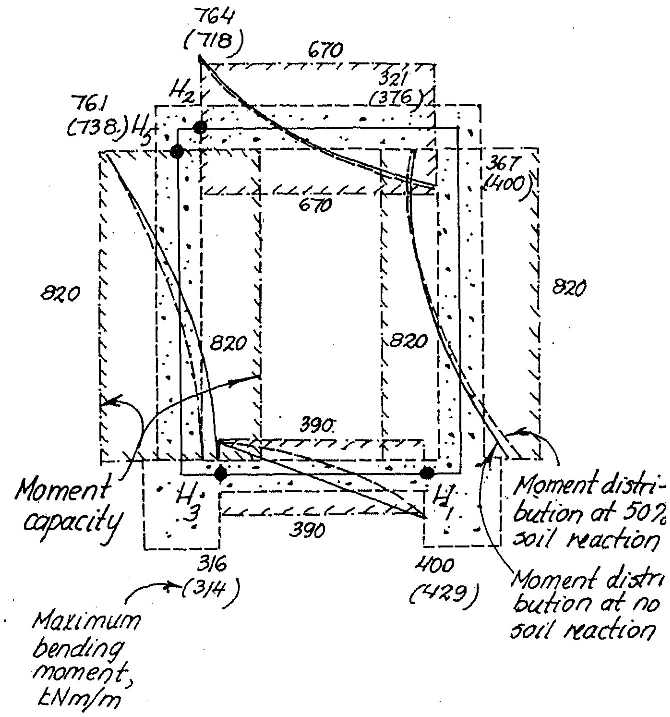

Fig 11 Moment distribution in the tunnel section

was not tied laterally. Therefore the bars buckled. The buckling was accompanied by a dislocation of the concrete in the compression zone, so that the shear force had to be resisted almost solely by dowel action of the compression reinforcement.

The average estimated in-situ cube strength of five concrete cores taken from the tunnel floor was 26 MPa. The maximum deviation from the mean was -7.7 MPa and 8.8 MPa. The variability of the quality of the concrete as indicated by the cones was thus unusually large.

At section H1 the wall was less overloaded in bending but the shear force was higher compared with H2. Of the two critical sections, H2 is theoretically more critical than Section H1. However, any reduction of the concrete strength in the floor slab at H1 would reduce this difference. Although the tunnel lining was overstressed, the sections did not fail completely mainly because of the shear resistance provided by aggregate interlock and by the dowel action of the steel reinforcement. Both the shear force and the bending moment were relatively low in the wall at H5 close to the roof at the time of the failure. The high compressive force in the wall would reduce the bending resistance while the shear resistance would be increased.

It is unlikely that downdrag caused by negative skin friction contributed to the failure of the tunnel. Field measurements did not indicate any significant differential settlements either along or across the tunnel as mentioned above. An elastic analysis of the tunnel structure showed that the piles and the soil supporting the tunnel were not overloaded at the time of the failure. Any excessive load on the piles would have been transferred through the floor slab to the underlying sandy clay because of its high stiffness.

The load redistribution due to arching in the stockpile material must have been small because the stress distribution corresponded to the Rankine state of stress. The slope of the stockpile corresponded to the angle of internal friction of the loose crushed dolomite.

The tunnel is partly supported by the piles and partly by the weathered material below the tunnel, firm to very stiff sandy clay. The load distribution depends on the relative vertical stiffnesses of the soil and of the piles. It should be noted that the upward soil reaction on the tunnel floor slab aggravates the stress conditions at H1 while at H3 the stress level is reduced.

Main Causes of the Failure

The investigation showed that the failure of the tunnel had mainly been caused by

(a) distortion of the tunnel section (sidesway) due to the unbalanced earth pressures on the tunnel walls and a transverse shear force across the roof slab

(b) low strength of the concrete in the floor slab at H1

(c) the absence of tranverse shear reinforcement and of ties to prevent buckling of the steel reinforcement in the compression zone, and

(d) transfer of load through the bottom slab.

The statically indeterminate tunnel section can support higher loads than predicted by an elastic analysis due to stress redistribution in the lining close to failure.

Remedial Works

Stiffening steel frames were installed within the tunnel as shown in Fig 12 to prevent further shear distortions of the tunnel section since the failure was mainly due to sidesway. The frames had to be stiff and strong enough to carry a large proportion of the loads on the tunnel and to limit the sidesway. They also had to be ductile. The location of the steel frames are shown in Fig 13.

The steel frames, each prefabricated in three sections, were connected in the field with high strength friction grip bolts. The tunnel roof was repaired with reinforced gunite while the wide crack along the floor slab was grouted. The gap between the steel frames and the floor slab was filled with structural concrete.

The stability of the tunnel was then increased by the construction of a stabilizing berm (7 x 3 m) at the toe of the stockpile as illustrated in Fig 14, which reduced the average slope as well as the average shear stress across the tunnel roof and in the completely weathered material below the base of the stockpile.

Summary

1. The main cause of the failure of the tunnel was sidesway due to the transverse shear force across the tunnel roof and the difference in lateral earth pressures on the tunnel walls. These forces were not considered in the design of the tunnel. The average horizontal shear stress in the weathered material just below the stockpile exceeded the shear strength of the soil.

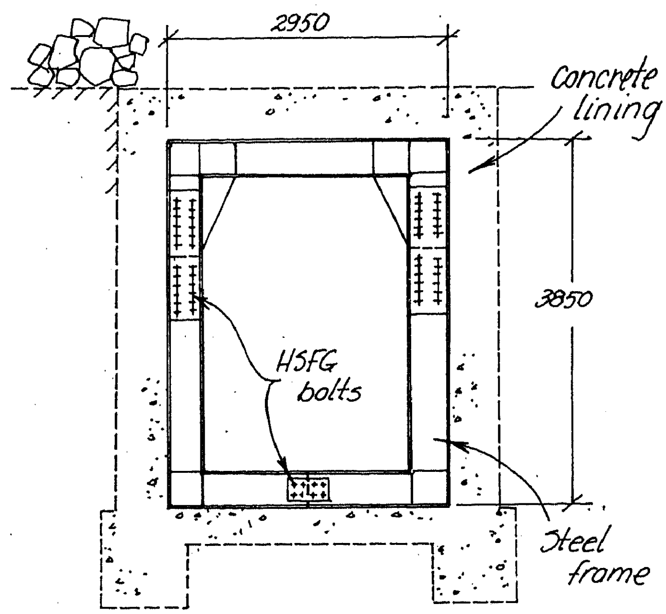

Fig 12 Stiffening of tunnel section with steel frames

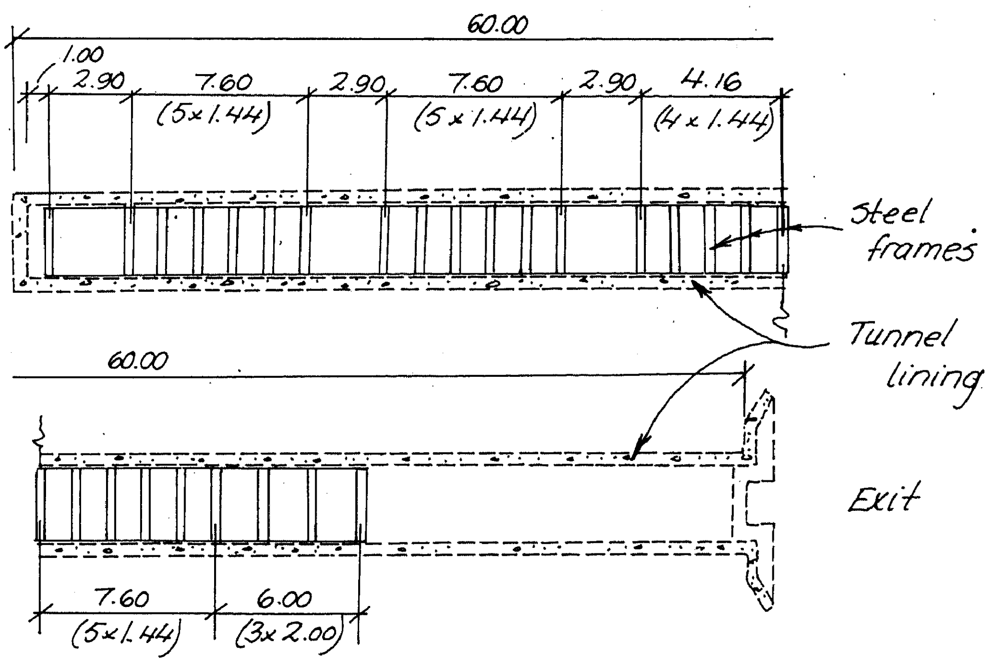

Fig 13 Location of steel frames

2. The lateral displacement of the tunnel estimated from the pressuremeter modulus and from the measured shear strength of the soil agreed reasonably well with the measured values.

3. The lateral earth pressure and the shear force acting across the roof of the tunnel were found to increase linearly with the height of the stockpile above the tunnel. In order to limit the lateral displacement of the tunnel, the height of the stockpile above the tunnel or the average slope of the stockpile had to be reduced. Stability has been increased by constructing a stabilizing berm along the toe of the stockpile.

4. In the design of a transport tunnel below a stockpile it is important to consider the increase of the vertical load on the roof of the tunnel caused by the inclined face of the stockpile. This load increase can be up to 50% higher than the weight of the stockpile material calculated from its height above the tunnel. The horizontal shear stress across the roof slab and in the soil around the tunnel can also be high. Also, the lateral pressure acting on the two tunnel walls will not be equal.

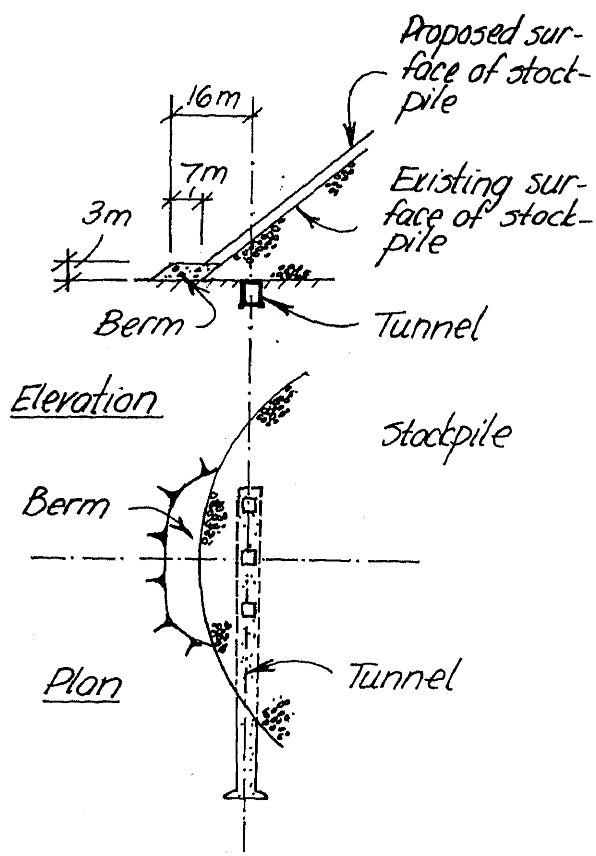

Fig 14 Stabilization of dolomite stockpile