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THE PREDICTION OF THE SETTLEMENT ABOVE SOFT GROUND TUNNELS BY CONSIDERING THE GROUNDWATER RESPONSE WITH THE AID OF FLOW NET CONSTRUCTIONS

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INTRODUCTION

When tunnels are driven through soft ground, settlements are known to occur above them. When driven below pre-existing structures the distribution and magnitude of settlement becomes critical because of the possible detrimental effect on foundations.

The settlements associated with tunnels have been shown to take the form of a trough transverse to the tunnel long axis approximating to a normal probability curve centred about the tunnel, with the greatest settlement occuring over the centre line (Fig. 1). This distribution is therefore able to produce high differential settlements in structures adjacent to the tunnel and develop both compressive and tensile strains within them. Attewell (1977a) has described a method of establishing the size of the settlement trough based on its mathematical form. It does not directly predict the magnitude of the settlement but rather requires that a number of estimated factors be varied by iteration until a suite of variables are produced which experience suggests are correct for the situation. From these data the strains developed by the settlement may be determined from size and shape of the trough.

In this paper the problem is tackled differently. A model is produced whereby the settlement above the tunnel is determined mechanistically. Back-analysis of settlements above two recently reported tunnels in soft ground indicates that the approach will give a good direct prediction of the magnitude and distribution of settlement.

GROUND RESPONSE TO TUNNELLING IN SOFT GROUND

Ground response around tunnels in stiff clay has been considered by Attewell and Farmer (1974a, 1974b) and the approach extended to alluvial clays (Attewell 1977b) and man-made fill (Dobson et al 1978). The common approach is that reviewed by Attewell (1977a), based on the surface response of the ground to a tunnel taking the form of a normal probability curve given by:

$$S = S_{max} \exp\left(\frac{-y^2}{2i^2}\right) \qquad (1)$$

Where S is the settlement measured at any point at a distance y from the tunnel centre line,

S_{max} is the maximum surface settlement and occurs at the centre line of the tunnel, and

i is the distance to the point of inflection on the settlement trough.

The approach relies on an experience of case histories in similar soil types to give the likely values for a number of otherwise unknown factors. From these the likely settlement associated with the tunnel may be established. Little work has been done on considering the response of groundwater and its role in settlement control, although its effect has been appreciated (Attewell et al 1978).

When a tunnel is driven through saturated soft ground it acts as a drain and the groundwater responds by flowing towards it. The response can be modelled by flow net construction in a similar way to that used in other groundwater seepage situations. The result of the hydraulic gradient, initiated by this drainage, is to lower the original pore water pressures in the ground in a way fully described by the net. If full saturation is maintained an increase in effective stress in the ground around the tunnel can be determined if the hydraulic gradient is quantified and the original pore water pressure is known.

The separate groundwater response to a tunnel driven in free or compressed air can be modelled by the appropriate flow net.

The flow net can be constructed for any geometry given by the ratio of tunnel depth to crown, D to tunnel radius r which will be valid for all sizes of tunnel with the same ratio and for all ground conditions regardless of soil type or condition excepting the ratio of vertical to horizontal permeability for which the net needs to be adjusted according to well published rules.

The rate of inflow of water Q may be determined using the D'Arcy Equation:

$$Q = kiA \qquad (2)$$

by determining the hydraulic gradient i and substituting k the ground permeability and A the area over which inflow is occurring.

The hydraulic gradient may be found from the flow net by multiplying the ratio of the number of flow lines N_f to the number of equipotential lines N_e representing the system, and multiplying by the pressure differential across the system p, such that:

$$i = \frac{N_f}{N_e} \times p \qquad (3)$$

Since the flow net is unaffected by the pressure differential the inflow is directly proportional to the tunnel pressure (Fig. 2). Tunnelling experience shows that at a certain value the tunnel pressure will have increased to a point where the tunnel crown becomes dry when inflow at this point has ceased, while seepage continues at lower elevations. This is diagramatically explained by a head distribution diagram (Fig. 3).

As the system is multiphase, consisting of two fluids contained in a solid (air, water and the soil skeleton), the behaviour is controlled by the interaction at the mutual interfaces and the pressures within the fluids. To simplify matters the head distribution may be considered in terms of pressure head.

The pressure head distribution is governed by the unit weight of the respective fluids. For the groundwater this increases with depth in the usual manner. Any downward drainage affects the distribution by increasing its gradient. The low unit weight of air results in a negligible hydrostatic increase over distances of the scale of the tunnel, therefore, there is no effective increase in pressure at different elevations within the tunnel.

As the tunnel pressure increases from atmospheric, here taken as a datum, the head distribution in the tunnel moves uniformly, by a distance equivalent to the increase in pressure (T_1 to T_2 of Fig. 3). The head distribution of the groundwater will also move by the same amount at the lower boundary but since it is fixed at the upper boundary, the amount of shift will be proportionately less at higher elevations (G_1 to G_2 of Fig. 3). At some increase in tunnel pressure the head distribution in the tunnel and in the ground water will intersect. At the point of intersection the pressure head in the two will be the same so no flow can occur at that elevation on the tunnel surface. Flow to the tunnel will continue below this elevation since the groundwater maintains a higher head than the tunnel.

As the tunnel pressure increases the point of overlap or balance point will first occur at the crown. It is clear that only in exceptional ground conditions would the required tunnel pressure be equal to the hydrostatic head of water above the crown. As the tunnel pressure is increased, the balance point moves down so that while flow continues to the tunnel at lower elevations, the direction of flow would seem to be reversed above. However, the interfacial characteristics of air and water in a porous solid medium are such that a greater tunnel pressure can be balanced than that equal to the groundwater pressure head alone. This is because of the capillarity effects produced by the interaction of the three phases at their interfaces acting into the tunnel.

As the tunnel pressure is raised beyond that required to produce a balance point in the crown, the flow net changes in response to the increased zone of no inflow between the crown and the balance point. The inflow Q falls off exponentially from this point since it is proportional not only to the pressure differential p, but also to the ratio of N_f to N_e and the reducing area of inflow A (equation 2 and 3).

CONSOLIDATION ABOVE A TUNNEL INDUCED BY DRAINAGE

By producing the flow net for any tunnel geometry a distribution of effective stress increase brought about by the pore pressure reduction can be determined at any point. According to consolidation theory the ground will settle as a response to an increased effective stress. Since the increase is variable according to the equipotential distribution of the flow net, it follows that the settlement will not be uniform. By taking a vertical line through the flow net adjacent to the tunnel the increase in effective stress can be substituted into standard consolidation formulae to give a measure of the settlement at that point.

In order to check the hypothesis the approach has been used on two recently published case histories. Good agreement is found between settlement predicted in this way both in terms of magnitude and distribution with that reported in the two cases.

1 Willington Quay

Attewell et al (1978) provide a detailed case record of the settlement produced by a tunnel forming part of the Northumbrian Authority's Tyneside Sewerage Scheme situated on the north bank of the Tyne at Willington Quay. The tunnel was excavated to 4.25m diameter under compressed air at an axis depth of 13,375m.

This gives a depth to crown, D to tunnel radius r ratio D/r of 6.29. If a flow net is constructed for this geometry (Fig. 4) the settlement d may be determined at any point away from the tunnel centre line by substitution into the consolidation formula

$$d = Mv \times H \times p$$

Where Mv is the co-efficient of consolidation, H is the vertical distance between two adjacent equipotentials and p is the increase in effective stress at the mid point between the two equipotentials as determined by the equipotentials. The sum of the settlements between each equipotential are assumed to give the settlements experienced at the surface at that point (Fig. 5).

The drive was completed under compressed air. If the balance point is assumed to be about axis level a flow net may be constructed for the groundwater response. From this a distribution of settlement may be determined (Fig. 6a). The coefficient of compressibility Mv was determined from an e-log p curve for material taken from the face of the tunnel (Farmer, 1977), p and H are found directly from the flow net. The settlements calculated in this way are larger than those indicated by the field measurements. However, since the settlement is not instantaneous the rate of settlement may be taken into account by substitution into the standard formula

$$t = \frac{T_v H}{C_v} \qquad (4)$$

The coefficient of consolidation has been determined for the ground from the field settlement records (Fig. 7, of Attewell et al 1978) and the drainage path determined from the flow net. Substitution then enables the settlement at any time to be determined.

Attewell et al (1978) published a distribution of vertical ground movement at 23 days after passage of the shield below the measurement station (Attewell et al 1978, Fig. 11), during which time the compressed air was maintained in the tunnel. The time settlement records show that settlement began almost 10 days before passage of the face. If a distribution is calculated for a period of 33 days by substitution into equation 4, a distribution is found which is remarkably close to the field measurement for the same point in time (Fig. 6a).

After this period a phase of caulking and high pressure grouting occurred around the lining. This would have the effect of increasing Cv by decreasing the effective permeability of the ground, with the result that the rate of settlement would decrease, as is borne out by the field records. Sixty six days after passage of the shield the tunnel pressure was released. The balance point would move accordingly and the new groundwater response can be modelled by redrawing the flow net. By comparing the two flow nets it is immediately obvious that the release of compressed air will widen and deepen the settlement trough (Fig. 4). A fact which has been recorded elsewhere when compressed air is released in a tunnel (Henry 1974, Glossop and Farmer 1979).

When the total settlement is calculated the influence of the grouted lining must be taken into account. Its thickness is insufficient to affect the size of the tunnel opening on the scale of the drawing, but the low permeability barrier set up by the lining will affect the values of the equipotentials. This is because the lining produces a greater energy loss across it than in an equivalent thickness of original ground. Since the upper and lower boundary conditions are unchanged it has the effect of maintaining higher pore pressure in the ground than if the tunnel was effectively unlined. The extreme case would be for a totally impermeable lining to be installed such that no drainage occurs, so no reduction in pore pressures results.

The scale of the drawing is too small to produce a flow net to take into account the energy loss across the lining. However, there is a relationship between the permeabilities of the two materials and the modified shape of the flow net within the second material (Cedegren,1967) where the sides of the rectangle of the modified flow are in the same proportion as the two permeabilities. For the purpose of the analysis the lining is considered to have an effective permeability two orders of magnitude lower than the surrounding ground and to influence a 30cm zone adjacent to the tunnel opening. Thus, the number of equipotentials crossing the system should be increased to 38 and the relevant ones crossing the surrounding ground used to compute the effective stress changes.

The final settlement trough as determined from the flow net for the tunnel in free air (Fg. 6b) can be related to the actual field settlement at 149 days as given by Attewell et all (1978) in their Fig. 12. Since the compressed air was released 66 days after passage of the face or 76 days after field settlements began, the predicted settlements for 83 days may be related to the field measurements at 149 days. Good agreement is shown between these values (Fig. 6b).

2 Stockton on Tees

The same procedure was followed for a one metre diameter tunnel driven 7.0m below ground surface through soft to very soft silty clay at Stockton on Tees (McCaul 1978).

The tunnel was driven entirely in free air and the settlements monitored by the Tunnels Division of the Structures Department of the Transport and Road Research Laboratory.

Using the appropriate $\frac{D}{r}$ ratio the flow net has been constructed and the amount and distribution of settlement determined as outlined above (Fig. 7). The value of Mv was assumed to be similar to that determined for the ground at Willington Quay.

The field settlement records indicate that settlement was complete within the period of measurement with 90% occurring within two months (McCaul 1978) with readings being taken to 200 days. A comparison has been made with the settlement recorded at measurement station D (Fig. 6b). Again, good agreement is found between this and the predicted settlement determined from the flow net.

CONCLUSIONS

A prediction of the size and shape of the settlement trough occurring above tunnels driven through soft ground may be determined using flow net construction. The groundwater response to differing tunnel construction conditions is modelled and standard one-dimensional consolidation theory and the changes in effective stress as determined from the flow net is used to calculate the total settlement. The amount of settlement at any time may be determined from the coefficient of consolidation Cv and the drainage path as determined from the flow net. In this way the development of the settlement trough may be established and the critical situation found as it passes below adjacent structures to the tunnel line.

It seems that consolidation settlement may adequately explain the majority of settlement above tunnels in soft ground, while the deepening and enlarging settlement troughs associated with the release of compressed air are shown to be a natural consequence of the changing groundwater response.

The use of low permeability linings in tunnels is shown not to affect the lateral extent of the settlement trough. The lower the permeability of the lining the smaller will be the maximum settlement experienced above the tunnel.

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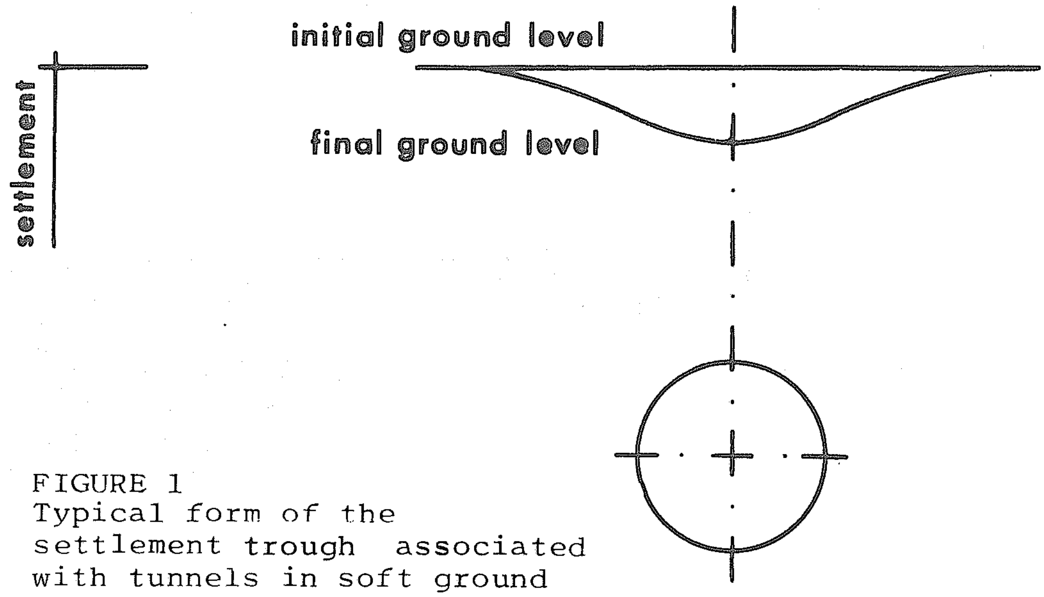


FIGURE 1
Typical form of the settlement trough associated with tunnels in soft ground

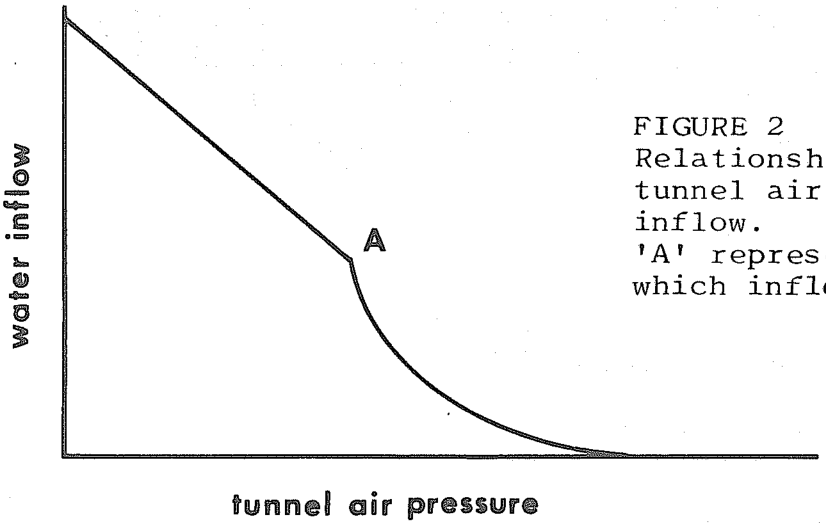


FIGURE 2
Relationship between increasing tunnel air pressure and water inflow.
'A' represents the pressure at which inflow at the crown ceases

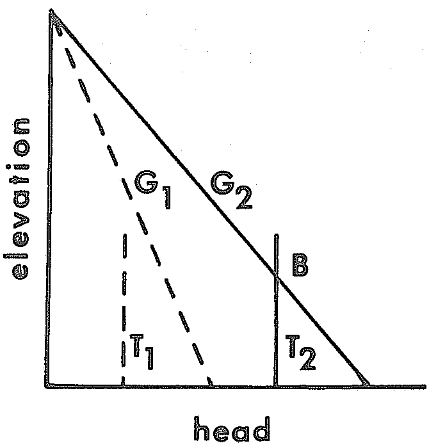


FIGURE 3
Diagrammatic representation of the pressure head distribution of the air in the tunnel T, and of the groundwater G.
This illustrates the response of the groundwater as the tunnel pressure increases from T_1 to T_2 and the development of the balance point, B.

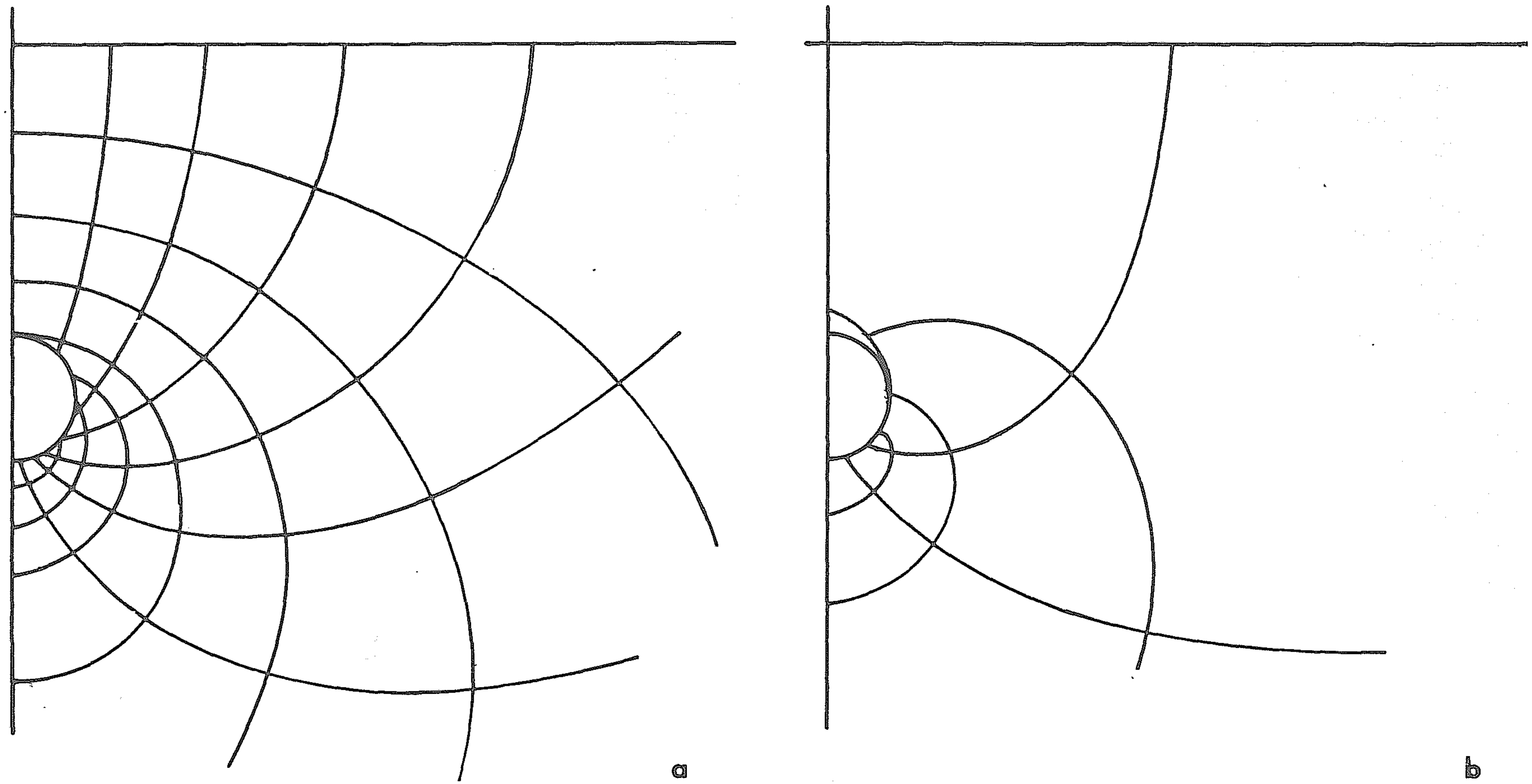
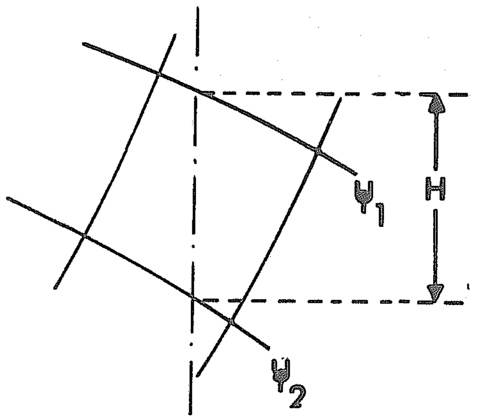


FIGURE 4
Flow net construction modelling the groundwater response to a tunnel at Willington Quay under a) free air and b) compressed air, where the balance point is produced as axis level



$$d = M_v . H . \frac{\Psi_1 + \Psi_2}{2}$$

FIGURE 5
Calculation of the settlement d experienced between two equipotentials of a flow net

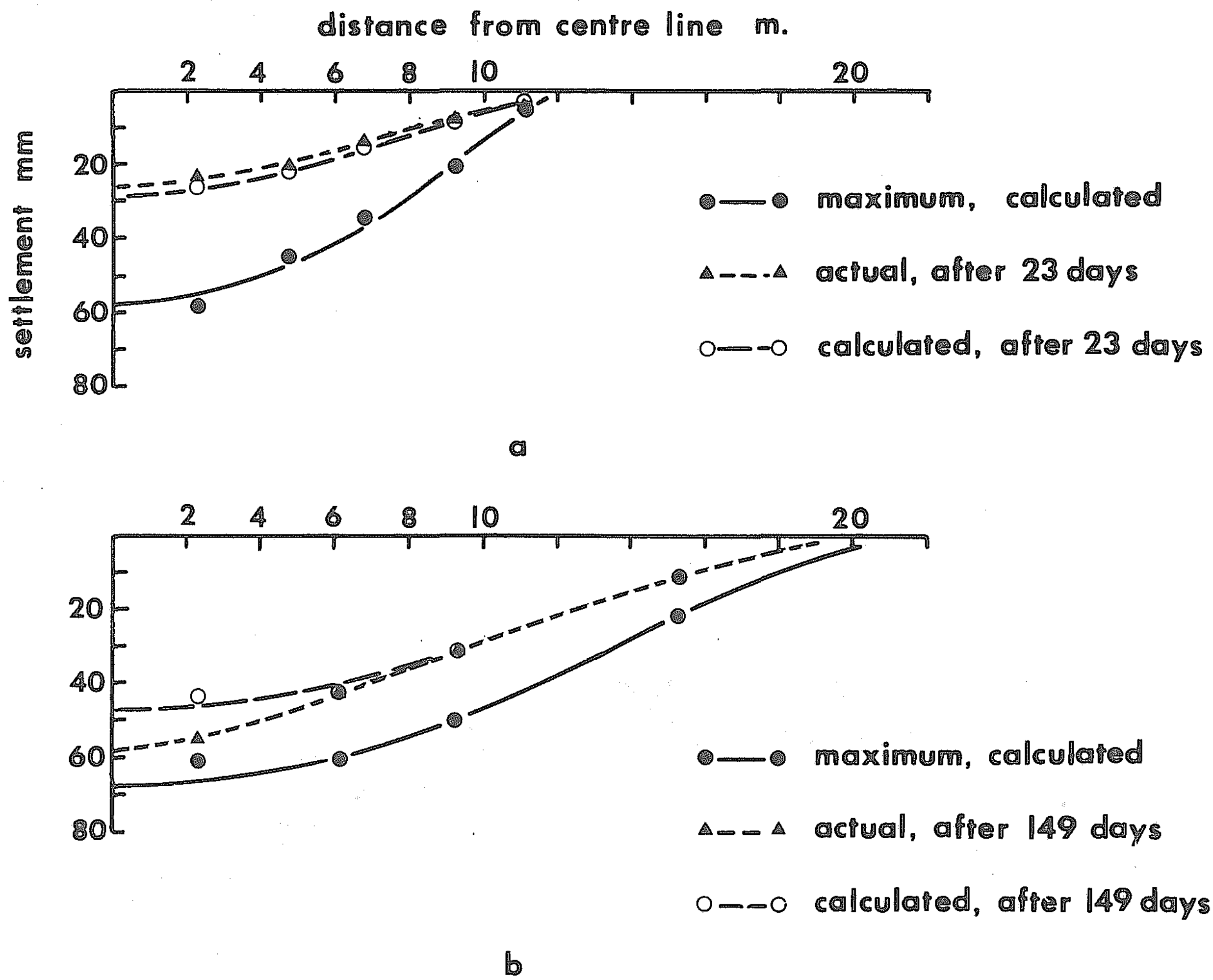


FIGURE 6
The distribution of settlement above the tunnel at Willington Quay with a) compressed air and b) free air in the tunnel. The predicted maximum settlement from the flow net construction, the actual field settlement and the predicted settlement at an equivalent period of time are shown. Partly from Attewell et al 1978.

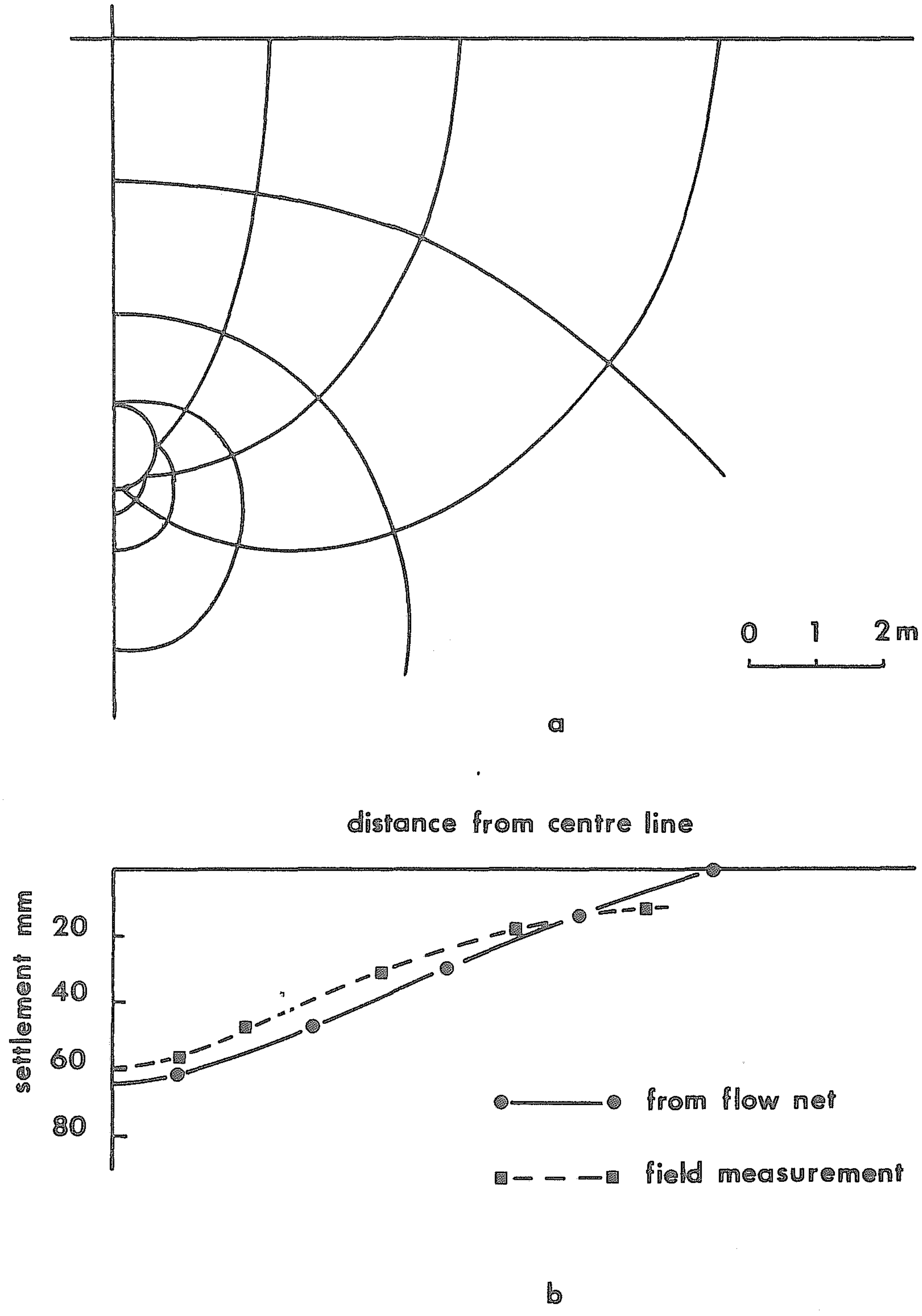


FIGURE 7
a) flow net and b) predicted and actual maximum settlements for the tunnel at Stockton on Tees. Partly from McCaul 1978.