Performance of a deep cofferdam around a collapsed tunnel in glacial clays

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A sheet pile cofferdam has been excavated to over 20m below ground level in order to reinstate a collapsed canal tunnel. The ground at the site is firm clayey till becoming stiff with depth, with horizons of stiff lacustrine clay.

During excavation strut loads were monitored using vibrating wire gauges and displacements of the ground adjacent to the sheet piles were observed using an inclinometer.

The loads developed in the lower supports during the later stages of the excavation were considerably less than those which would be calculated using the traditional approaches. It is suggested that consideration of the horizontal stress field existing prior to excavation leads to an explanation of the bottom strut observations.

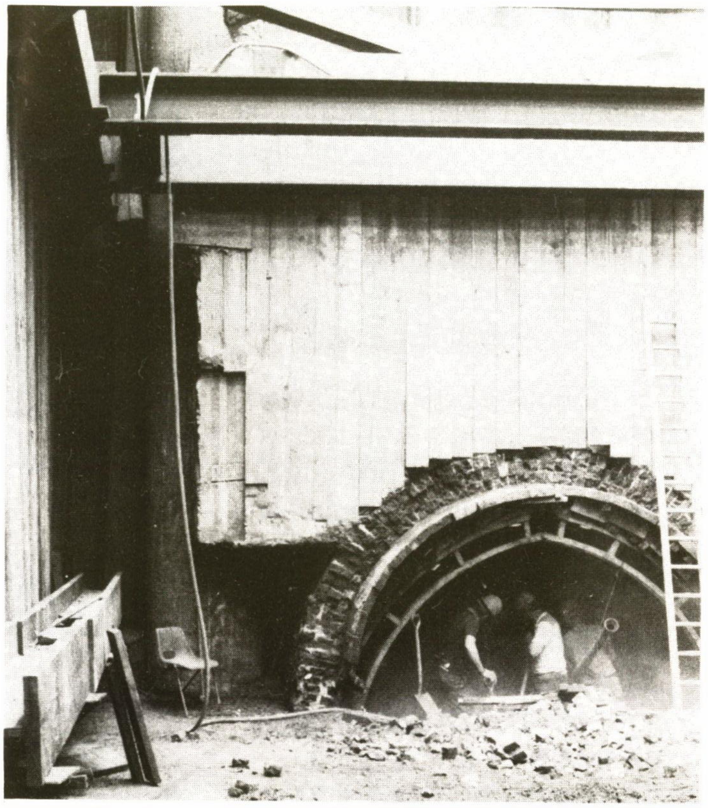

Fig. 1 (right). Details of portal around existing tunnel

Introduction

PRESTON BROOK TUNNEL is owned by British Waterways Board and is at the northern end of the Trent and Mersey Canal. The tunnel was constructed between 1769 and 1775 under the direction of James Brindley and was brick-lined. The waterway width within the tunnel was about 4.3m, the cover to the crown about 14m and the total tunnel length over 1 100m.

Following the collapse of about 14m of tunnel late in 1981, extensive works were undertaken to enable it to be re-opened to traffic. The works involved the reconstruction of the failed section of the tunnel within a sheet pile cofferdam excavated to over 20m below ground level. Special steps were taken to ensure that the stability of the remainder of the tunnel was not threatened. Physical measures taken included the installation of temporary supports within the adjacent sections of tunnel and the construction of bored pile and reinforced concrete portals to form the ends of the cofferdam. These elements are shown on Fig. 1. Monitoring was also undertaken; involving convergence measurements within the tunnel and strut load and deflection observations within the cofferdam itself.

The data presented in this Paper concern the loads and deflections developed towards the centre of the cofferdam.

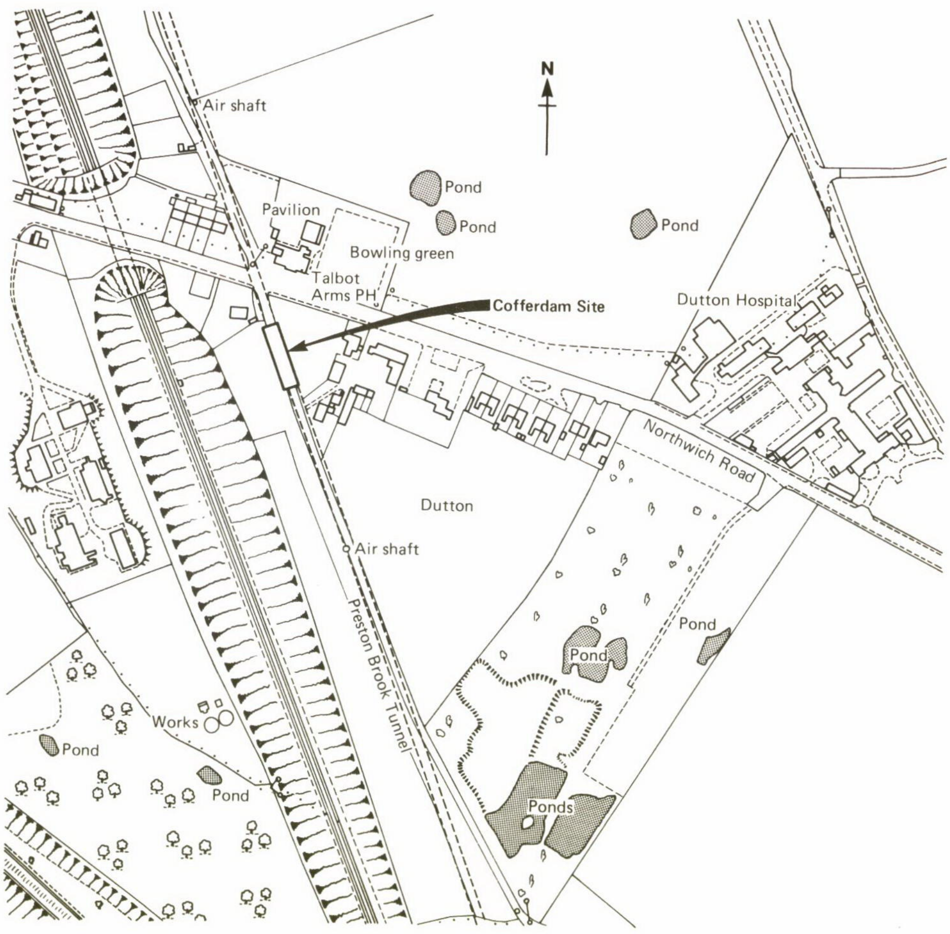

Fig. 2 (below). Site location

Ground conditions

The location of the site is shown on Fig. 2. Although the published geological data show Lower Keuper Marl strata, the materials encountered at the site are now considered to be glacial drift deposits to a depth of about 25m. Below these is a dense sand which is believed to be either a weathered solid formation or a well-sorted sandy drift (Lowe, 1983).

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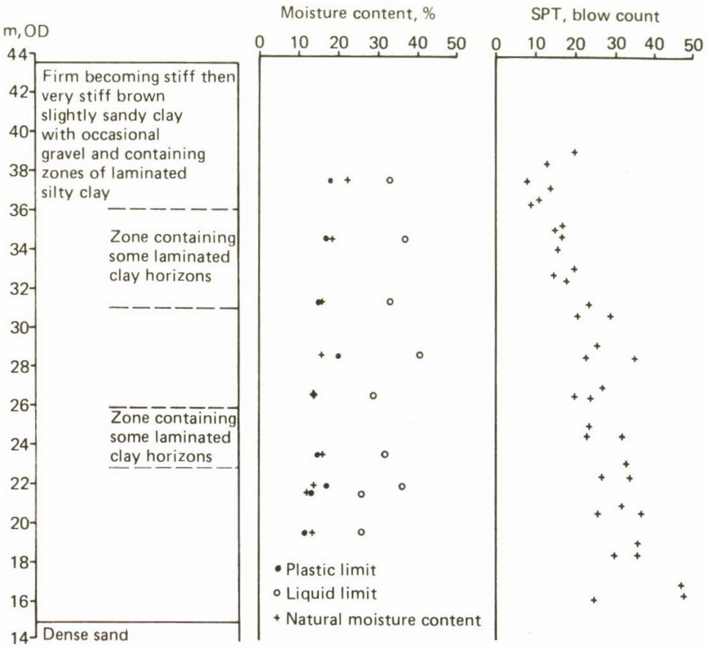

Fig. 3 (left). Summary of site investigation data

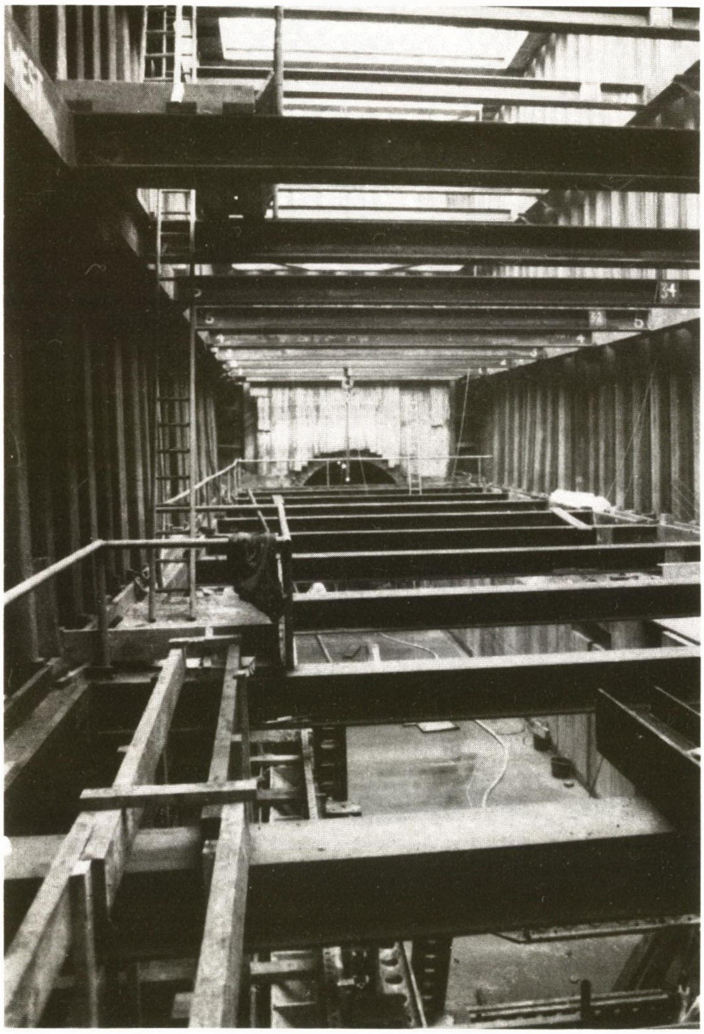

Fig. 4 (left). General view of cofferdam from 4th frame level

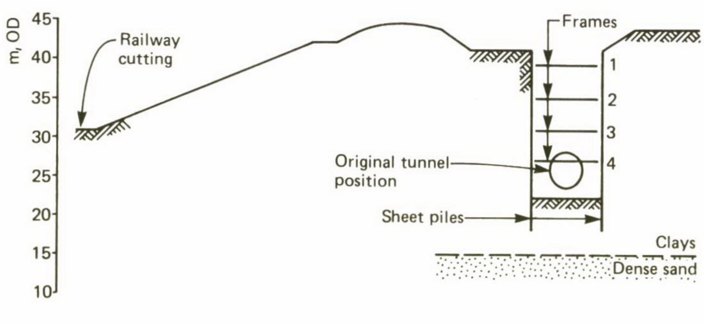

Fig. 5 (left). Cross-section through cofferdam

The glacial drift deposits consist mainly of a sandy pebbly clay till with horizons of laminated, locally silty, glacio-lacustrine clay. These laminated clays are found in two main zones within the cofferdam depth. These zones are broadly between 36.00 and 31.00m OD and between 26.00 and 23.00m OD with the zone depth being typically about 2m.

During drilling at the site U100 samples were taken and Standard Penetration Tests were carried out. Water was encountered; this took the form of slight seepages from sand layers above tunnel level and major strikes at the level of the dense sand stratum. In boreholes penetrating to the sand, the water level rose to between 1 and 2m above the clay/sand interface. Boreholes adjacent to the tunnel also encountered water at and below normal canal water level.

Piezometers demonstrated the presence of perched groundwater above tunnel level.

Laboratory testing was carried out on samples from the boreholes. The results of classification tests on till samples show that the natural moisture content reduces with depth and that the liquid and plastic limits show the same trend (Fig. 3). Typical values of the liquid and plastic limits for the till are 31 and 16 respectively, indicating a clay of low plasticity. The laboratory data show that the moisture content of the laminated materials is somewhat higher than that of the tills, particularly in the lower of the two zones.

Extruded U100 samples from the boreholes showed considerable disturbance in some cases. As a result, reliance was placed on the in situ tests for the assessment of the undrained strength.

The trend of the SPT values (Fig. 3) indicates a fairly steady increase in strength with depth. This is consistent with the observed decrease in moisture content in the tills. The results of Stroud & Butler (1975) suggest that for insensitive materials of the plasticity of the till the undrained strength would be expected to be about six times the SPT value. For design purposes the undrained strength around the cofferdam was taken as increasing from about 50kN/m^2 at the top to about 170kN/m^2 at the base.

Additional samples were taken from a trial pit near the top of the cofferdam for strength testing in terms of effective stresses. The results of the drained triaxial compression tests indicated that the lower bound shear strength parameters were $c' = 17$kN/m^2 and $\phi' = 23°$.

Observations made during excavation

Excavation of the cofferdam, which was 36m x 9m in plan, was started from a platform formed a little below existing ground level at 41.25m OD. Struts and walings to the Larssen 5 sheet piles were placed as material was removed from the cofferdam in a series of horizontal slices. The elevations of the centres of support frames 1 to 4 were 39.25, 35.00, 31.00 and 27.15m OD respectively. The general arrangement of the supports is shown on Fig. 4 and in cross-section on Fig. 5. Excavation was completed at an elevation of 22.25m OD.

In each of the support frames three of the struts near the centre of the cofferdam were fitted with pairs of vibrating wire strain gauges, one gauge on each side of the member on the centreline of the web. The gauges were read regularly as the excavation progressed in order to monitor the development of load.

Two difficulties were encountered with the

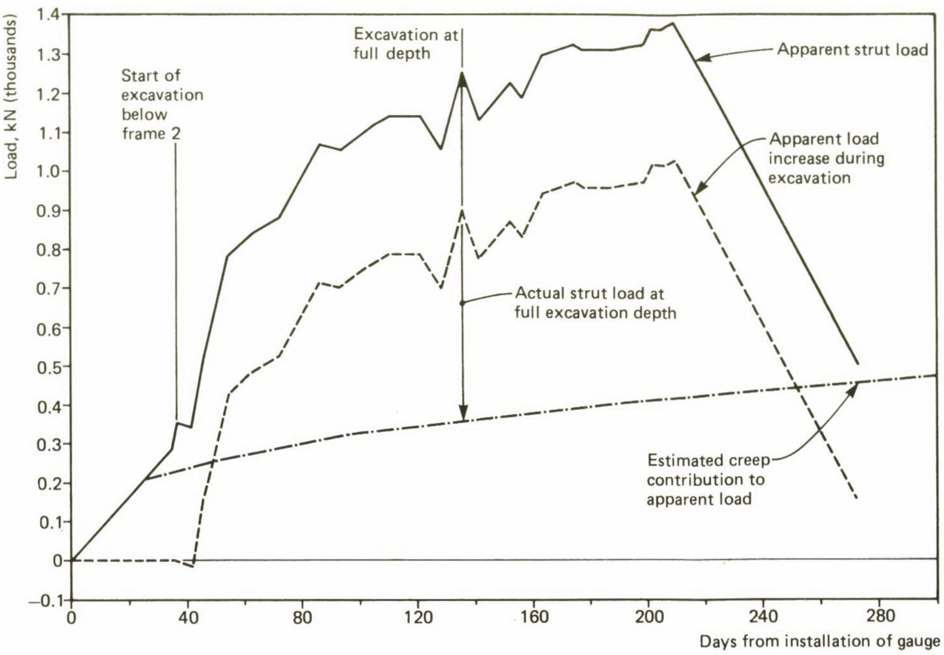

Fig. 6 (above). Plots of apparent strut load and creep load for Strain Gauge 2.2

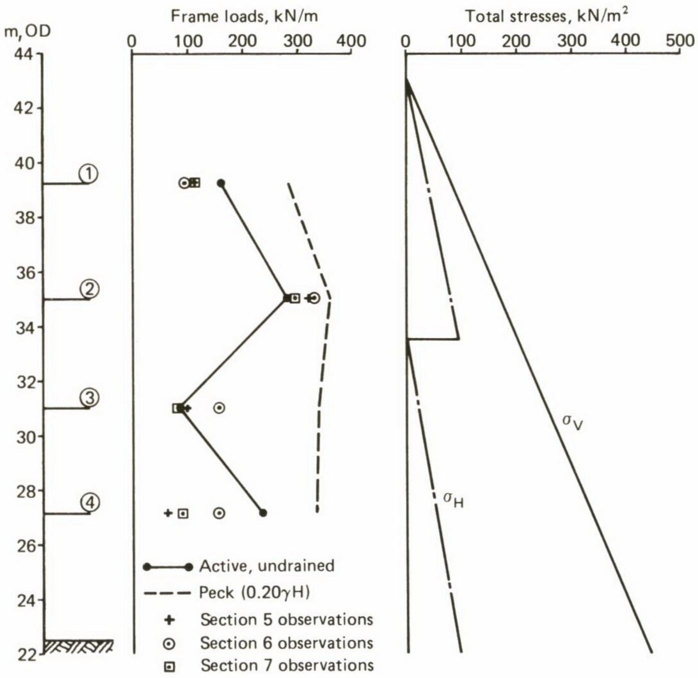

Fig. 7 (left). Calculated and observed support loads

data from the strain gauges; these resulted from the gauges' susceptibility to disturbance by shocks during excavation and their creep behaviour. In order to resolve the uncertainties arrangements were made to measure the apparent loads in the struts after their removal from the cofferdam and reference was made to creep test data supplied by the manufacturers for the type of wire used in the gauges.

In cases of shock loading, sudden changes in apparent load occurred which were not matched by the gauge on the other side of the web. Comparisons between the members of gauge pairs, and examination of the unloaded values allowing for creep, enabled the occurrence and effect of the shock loading to be identified.

To allow for creep, the apparent loads resulting from gauge wire relaxation were estimated for each gauge based on the operating frequency of the gauge, the manufacturer's test data and the cross-sectional area of the strut. The results of this procedure are shown for one of the gauges in Fig. 6.

The results show that the loads in each frame increased sharply as material was removed from directly beneath it and thereafter to a very much smaller extent on the excavation of material from below subsequent frames. In addition to these changes, load fluctuations occurred due to climatic effects, particularly in the upper frames. The mean load for each strut at full excavation depth is plotted in Fig. 7 in terms of load per unit length of cofferdam.

An inclinometer tube was installed in the ground adjacent to the side of the cofferdam in a position between two of the struts fitted with strain gauges. Its lower end was placed within the dense sand stratum. Readings were taken between successive stages of excavation with the objective of determining the depth below the excavation to which significant movements were occurring as a result of the excavation process. Owing to installation difficulties, data are only available for the stages of excavation below Frame 2. The movement increment resulting from each of the last three excavation stages is shown on Fig. 8. It is evident that, contrary to the normal expectation, the observed movements associated with each successively deeper stage of excavation reduced in magnitude.

Discussion

The mechanisms influencing the magnitudes and distributions of support loads in deep strutted excavations for various configurations of excavation and strata have been discussed by Peck (1969) and by Bjerrum *et al* (1972). They have considered the applicability and significance of factors such as arching and the deep seated displacements resulting from the excavation process.

For the cofferdam at Preston Brook, the stability number $\gamma H/Cu$ is less than 3 at full depth. For such a case Peck (1969) suggested that earth pressure analysis might be inappropriate and proposed maximum apparent earth pressure envelopes as design guides. These envelopes were based on a limited number of case histories in which control of lateral movements was important. By taking the smaller of these envelopes of maximum apparent pressure as actual pressures it is possible to make an estimate of support load for each frame by assuming that the sheet piles are cut at the mid-points between frames and then calculating the areas of the relevant portions of the apparent pressure envelope. The calculated strut loads for a trapezoidal envelope as proposed by Peck (1969) are shown in Fig. 7 in terms of load per unit length of cofferdam.

To make use of the approach proposed by Bjerrum *et al* (1972) it is necessary to ascertain whether active conditions are likely to have been mobilised and to what depth. During the excavation at Preston Brook, observations of displacements were made using the inclinometer (Fig. 8). The maximum horizontal displacement at the completion of excavation was 51mm which represents 0.43% of the portion of the excavation depth over which the displacement was observed. Bjerrum *et al* (1972) considered this to be sufficient to mobilise active conditions behind the sheet piling from the ground surface to a depth to below which there are no significant displacements. The inclinometer data show that significant effects are restricted to a very small depth (about 1m) below the base of the excavation.

Taking account of the measured soil strengths and the observed groundwater pressures, it is possible to calculate the horizontal stresses on the cofferdam in the active state. Horizontal and vertical stress distributions for undrained conditions are shown on Fig. 7. The strut loads for each frame can be calculated from the pressure diagrams as described above. The calculated values for undrained conditions are shown on Fig. 7 in terms of load per unit length of cofferdam.

Comparison of frame loads calculated as above with the observed loads show that, in this case, even the lower of the Peck envelopes forms an upper bound to the observations. For loads calculated on the basis of undrained conditions there is reasonable agreement with the observations for the upper frames. It can be seen that the differential between the calculated and observed loads is most marked in Frame 4 where the observed values are much less than that calculated. This is consistent with the smaller than expected movements recorded by the inclinometer for the later excavation stages. The reasonable agreement between observed and calculated

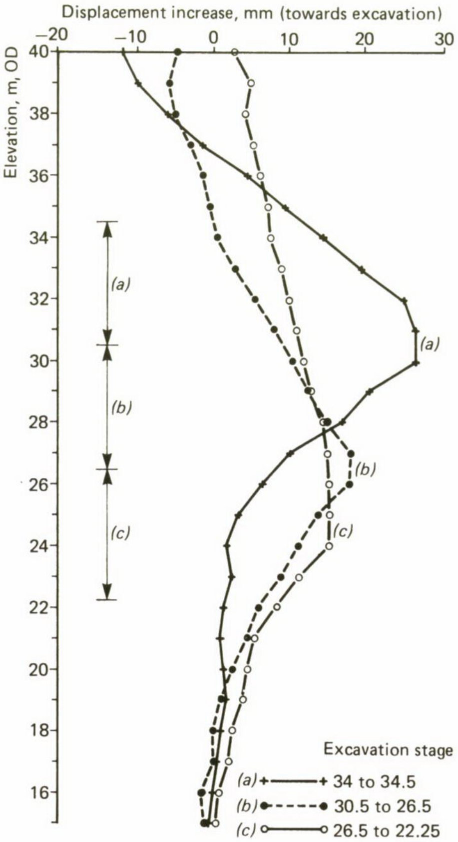

Fig. 8 (left). Inclinometer data

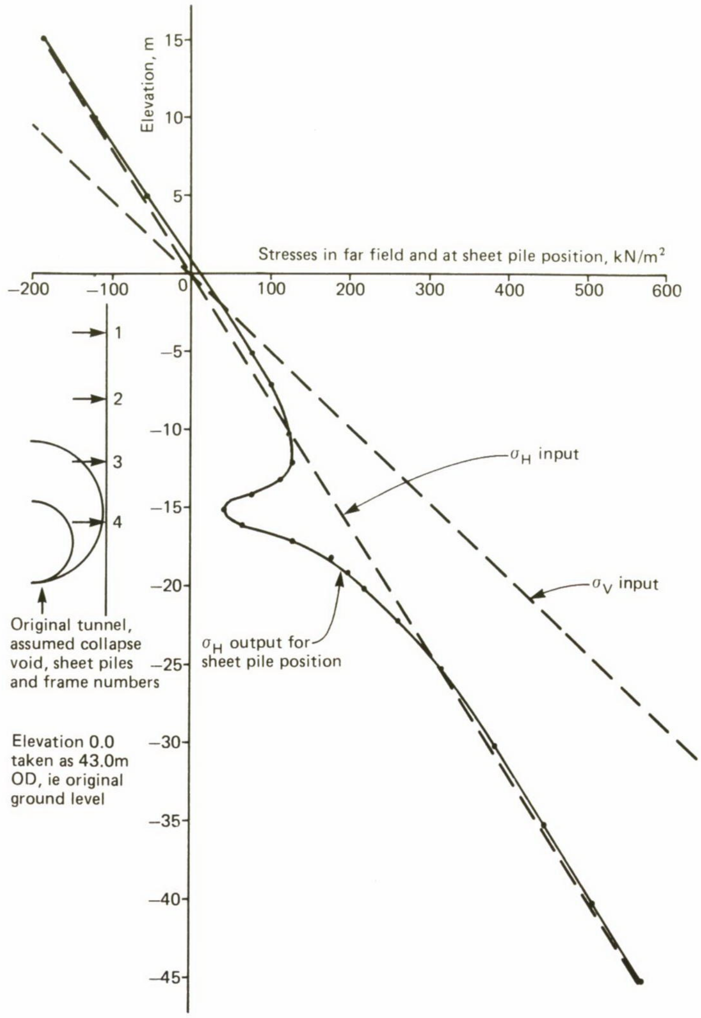

Fig. 9 (right). Results of elastic analysis of tunnel collapse

loads in Frames 1 and 2 might be anticipated, as these are dominated by water pressure effects. Confirmation that the assumption of high water levels near the ground surface is correct is also obtained from the observation during construction of continuous flows of water entering the cofferdam at high levels and, further from the cofferdam, from the groundwater observations in piezometers. Allowing for some drawdown adjacent to the cofferdam would improve the fit between observation and calculation.

The previous events in the area of the cofferdam, namely the construction of the tunnel, the excavation of the railway cutting and the collapse of the tunnel, would all be expected to influence the distribution and magnitude of horizontal stress in the ground. Comparison of calculated loads with those observed suggests that, as might be anticipated, the dominant effect appears to be a local one, concentrated around the collapsed tunnel zone. Horizontal stresses in the ground around the collapsed tunnel zone will have been reduced considerably from their initial values. Arching around the collapsed zone could even have permitted horizontal stresses to be sustained locally at values lower than those predicted from an undrained analysis.

To illustrate the probable effects of the collapse a two-dimensional elastic analysis was undertaken to derive the stress distribution in the ground around an opening taken to represent the collapsed tunnel. The size of the collapse zone was estimated on the basis of the original tunnel dimensions together with some bulking in the collapsed materials. Its position was fixed by that of the original tunnel invert which was shown during excavation to be undamaged. Fig. 9 shows the horizontal stresses at the position of the sheet piles resulting from the set of assumptions incorporated in the model.

The outcome of analyses such as that described above is very sensitive to the assumptions made in developing the model. Choice of a horizontal stress field in this case must take account of the probable over-consolidation of the clays; of the observed distribution of pore water pressures; of the presence of the adjacent unfailed sections of the tunnel and of the effects of excavation of the railway cutting. There is also little data to guide in the selection of the size and shape of the collapse zone. With these difficulties use of this approach for the prediction of stress distributions would be unrealistic. It may, however, be possible to make direct measurements prior to construction by making use of a push-in cell or self-boring pressuremeter (Tedd & Charles, 1981).

Conclusions

Examination of the load and displacement data from the excavation of the cofferdam has shown the marked influence of the collapsed tunnel zone. Arching of the ground around the collapsed section of the tunnel has permitted low levels of horizontal stress to be sustained at the position of the sheet piles. The presence of these low stresses has been demonstrated by the unusually low strut loads at depth and by the limited additional displacements resulting from the latter stages of the excavation. It is considered that prediction of the magnitude of the effects would be very uncertain without direct measurement of the stress field around the tunnel prior to excavation of the cofferdam.

Above the tunnel affected zone the observed support load and its distribution between frames compares well with values calculated using an active earth pressure approach with the assumption of undrained conditions. Some caution is required in extending this approach to all excavations with similar or lower stability numbers and an equivalent support stiffness. It is possible that the presence of the collapsed tunnel in this case, by reducing horizontal stress levels, has had the effect of reducing the effective depth of the excavation and thus limited the extent of load redistribution between frames.

Acknowledgements

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