SAGE CRISP PUBLICATIONS DIRECTORY

Authors:

Powrie, W., Davies, J.N. and Britto, A.M.

A CANTILEVER RETAINING WALL SUPPORTED BY A BERM DURING TEMPORARY WORKS ACTIVITIES

Publication:

ICE CONFERENCE ON RETAINING STRUCTURES, ROBINSON COLLEGE, CAMBRIDGE
pp 418 - 428

Year of Publication:-

1992

A cantilever retaining wall supported by a berm during temporary works activities

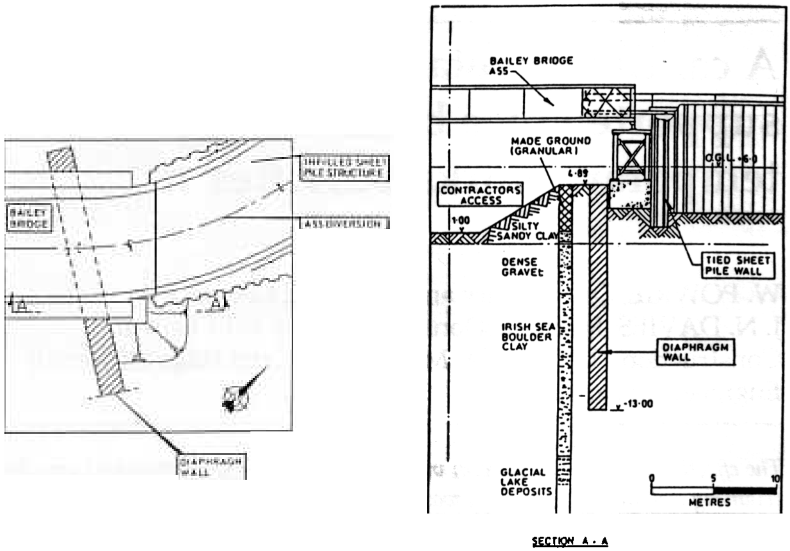
W. POWRIE, Queen Mary and Westfield College, UK, J. N. DAVIES, Mott MacDonald (formerly with Tarmac Construction), UK, and A. M. BRITTO, Cambridge University Engineering Department, UK

The effectiveness of an earth berm in providing support to an embedded cantilever retaining wall during temporary works activities is investigated with reference to a particular case study. Finite element analyses were carried out to assess possible movements of the wall during and for some months after excavation. The effect of the assumed lateral stresses acting on the wall prior to excavation was investigated. The results of the finite element analyses are compared with those of a hand calculation based on a modified limiting stress field and a conservative assumption concerning the effectiveness of the berm, and with the wall movements actually measured.

Introduction

The new A55 North Wales Coast Road passes beneath a causeway on the east side of the Conwy estuary in an underpass whose sides are retained by diaphragm walls. The causeway carries two twin-track railway lines in addition to the old A55 road. A plan view of the site is shown in Fig. 1 (a). Where the function of the diaphragm walls is earth retention alone, the panels are of conventional monolithic design and are installed to a depth of about 19m below ground level. The diaphragm walls supporting the causeway overbridge are formed of T-shaped panels, for additional structural stiffness.

In the permanent condition, the underpass structure is propped below carriageway level with reinforced concrete struts bearing onto cill beams cast against, and fixed into, the diaphragm walls. The specified construction sequence required the use of temporary props at two levels during excavation before placement of the permanent struts. In order to provide access between two parts of the site the contractor, Costain Tarmac Joint Venture, wished to excavate a corridor between the diaphragm walls along the line of the new A55 road. To enable the construction plant to pass beneath the soffits of the overbridge supported by the diaphragm walls, it was necessary to excavate



a) plan b) cross-section

Fig. 1. Plan view of site and design cross-section

locally below the level originally specified for the first row of temporary props. To overcome the need for temporary support, the contractor proposed to minimize wall deflexions and structural stress resultants by leaving wedges or berms of soil in place against the walls. The purpose of the analysis described in this Paper was to investigate the suitability of the proposed new excavation procedure.

Geometry

The analysis was carried out at the most critical cross section, adjacent to the north east pier base of the diversion structure provided for the original A55 road. At this location, imposed loadings from the pad foundation of the Bailey bridge supporting trestles and the retained fill of the approach embankment were closest to the rear of the western diaphragm wall. For the purpose of analyses, the foundation and fill loadings were modelled as surcharges acting at ground level, +4.9m AOD. The design section is shown in Figure lb, and the idealized model in Fig. 2.

Ground conditions

The ground conditions associated with the Conwy estuary form a com-

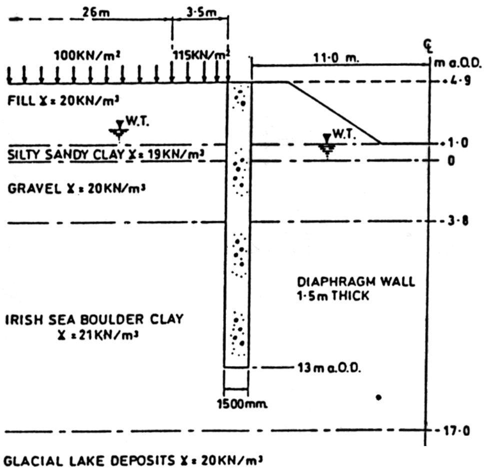


Fig. 2. Idealized geometry and soil profile

plex, interstratified sequence of sediments resulting from the direct influence of successive glaciations. These are overlain by Recent alluvial deposits which also exhibit considerable variation in soil type. Around the periphery of the estuary and beneath the causeway area, soft marine clays and silts have been displaced by Made Ground, which is generally granular in nature.

A typical succession of the soil strata in the area of the causeway underpass is fresh to slightly weathered rockhead overlain by stiff/dense, grey, gravelly North Wales Lodgement Till which is in turn overlain by Glacial Lake Deposits. This last stratum, locally in excess of 20m thick, is a laminated sequence of silts and clays with partings and thicker horizons of fine to medium sand. It generally exhibits a well defined varved fabric. The Glacial Lake Deposits are overlain by stiff overconsolidated brown Irish Sea Boulder Clay. These glacial soils are covered by a deposit of poorly sorted granular material, the origin of which is uncertain. Above the gravels, variable but generally granular Made Ground was encountered with an impersistent layer of silty, sandy clay at the base. This lower cohesive material is considered to represent a remnant of the estuarine deposits removed when the original causeway was constructed by Thomas Telford in 1826. The ground conditions are summarized in Fig. 2.

In situ lateral stresses

The geological stress history of the Irish Sea Boulder Clay and the Glacial Lake Deposits was believed to consist (following deposition) of the removal

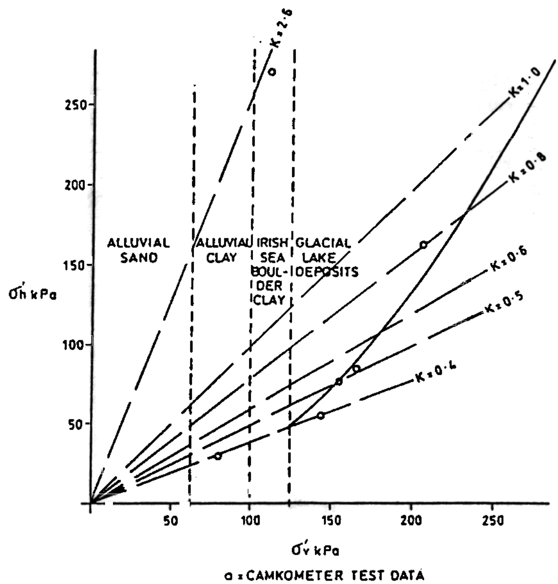
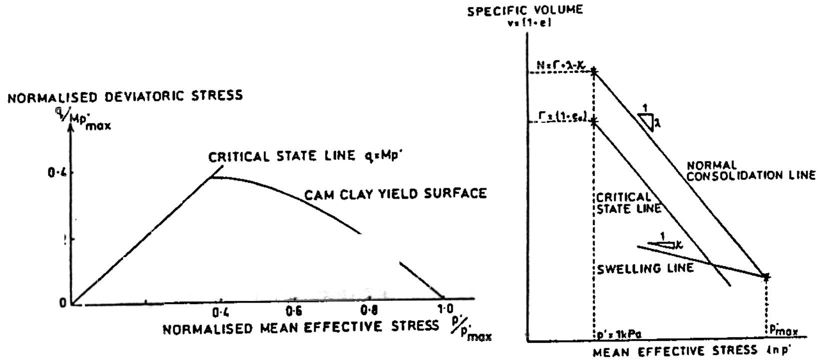


Fig. 3. In situ lateral effective stresses measured using a Camkometer

of an effective overburden of 500kPa. The overconsolidation ratio should therefore decrease with depth, and empirical expressions for the in situ lateral earth pressure coefficient K_o such as $K_o = (1\text{-}\sin\phi').OCR^{\sin\phi'}$ (equation 1: Mayne and Kulhawy, 1982) would suggest a corresponding decrease in K_o with depth. However, the in situ lateral effective stresses measured using a Camkometer indicate a more complex situation (Fig. 3). Although in the Irish Sea Boulder Clay the earth pressure coefficient is much as would be expected, there appears in the Glacial Lake Deposits to be an increase in K_o with depth from a value of approximately 0.4 at the top at the stratum. This may be due to the layered structure of the Glacial Lake Deposits, which results in significant cross-anisotropy. Variations to the general trend might arise from inhomogeneities within the stratum at certain horizons.

For the purpose of the analysis, the in situ earth pressure coefficients in the Irish Sea Boulder Clay were calculated on the basis of equation 1 with the removal of an overburden of 500kPa. In the Glacial Lake Deposits, a linear increase in K_o from 0.4 at the top of the stratum to 1.0 at the base was assumed. In the backfill K_o was taken as 0.5, which is approximately equal to $(1\text{-}\sin\phi'_c)$ for the granular material. The sensitivity of the analysis to the pre-excavation lateral stresses in the upper strata was, however, investigated.



a) Yield surface in q:p' space

b) State paths in v:lnp' space

Fig. 4. Soil model used to represent Irish Sea Boulder Clay and Glacial Lake Deposits

Soil properties

Stress-strain data were not available for the granular fill, nor for the gravel. These strata were therefore modelled simply as elastic-perfectly plastic with a Mohr-Coulomb failure envelope having a critical state angle of shearing $\phi'_c = 32°$. Values of Poisson's ratio $\nu'=0.2$ and normalized shear modulus $G/p'=100$ and 200 were taken as typical of medium-dense granular materials over a shear strain increment of approximately 0.2% (eg Bellotti *et al*, 1989). Representative Young's moduli based on the initial average effective stress p' at the centre of each stratum were 6.25MPa and 15.5MPa for $G/p'=100$, and 12.5MPa and 31MPa for $G/p'=200$. The unit weight γ of both materials was taken as $20kN/m^3$.

The Irish Sea Boulder Clay and the Glacial Lake Deposits were modelled using Cam Clay, with the shape of the yield surface altered to prevent the computation of unrealistically high stress ratios on the dry side of the critical state (Fig. 4 (a)).

The parameters used in the analysis are given in Table 1, and were determined on the basis of laboratory and in situ test data as follows: λ, κ

Table 1. Values of parameters used for the Irish Sea Boulder Clay (ISBC) and Glacial Lake Deposits (GLD).

	λ	κ	e_0	ϕ'_c	G MPa	γ kN/m^3	k_h m/s	k_v m/s
ISBC	.028	.0065	.60	31^o	30	21	$7x10^{-11}$	$7x10^{-11}$
GLD	.040	.010	.857	30^o	35	20	$1x10^{-7}$	$5x10^{-10}$

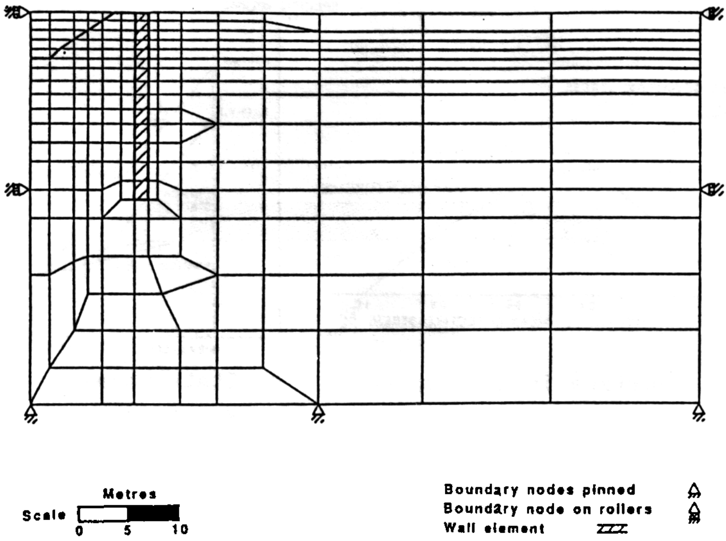


Fig. 5. Finite mesh element

ande $_o$ (defined in Fig. 4 (b)) in one-dimensional compression and swelling from oedometer tests; ϕ'_c in triaxial compression; shear moduli G using a Camkometer; and the vertical permeabilities k_v from isotropic and one dimensional consolidation tests in the triaxial cell and oedometer. The horizontal permeability k_h of the Glacial Lake Deposits was estimated from the results of in situ pumping tests, but for the Irish Sea Boulder Clay it was assumed that $k_h = k_v$.

The back-filled clay was assumed to have the same properties as the Irish Sea Boulder Clay, except that the shear modulus was reduced to 5MPa and the unit weight to 19kN/m^3.

Finite element analysis

The idealized geometry and soil profile were shown in Fig. 2. The Bailey bridge foundation was modelled as a surcharge of 115kPa acting at the retained soil surface, and the 5m high approach embankment as a surcharge of 100kPa. The finite element mesh is shown in Fig. 5. The analysis was carried out using the program CRISP (Britto and Gunn, 1987).

The initial pore water pressures were assumed to be in hydrostatic equilibrium with a water table at +2mAOD. After excavation, groundwater levels were assumed to be at +2mAOD behind the wall and at +1mAOD (ie at formation level) in front. It was further assumed that in the steady state

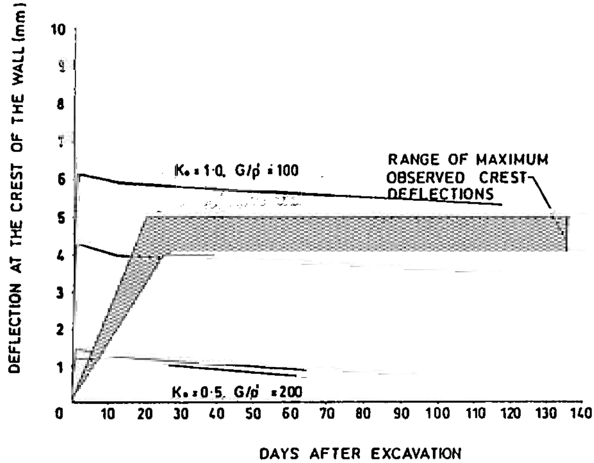


Fig. 6: Observed and computed crest deflexions as functions of time

flownet which would eventually develop, all excess head loss would occur in the Irish Sea Boulder Clay, with pore water pressures in the overlying strata in hydrostatic equilibrium below the appropriate groundwater level. This is reasonable in view of the high permeability of the granular materials, and the relatively small thickness of the layer of backfilled clay.

The 1.5m thick reinforced concrete diaphragm wall was modelled as an elastic material with Young's modulus 15.25×10^6 kPa and Poisson's ratio 0.2. In reality the wall was thicker over the top 2.89m, and the Young's modulus was increased to 175.84×10^6kPa in this region in order to achieve the correct flexural rigidity.

The starting point for the analysis was with the diaphragm wall in place and a ground level of +4.9mAOD. The temporary works activities were modelled using the following procedure

- Application of surcharges behind the wall to represent the Bailey bridge foundation and the approach embankment.
- Three months' dissipation of excess pore water pressures in the Irish Sea Boulder Clay and the Glacial Lake Deposits.
- The instantaneous removal of elements to simulate excavation in front of the berm to a level of +1mAOD.
- Ten months' dissipation of excess pore water pressures.

Throughout the analysis, excess pore water pressures in the fill and the dense gravel were assumed to dissipate immediately. In the Irish Sea Boulder Clay and the Glacial Lake Deposits, excess pore water pressure dissipation was coupled to the rate of consolidation of the soil.

Further analyses were carried out, in which the pre-excavation earth pressure coefficients above the toe of the wall were set to unity in an attempt to model the possible effect of diaphragm wall installation. The computed deflexions of the crest of the wall for all four analyses are compared in Fig. 6 with the maximum wall movements actually measured (using accurate surveying techniques) during the temporary works period.

Approximate analysis

In the approximate analysis (following Bolton *et al*, 1990), the mobilized soil strength ϕ'_{mob} required to maintain the wall in equilibrium was calculated on the basis of the idealized long-term pore water pressure distribution used in the finite element analysis and the assumed distributions of horizontal effective stress shown in Fig. 7. Assumptions had also to be made concerning the lateral pressure exerted by the berm in front of the wall above formation level, and the effectiveness of the berm as a surcharge: these have an important influence on the calculated mobilized soil strengths. In the current analysis, it was assumed conservatively that fully-active conditions would develop in the berm, with the active earth pressure coefficient K_a based on the critical state soil strength and zero wall friction, $K_a=(1-\sin\phi'_c)/(1+\sin\phi'_c)$. The effective surcharge was estimated as 35kPa, which is equal to the total weight of the berm divided by the half-width of the excavation. The stress-strain relationship was based on a consolidated-undrained triaxial compression test on a sample of Irish Sea Boulder Clay, starting from an effective cell pressure of 150kPa and an earth pressure coefficient of unity (Fig. 8).

The result of the approximate analysis is summarized in Table 2. The earth pressure coefficients were taken from Caquot and Kerisel (1948), assuming full wall friction ($\delta=\phi'_{mob}$ on both sides of the wall.

Discussion

It may be seen from Fig. 6 that the results of the finite element analysis are sensitive to the stiffness of the granular materials, and more especially to the pre-excavation lateral earth pressures. However the analyses with the pre-excavation lateral stresses set to unity in the soil above the toe of the wall seem to bracket the measured displacements. The discrepancy between the

Table 2. Result of approximate analysis

K_1	K_2	ϕ'_{mob}	shear strain γ	crest deflexion
0.502	2.17	15.5°	0.4%	29mm

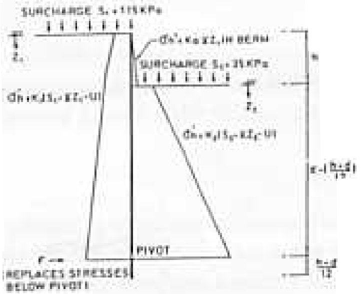


Fig. 7. Lateral effective stress distribution, approximate analysis

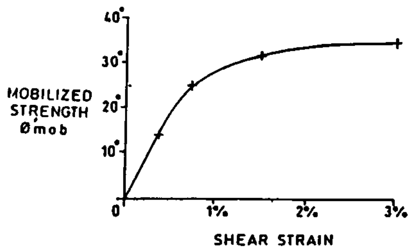


Fig. 8. Relationship between shear strain and mobilized soil strength

initial rates of movement probably arises because no attempt was made to model the actual timescale of excavation in the finite element analysis. All of the finite element analyses indicate a gradual movement of the wall back into the retained soil during the period following excavation. This is probably due to the continuing dissipation of excess pore water pressures in the Irish Sea Boulder Clay and the Glacial Lake Deposits following the application of the initial surcharge loads and excavation in front of the wall, together with the assumption of reversibly elastic soil behaviour.

The displacement of the wall was significantly over-predicted by the approximate analysis, due primarily to the use of a single stress-strain relationship for the Irish Sea Boulder Clay and the fill materials, which did not follow the correct stress or strain path for the soil on either side of the wall. Since the stress-strain relationship started from a mobilized strength of zero (ie K_0=1), the approximate analysis may reasonably be compared with the second pair of finite element analyses. The difference between the calculated displacements is due partly to the discrepancy between the shear moduli used in the two analyses. In the approximate analysis, an increase in shear modulus with depth is implicit in the use of a unique relationship between the mobilized soil strength and the shear strain. At a shear strain of 0.4%, the secant shear modulus according to Fig. 8, based on the pre-excavation average effective stress at a level of -8.4mAOD (ie at the centre of the part of the Irish Sea Boulder Clay above the toe of the wall), is approximately 11.5MPa: this may be compared with the shear modulus of 30MPa used in the finite element analysis. Furthermore, the effect of the berm was deliberately modelled conservatively: this would also tend to increase the calculated wall displacement.

The analyses described in this Paper were carried out after initial calculations based on CP2 (1951) had been completed. The initial calculations had indicated bending moments well within the structural capacity of the wall, but offered no guidance on the likely magnitude of wall movement. A more detailed treatment of the soil/structure interaction is often beyond the scope of much temporary works design, but was carried out in this case to support a revised construction technique which led to a considerable saving in cost.

Conclusions

The deflexion of the wall during temporary works activities was satisfactorily limited by the use of soil berms. The measured wall movements fell generally within the range indicated by finite element analyses in which it had been attempted to model the effects of wall installation. The results of the finite element analyses were dependent to some extent on the effective stiffness of the granular strata. They would undoubtedly also be sensitive to the other input parameters, whose values it was in this case possible to estimate with more confidence. The approximate hand calculation over-predicted the deflexion of the wall: this was due partly to the use of inappropriate stress-strain data, and partly to the deliberately conservative representation of the effect of the berm. The influence of the soil stiffness and of the pre-excavation lateral stresses on the results of either analysis cannot be over-stated.

The implications of the construction sequence in terms of programme and hence cost can be significant. Although unusual in traditional temporary works design, analyses of the type described in this Paper, together with appropriate performance monitoring, can be extremely cost effective as the basis of a "value engineering" approach to the optimization of construction methodology. It is vital, however, that the parameters used in analysis are determined with considerable care.

Acknowledgement

The Authors are grateful to Costain Tarmac Joint Venture for permission to publish this Paper.

References

BELLOTTI R. *et al.* Interpretation of moduli from self boring pressuremeter tests in sands. *Geotechnique,* 1989, 3, No.2,pp 269-292.

BOLTON M.D., POWRIE W. and SYMONS I.F. The design of stiff in situ walls retaining overconsolidated clay: part II. long term behaviour. *Ground Engineering,* 1990, **23**, No.2,pp 22-28.

BRITTO A.M. and GUNN M.J. *Critical state soil mechanics via finite elements.* Ellis Horwood, Chichester, 1987.

CAQUOT A. and KERISEL J. *Tables for the calculation of passive pressure etc.* Gauthier-Villars, Paris, 1948.

INSTITUTION OF STRUCTURAL ENGINEERS. *Civil Engineering Code of*

Practice No. 2 : Earth Retaining Structures. Institution of Structural Engineers, London, 1951.
MAYNE P.W. and KULHAWY F.H. K2 -OCR relationships in soil. J. *Geot. Eng. Div. ASCE*, 1982, 108, GT6 ,pp 851-872.