construction[25] showed that significant movement and reduction in stress in the ground were caused by the installation of the bored secant pile walls. This probably contributed to the smaller bending moments measured on completion of tunnel construction compared with the class A predictions from finite element analyses,[23] as shown in Fig. 19. In these analyses, like those in the Paper, the wall was 'wished in place' and therefore any movements or stress changes in the ground caused by the method used to form the wall were neglected. Better agreement was obtained between the measured maximum bending moment and the results of further finite element analyses[26] (also shown in Fig. 19) in which an attempt was made to model the installation of the wall.

67. If the walls of the Bell Common tunnel had been formed from driven, rather than bored piles, one might have anticipated a local increase in lateral stress in the ground with a consequent rise in the initial earth pressure coefficient.

68. How might installation effects influence the bending moments calculated by the Authors?

C. Raison, *Keller Foundations*

Figure 20 illustrates the results of computer analyses carried out by Ove Arup & Partners for the diaphragm wall originally proposed for the new British Library.[34] This compares two computer runs with wall stiffness varied by a factor of 3. There is almost no change in soil pressures or wall displacements. The significant change in bending moment is a function primarily of the wall stiffness.

70. This effect applies to propped retaining walls, particularly multi-propped walls. Retaining wall movements and soil pressures are far more sensitive to the

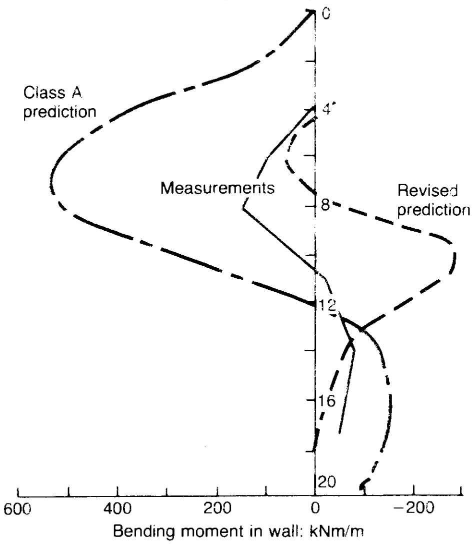

Fig. 19. Measured and predicted bending moment profiles for stage VI[26]

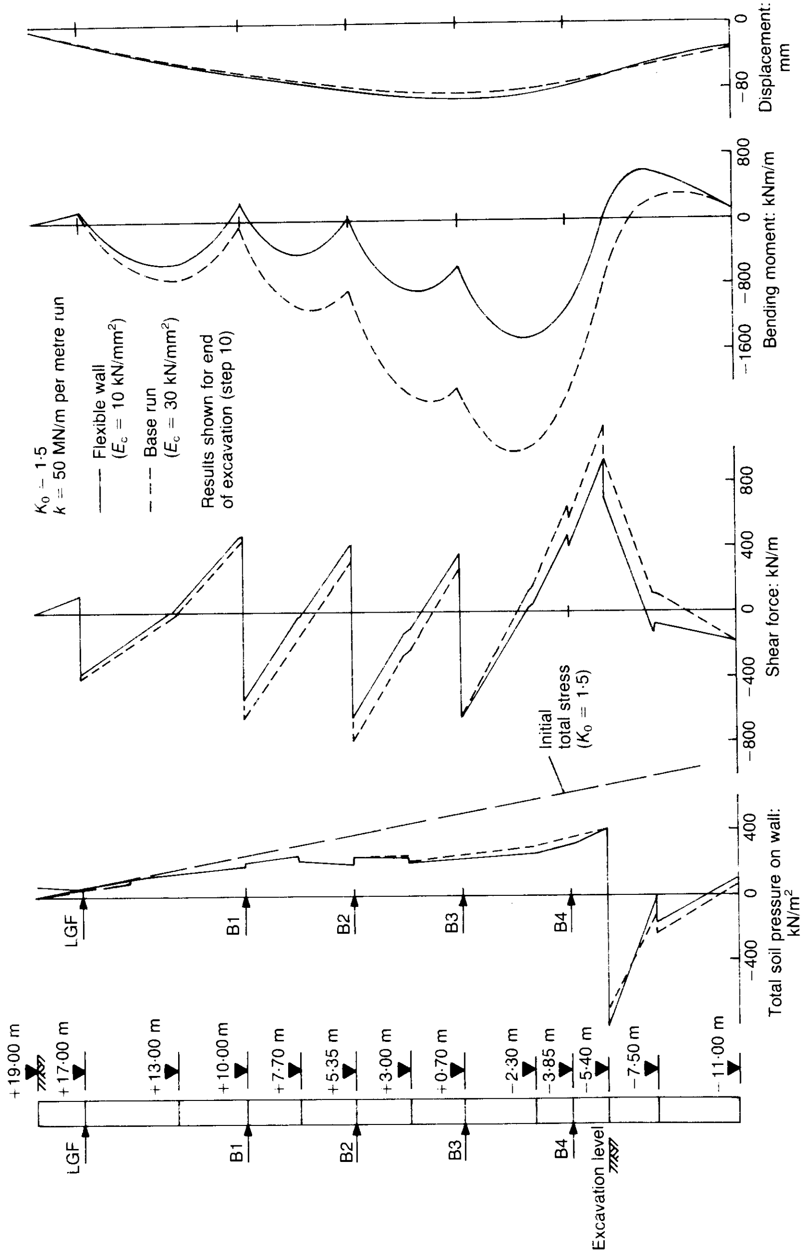

Fig. 20. Effect of varying wall stiffness

assumptions made about initial in-situ stresses and prop stiffnesses than to the wall stiffness.

71. Where retaining walls have to act as unpropped cantilevers or to remain unsupported in basement core or ramp areas, there are often good reasons for using stiffer walls. For the more general stiuation the Paper makes a good case for the use of more flexible walls—an approach which I hope can be adopted more widely.

72. Steel sheet piling is not the only alternative for more flexible walls. Bored cast in place piling can also provide the full range of stiffnesses.

73. Rowe's flexibility number H^4/EI has traditionally been used for comparisons of wall stiffness. Fig. 21 is a copy of Fig. 1(a) annotated to show the more common bored pile stiffnesses for which a value of $H = 20$ m has been taken. Because of the wall length term this comparison is not ideal and the use of ln(EI) is preferable, as in Table 1.

74. Figure 22 shows a direct comparison with Table 1. Stiffnesses for steel sheet piling are shown compared with those for bored piling. EI for the concrete piling has been computed using an uncracked moment of inertia and a Young's modulus of 25 000 MN/m^2, and is given in terms of stiffness per metre run of wall. As can be seen, bored piling can offer stiffnesses covering a similar range to those given by steel sections. Use of mini piles can result in a wall stiffness less than the Frodingham 1N section, although mini piles are usually used with a permanent steel liner to give increased stiffness.

75. The results shown in Fig. 22 do not tell the whole story. The wall stiffness is based on a moment of inertia calculated for an uncracked section. Unfortunately,

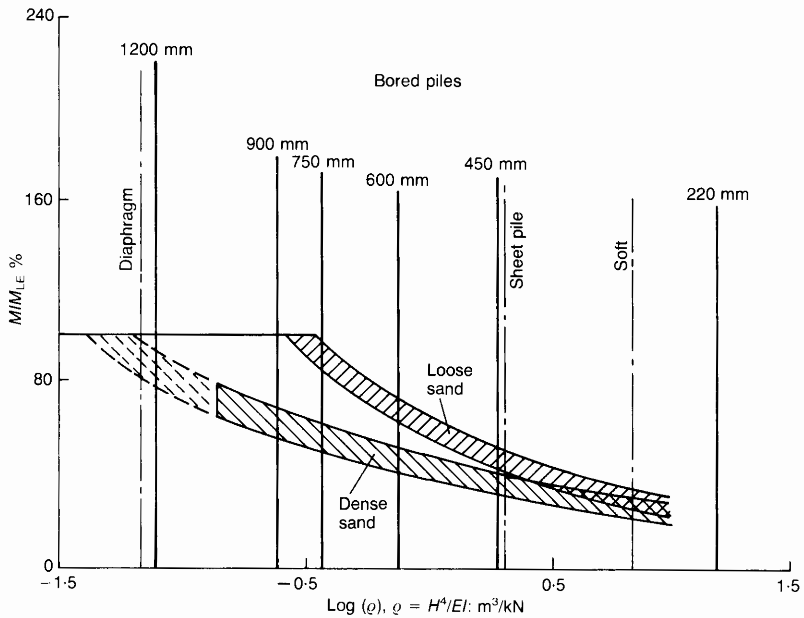

Fig. 21. Common bored piled stiffnesses

for reinforced concrete there is a distinct change in stiffness of the section with the onset of cracking which causes a reduction in the moment of inertia.

76. Methods exist for determining the moment/curvature relationships for a particular reinforced section. The example in Fig. 23, carried out by Ove Arup and Partners, is based on the CEB–FIP method applied to a 1 m thick diaphragm

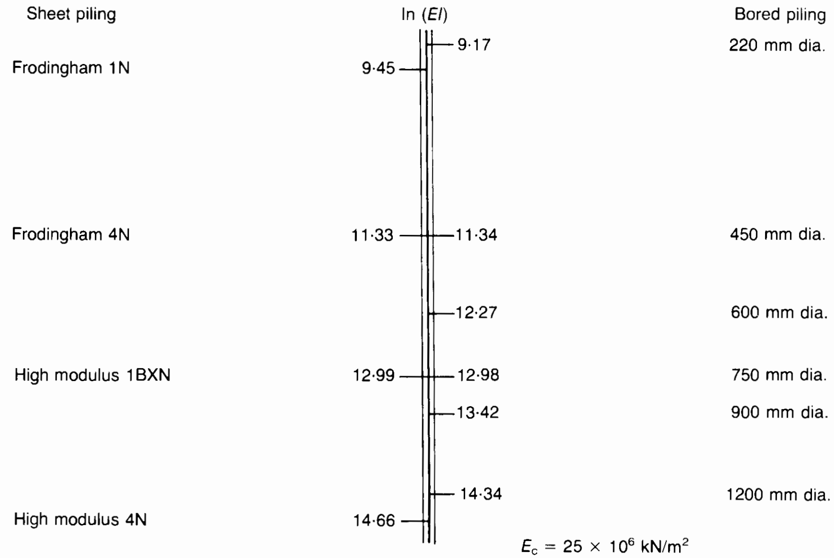

Fig. 22. Retaining wall stiffness per metre of wall

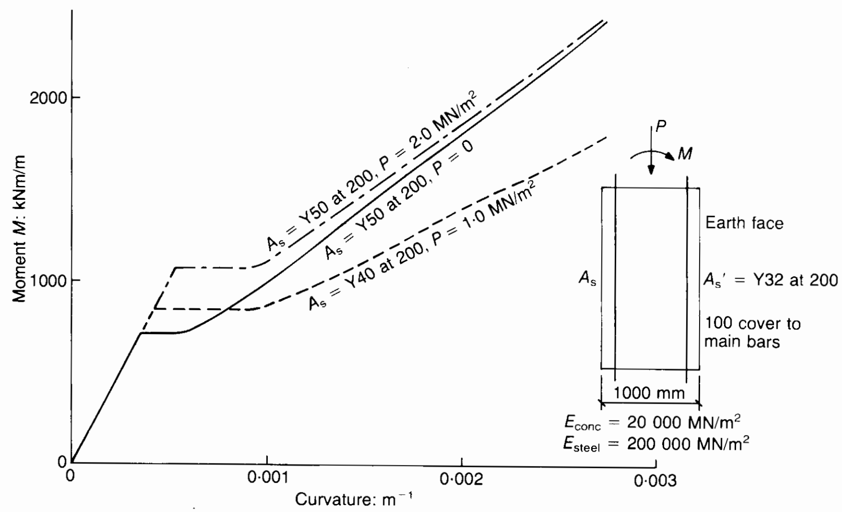

Fig. 23. Moment–curvature relationships for diaphragm wall, CEB–FIP method[35]

wall.[35] Three cases are shown with varying reinforcement and axial forces. The bi-linear relationship should be noted.

77. This form of analysis is important, particularly for serviceability checks where it is necessary to predict the likely maximum crack width. It is also important in order to model correctly the non-linear behaviour of the structure as well as that of the soil in any soil–structure interaction analysis.

78. The example in Fig. 24 shows results for a 600 mm bored pile with a 6Y32 reinforcement cage.[36] The behaviour is again non-linear and shows two distinct phases. Fig. 24 also shows the effective pile stiffness *EI*/moment relationship for the same bored pile, and illustrates the difficulty in choosing a representative value for use in soil–structure analyses. Similar relationships can also be computed using long-term concrete properties.

79. I hope that the approach proposed in the Paper can be adopted more widely, not just when using steel sheet piling but also with reinforced concrete bored piles. It is to be hoped that comments about the difficulties of choosing

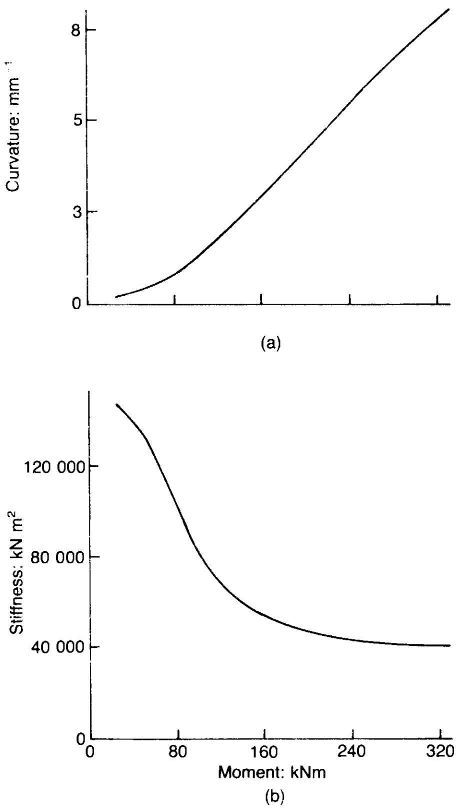

Fig. 24. Short-term ultimate bending moment, load case 1: (a) moment–curvature plot; (b) moment–stiffness plot

representative stiffnesses for the wall will act as a timely reminder. Although sophisticated computer analysis can be carried out to investigate soil–structure interaction, there are still areas where further investigation is required.

T. Paul, *Ove Arup and Partners*

With regard to the new British Library, sheet piles were used as retaining structures during the construction of both single and double level basements.

81. Two separate packages of sheet piling were carried out in roughly the same area of the site at different times. In both cases, the sheet piles were driven from a ground level of +16·0 m OD. The ground profile consisted of London clay overlying Woolwich and Reading beds with the interface at approximately −1·0 m OD. Undrained strength values for the London clay were 50–250 kPa and SPT *N* values were 20–60, both parameters increasing with depth.

82. The first package of sheet piling was carried out using the Pilemaster system. Approximately 250 Frodingham 5N sections were installed to form a retaining wall of 110 m plan length. The sheet pile sections were driven 11–15 m below ground level, retained heights being up to 7 m. A special measure was adopted to improve the driveability of the sheet piles. This consisted of a metal block welded to each sheet pile approximately 3 m from the toe of the pile. As the pile was driven the block forced the soil away from the pile face, reducing the adhesion forces and thereby ensuring easier penetration. Less than 3% of the total number of piles failed to reach the design toe levels.

83. In 1989, a second sheet pile retaining wall was installed in a similar region of the site. Frodingham 4N sections were used for this contract due to the unavailability of 5N sections. Analysis of the driving records shows that for those sections with design toe levels 13–14 m below ground level, all were driven to the design levels. However, approximately 15 sheet piles with deeper design toe levels (14–16 m OD) reached refusal more than 2 m short of the design levels. To accommodate this problem with minimum delay to the overall programme the basement construction sequence was modified to reduce the retained height in the critical areas.

84. These case histories illustrate the importance of assessing sheet pile driveability and choice of sheet pile section. In London, the second case history described shows that driving problems may occur in London clay, particularly at depth where the silt/fine sand content increases towards the interface with the Woolwich and Reading beds.

Dr H. D. St John, *Geotechnical Consulting Group*

I want to describe a site where steel sheet piles have been used as a remedial measure because the bored piles gave problems. The site originally contained two adjacent buildings, both founded on shallow footings at basement level. One building was demolished and an additional basement level constructed beneath the new structure. In order to be able to excavate the basement it was necessary to install a peripheral cut-off wall which extended through the upper alluvial deposits into the London clay.

86. The London clay was about 8 m below the original basement level. The upper alluvium comprised loose to medium dense sands, soft clays and peats. A thin gravel layer was present just above the London clay. A significant feature was the proximity of the (tidal) water-table to the original basement level, which made it difficult to install a bored pile wall without losing ground into the bore and