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Long term performance of an anchored diaphragm wall embedded in stiff clay

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Scope of project

Ground anchorages are commonly used to support retaining walls where there is no other viable practical means of construction. Thus ground anchored structures are commonly used in areas of restricted land take, such as trunk roads in urban areas, and in areas of restricted access, such as steep slopes above roads in hilly areas.

The objective of the project is to provide advice on the design and construction of rock bolts and ground anchorages. Issues of particular interest include the long-term properties of the plastic ducting used to encapsulate the tendons, and the creep behaviour of anchorages in clayey soils and weak rocks.

Summary of report

An anchored diaphragm wall was constructed in 1972 in heavily over-consolidated London Clay as part of a relief scheme for the North Circular Road (A406) in Neasden, North-West London. During construction, instruments were installed in a section of the wall by the Building Research Establishment (BRE) who monitored the performance of the wall until the mid-1970s. In 1988 additional instrumentation was installed by the TRRL (now TRL) and the structure has been monitored by the TRL since then. This report examines the performance of the wall from the end of its construction.

The findings of this investigation indicate that the pore water pressure regime has attained equilibrium but is subject to seasonal fluctuations. The measured lateral earth pressures behind the wall appear to have remained fairly constant since 1988 although those in front of the wall seem to show a small but steady downward trend with time. This may indicate that stresses generated as a result of installation of the instruments in 1988 are still dissipating some 12 years later. Although a number of the load cells on the anchorages have malfunctioned, the data acquired suggest that loads in the anchorages have slowly but steadily increased since the end of construction. Similarly, measurements of lateral ground movements suggest that both the wall and retained ground have continued to move over the last 10 years or so: movement has been measured as far back as 19 m behind the face of the wall.

1 Introduction

One of the objectives of a research project undertaken at the Transport Research Laboratory (TRL) for the Highways Agency (HA), was to review the long-term performance of ground anchorages embedded in clayey soils. To this end, a survey of the ground anchored structures on the motorway and trunk road network was undertaken. Although the database of such structures was incomplete, it would seem that only a few structures are supported by anchorages embedded in clayey soils.

It was thought that a review of the long-term performance of ground anchorages in clayey soils would either highlight in-service problems, such as creep of the anchorages, or establish that there were no such problems. There is anecdotal evidence that engineers and clients will not accept the use of anchorages in clays because of the *perceived* problem of long-term creep. For safety and economy it is necessary to critically examine this position.

The provision of an overall view of the performance of a number of structures supported by anchorages installed in clayey soils had to be substituted by a more detailed review of the long-term performance of an instrumented ground anchored structure on the A406 in Neasden, North-West London. There are dangers in extrapolating the performance of one structure to others, which for example have different types of anchorage. But a review of the performance of this structure would be particularly useful because the structure was much more heavily instrumented than is normal and the data are accessible.

The anchored diaphragm wall, embedded in the heavily over-consolidated London Clay, was constructed in 1972 as part of a relief scheme for the North Circular Road (A406) in Neasden, North-West London. During construction, instruments were installed in a section of the wall by the Building Research Establishment (BRE) who monitored the performance of the wall until the mid-1970s; the instrumentation has been described by Sills et al (1977). Site monitoring was then undertaken by the TRRL (now TRL) and additional instrumentation was installed by them in 1988. Monitoring was undertaken regularly over the following two years or so, but rather intermittently from then until recently. This report provides an update of the performance of the wall.

2 Construction details

As is evident from Figure 1, the road scheme at Neasden was extensive and involved the construction of approximately 600 m of diaphragm wall and the installation of 580 ground anchorages.

The instrumented section of the retaining wall was sited on the north-east side of Neasden Lane underpass at the end of Winslow Close (formerly Elm Way) at grid reference OS211861. At this location, the overall height of the wall is 13.5 m and it has a retained height of 8.5 m. Each full-height reinforced concrete wall panel (4.57 m wide and 600 mm thick) is supported by eight anchorages (in four rows of two) inclined at 20° to the horizontal and extending some 16 to 18 m into the retained ground. But, to minimise encroachment under nearby houses, in areas adjacent to the instrumented section, some of the anchorages were inclined at 40°. A plan and cross section through the instrumented part of the wall are shown in Figure 2.

Each anchorage was constructed with seven underreams. The tendon arrangement (for the instrumented anchorages) consisted of four 15.2 mm diameter steel strands greased and sheathed in polypropylene. The design working load of the instrumented anchorages was 400 kN and they were pretensioned in service to 115% of this value (i.e. 460 kN). According to BS 8081: 1989 the characteristic strength of a 7-wire strand steel tendon of 15.2 mm diameter is 232 kN. Thus the characteristic strength of four such strands is 928 kN, and the ratio of this strength to the lock-off load is close to 2.0. For permanent anchorages, Table 2 of the Standard recommends a minimum factor of safety against rupture of 2. (The design strength of the tendons has yet to be confirmed.)

Behind the retaining wall, 100 mm diameter sandwicks were installed at 1.15 m centres along the length of the wall to about 9 m depth. Weep holes through the wall connected into a 300 mm diameter drainage pipe running longitudinally at a depth of about 1 m below the level of the carriageway.

3 Soil properties

The properties of the in situ material can be determined from a number of sources including the borehole records from the original site investigation conducted in 1969 and from the installation of instruments in 1988. Self-boring pressuremeter (Camkometer) and dilatometer tests were conducted in May 1988 to establish the in situ properties of the London Clay, and the consistency and moisture content of the soil was determined on recovered soil samples. The results of the testing are presented in Figures 3, 4 and 5, all of which have been reproduced from Carswell et al (1991).

The soil profile at a position located 1.5 m from the wall is shown in Figure 3a. At this location fill material is found to a depth of 2.6 m (due to excavation and backfilling operations related to the construction of the wall) but generally fill material was only found to a depth of about 0.5 m. As shown in Figure 3a, weathered brown London Clay is found at depths of between about 2.5 m and 8 m, overlying grey-blue London Clay. Sills et al (1977) reported that, at this site, the underlying Woolwich and Reading beds are found at a depth of about 30 m.

A fuller description of the soil properties has been given by Carswell et al (1991).

4 Instrumentation

A number of different types of instrument were installed during the construction of the wall. Details of the instrumentation have been provided by Sills et al (1977): the instruments included magnet extensometers, inclinometers, survey stations, piezometers, and vibrating wire load cells on the anchorages supporting panel number 26.

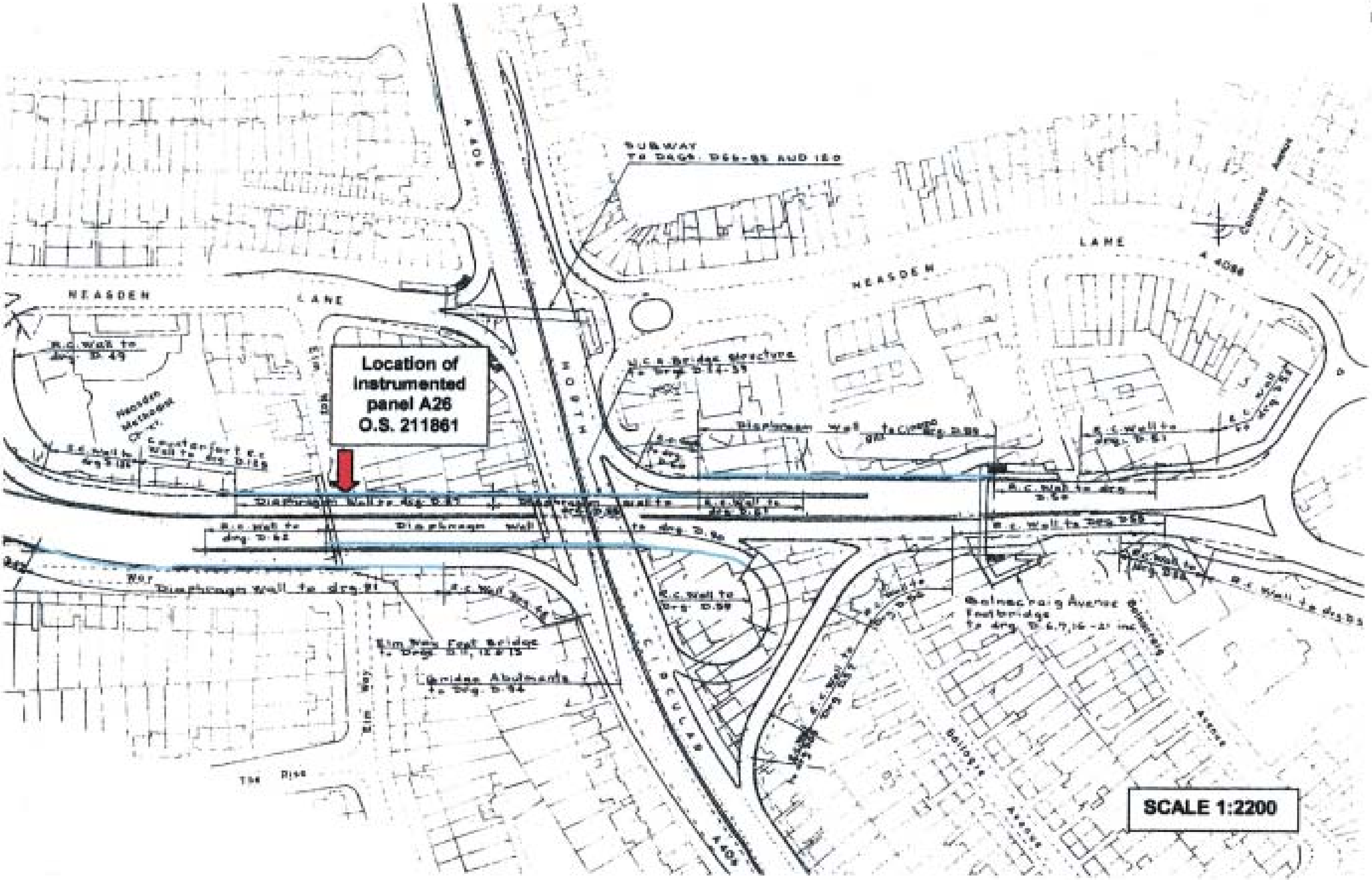
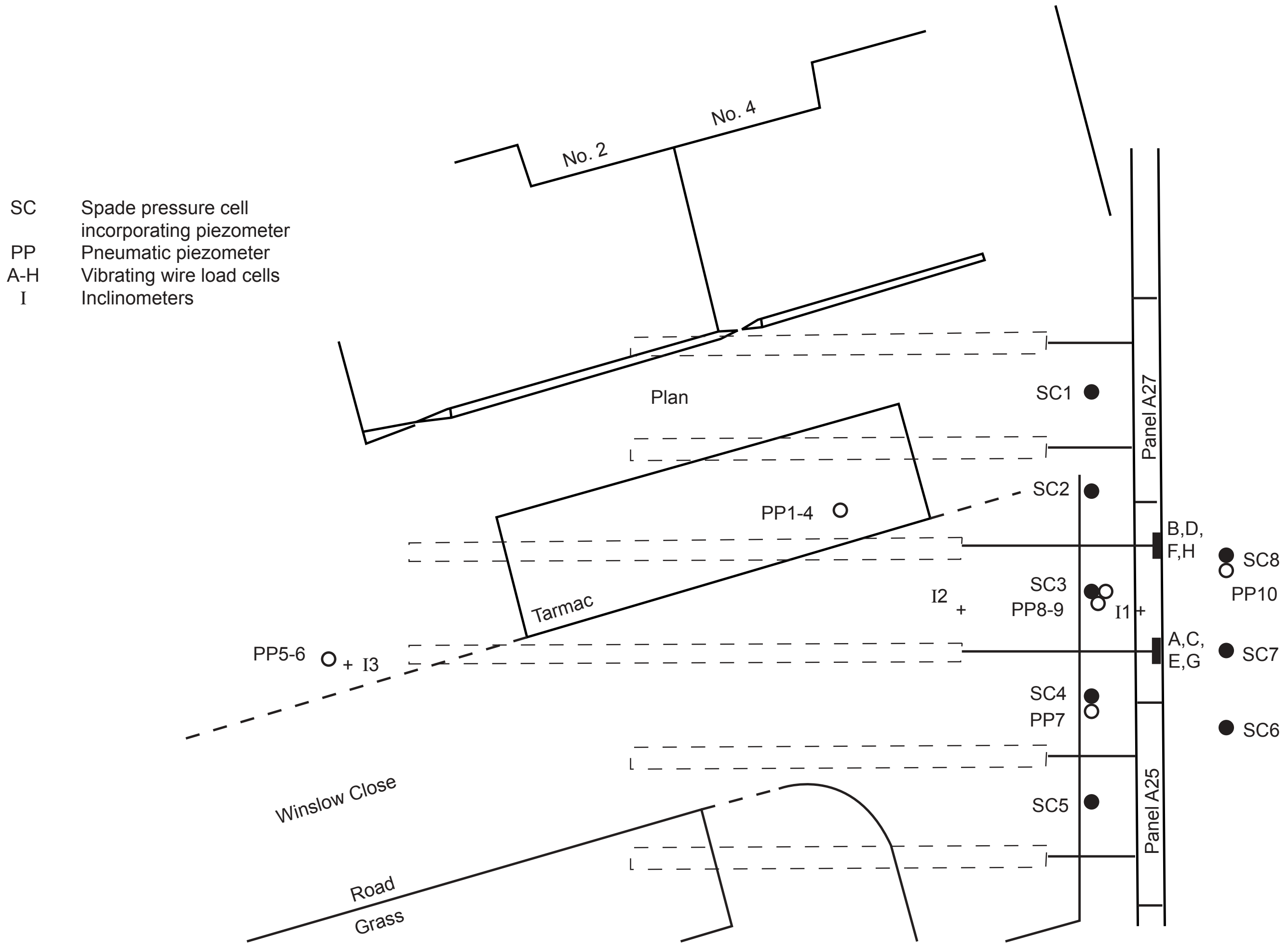
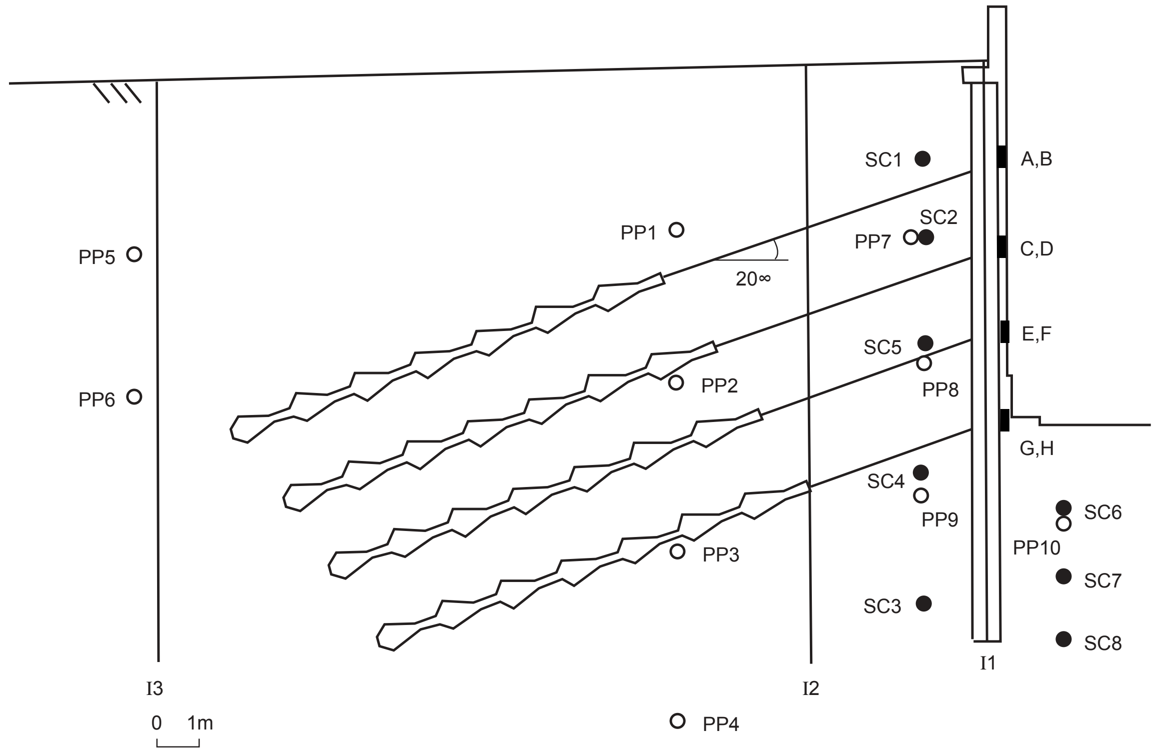


Figure 1 Neasden Lane scheme showing the locations of the anchored diaphragm retaining walls and the instrumented panel A26



2(a) Plan View



2(b) Section through panel A26

Figure 2 Instrumentation at Neasden Lane Underpass, from Carswell et al (1991)

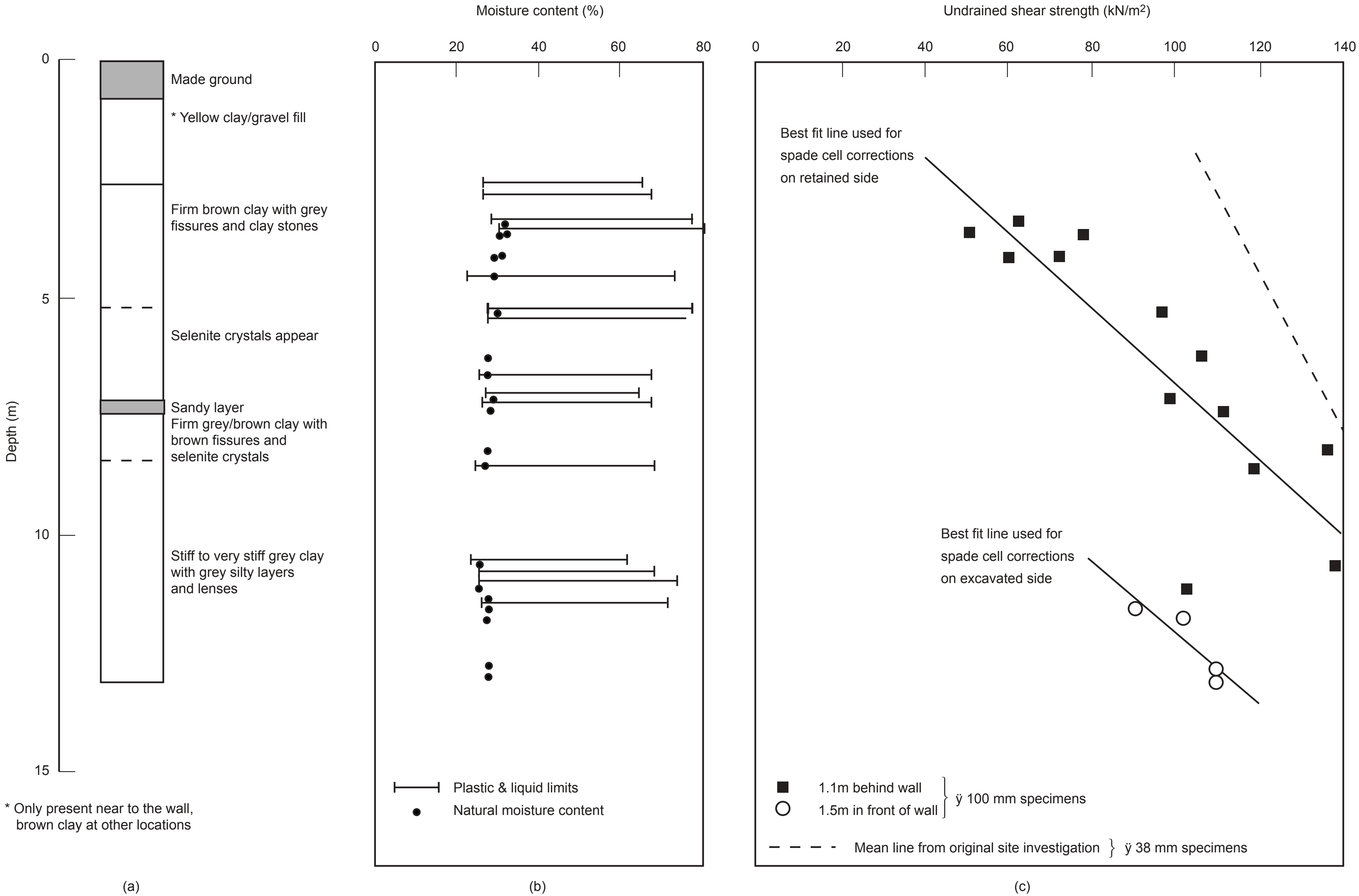


Figure 3 Soil properties — Neasden Lane Underpass, from Carswell et al (1991)

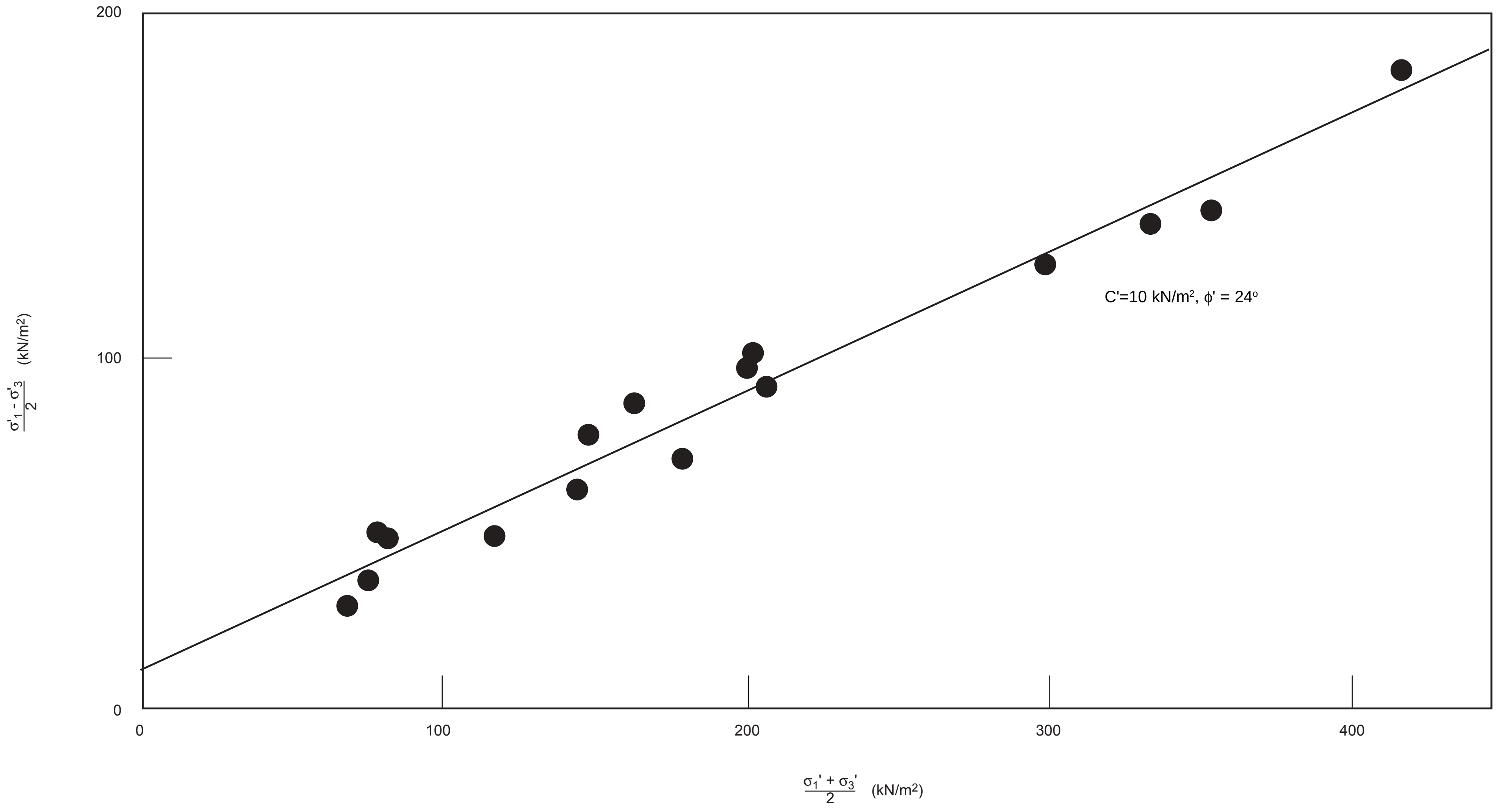


Figure 4 Results of consolidated undrained triaxial tests on undisturbed samples, from Carswell et al (1991)

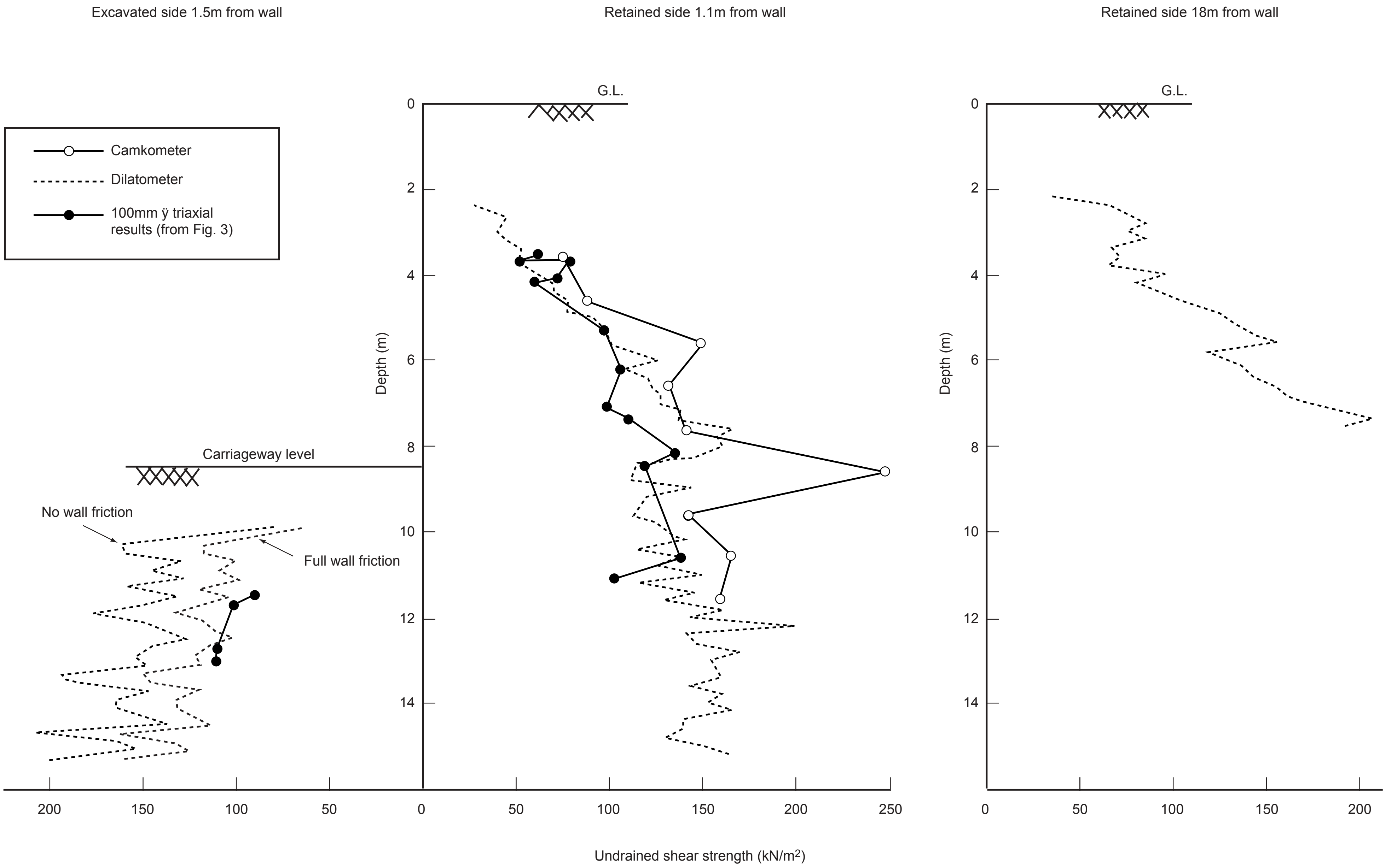


Figure 5 Variation of undrained shear strength with depth, from Carswell et al (1991)

In May 1988, the TRRL installed eight spade cells equipped with piezometers (five in the retained soil and three in the soil at the front of the wall) and ten pneumatic piezometers (nine behind the wall and one in front of it). The location of the instrumentation is shown in Figure 2. The three inclinometers are marked as I1, I2 & I3; the eight spade cells as SC1 to SC8; the ten pneumatic piezometers as PP1 to PP10; and the eight vibrating wire load cells as A to H.

In 1991, Carswell et al reported that the majority of the vibrating wire load cells were still functioning and that readings could still be taken from the inclinometer tubes but the remainder of the earlier instrumentation was not functioning.

5 Results of site study

5.1 Pore water pressures

Figure 6 shows the variation with time of the pore water pressure some 19.25 m behind the face of the wall. Overall there has been little change in the pressure recorded by the piezometers. Figures 7 and 8 show the variation with time of the pore water pressures some 6.75 m and 1.1 m behind the face of the wall respectively, and Figure 9 shows the variation of pore water pressures 1.5 m in front of the wall. Although a number of the piezometers provide somewhat erratic measurements over some period or other, the pore water pressure recorded by most of the piezometers has remained largely unchanged relative to their pre-1991 levels.

Figure 10 shows the maximum and minimum recorded pore water pressures together with the summer, winter and yearly averages for PP5 and PP6. This figure indicates that although PP5 exhibits a greater range than PP6, there is no systematic seasonal change in the recorded pore water pressures for either piezometer. The mean readings for the piezometers are close to the (mean) hydrostatic line. Figure 11 illustrates the maximum, minimum and mean pore water pressures for the piezometers installed 6.75 m behind the wall. The mean values for PP1 indicate a level generally higher than the (mean) hydrostatic pressure whilst PP2, PP3 and PP4 show levels generally lower than hydrostatic. Figure 12 shows the maximum, minimum and mean pore water pressures for SC2, SC3, SC4, SC5, PP7, PP8 and PP9 installed 1.1 m behind the wall. Piezometer SC1, installed at a depth of 2.1 m, has remained dry throughout the monitoring period. Figure 13 shows the maximum, minimum and mean recorded pore water pressures for SC6, SC7, SC8 and PP10 installed 1.5 m in front of the wall. The piezometers indicate mean pore water pressures which are at least equal to the (mean) hydrostatic pressure.

5.2 Loads in anchorages

The eight anchorages, arranged in four rows of two, which support panel number 26 were each provided with a load cell. The cells have been monitored from construction in 1972, initially by the BRE and latterly by the TRL. Figure 14 shows the variation in the measured loads from the end of construction. Each load cell contains three vibrating wire strain gauges: the load is calculated as the mean of the three readings, or less if one or two of the gauges have failed. Some of the variation in the data may therefore be due to the loss of a gauge rather than to a change in the anchor load. Indeed, the most reliable data seem to be provided by anchorages D, E and F, the only ones with all three strain gauges functioning correctly. The variation in the measured lock-off loads in the various anchorages at the end of construction has not been discussed in any of the references that have come to hand.

The measured loads in the top row of anchorages (A and B) have remained reasonably constant from 1988 to 1992 at about 500 kN and 480 kN respectively but none of the vibrating wire gauges of the load cell on anchorage A have functioned since 1992 and the record for anchorage B is somewhat erratic perhaps because only one of the vibrating wire strain gauges has provided reliable data from 1999 onwards.

The measured loads on the second row of anchorages (C and D) increased by 30 to 35 kN between mid-1974 and 1990 with 5 kN of the increase occurring after mid-1988. Between 1988 and 1990 the load recorded on anchorage C was fairly constant at about 490 kN, but post-1990 the readings have become more erratic; however again only one of the vibrating wire gauges has provided reliable data since 1988. By contrast, all three vibrating wire strain gauges of the load cell on anchorage D seem to have provided reliable data from 1984. The recorded load in anchorage D appears to show a small but steady increase throughout the entire monitoring period.

The loads on the third row of anchorages (E and F) have also shown a small but steady increase over the monitoring period. The loads increased by 10 to 15 kN between 1974 and 1990. The load in anchorage E increased to about 500 kN by 2000, whilst the load in anchorage F increased to about 335 kN by 2000. All the vibrating wire strain gauges of the cells to these anchorages seem to have provided reliable data.

The data on the loads for the bottom row of anchorages (G and H) are limited. Only one of the vibrating wire strain gauges on anchorage G has provided any data from 1988 onwards and all the gauges on anchorage H had failed by the time monitoring recommenced in 1984. The measured load in anchorage G has reduced from about 400 kN in 1988 to about 360 kN by 2000.

Carswell et al (1991) noted apparent seasonal fluctuations of up to 20 kN on anchorage G. Close examination of the data in Figure 14 indicates that a number of the other anchorage loads also exhibited seasonal variations of up to about 5 kN in the period 1988 to 1990, with the greater load recorded in the summer months.

5.3 Total lateral stresses

Five spade cells (SC1, SC2, SC3, SC4 and SC5) were installed 1.1 m behind the wall at depths of 2.1, 3.9, 11.5, 9.3 and 6.4 m (respectively), and three spade cells (SC6, SC7 and SC8) were installed 1.5 m in front of the wall at depths of 1.8, 3.2 and 4.8 m. The variation in the pressures measured by these two sets of spade cells are shown in Figures 15 and 16 respectively. These plots are measured

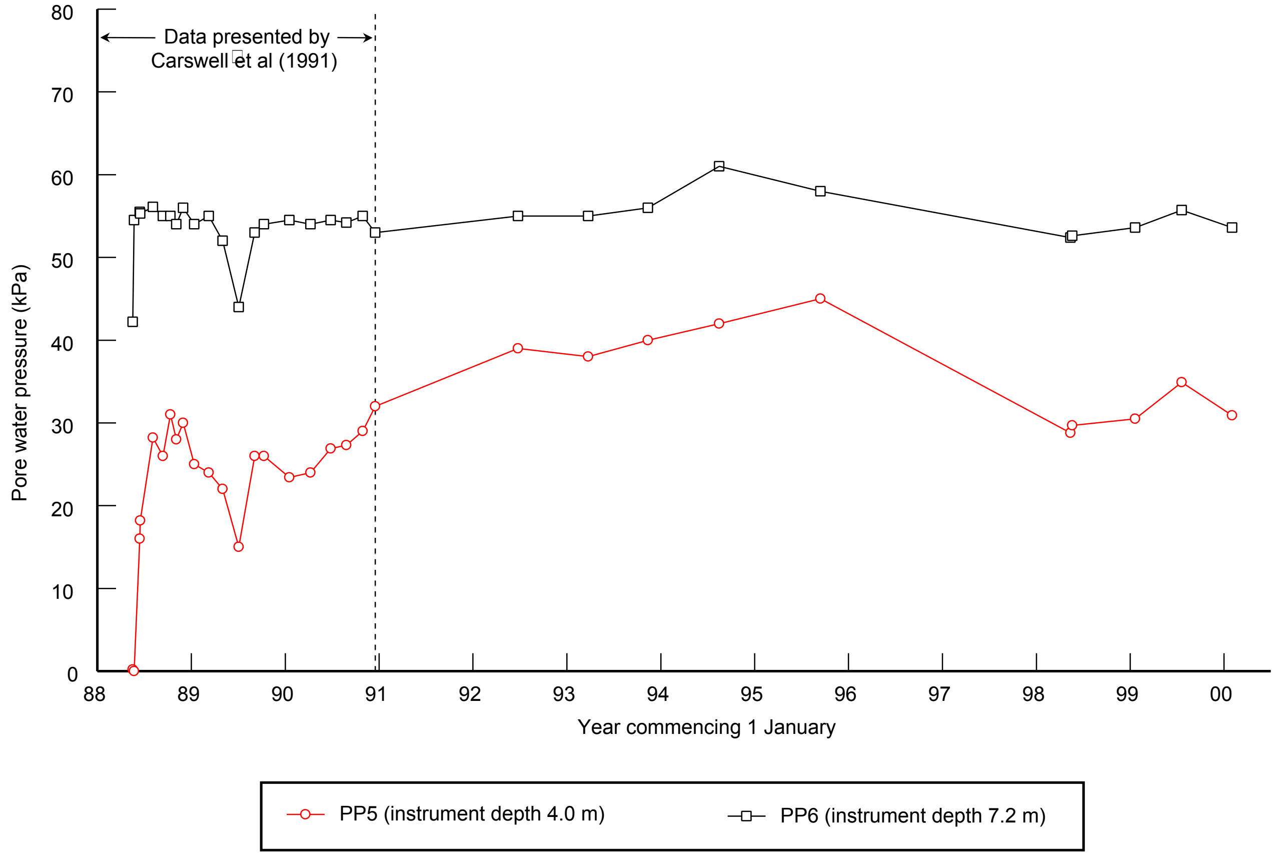


Figure 6 Variation of pore water pressure some 19.25 m behind the face of the wall

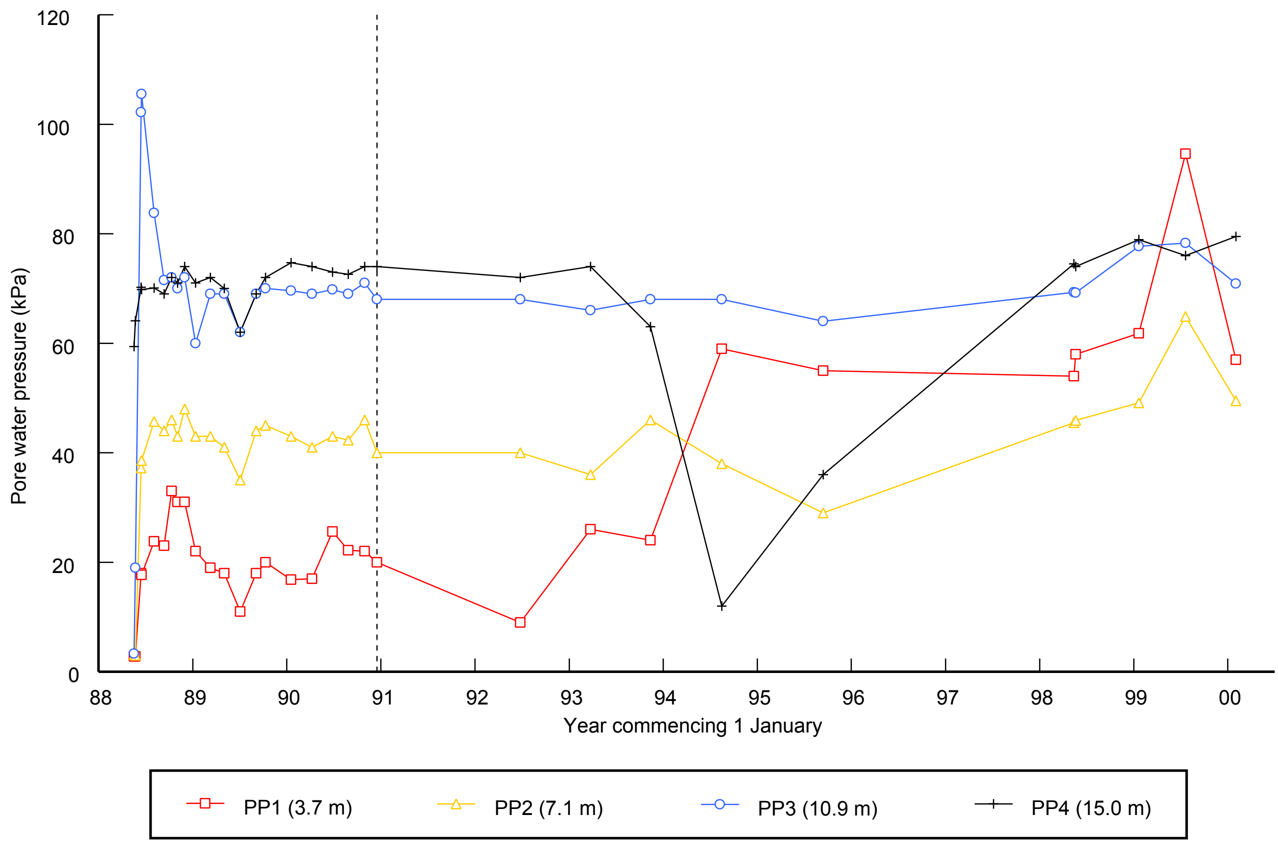


Figure 7 Variation of pore water pressure some 6.75 m behind the face of the wall

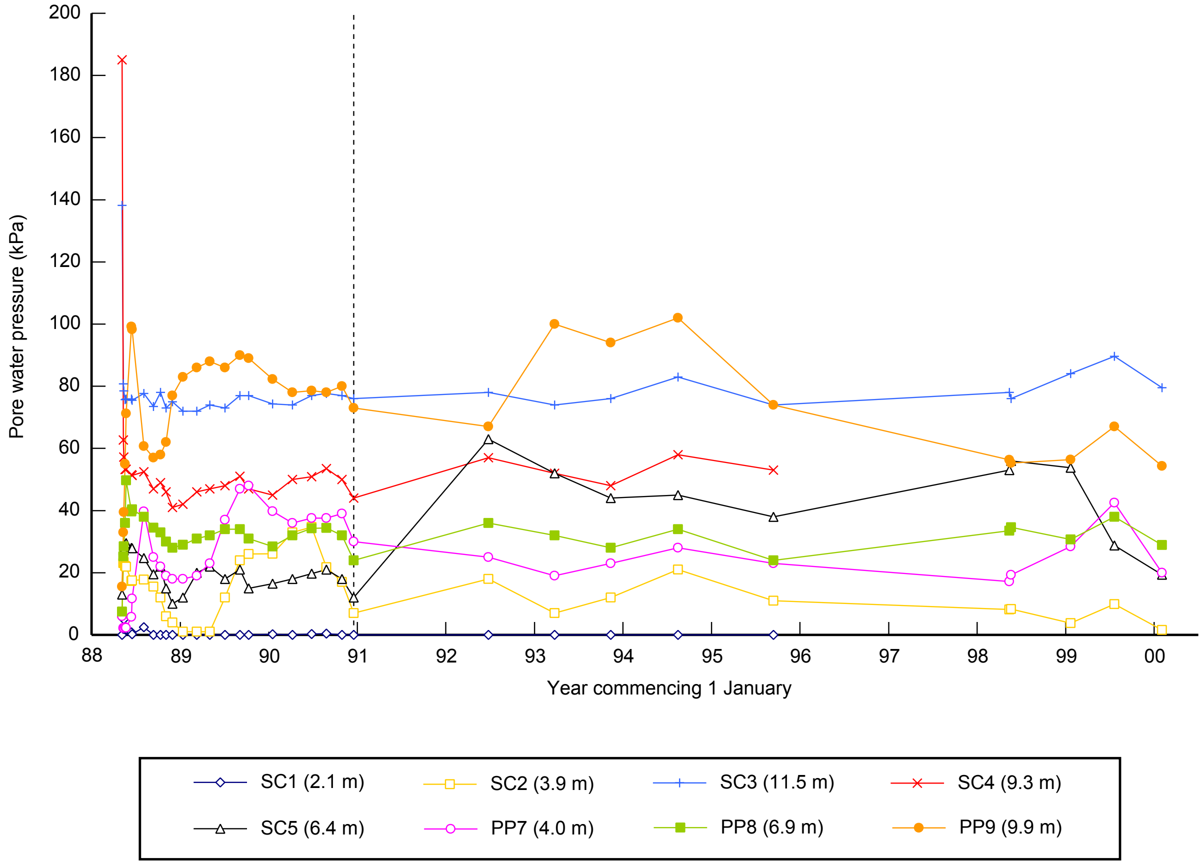


Figure 8 Variation of pore water pressure some 1.1 m behind the face of the wall

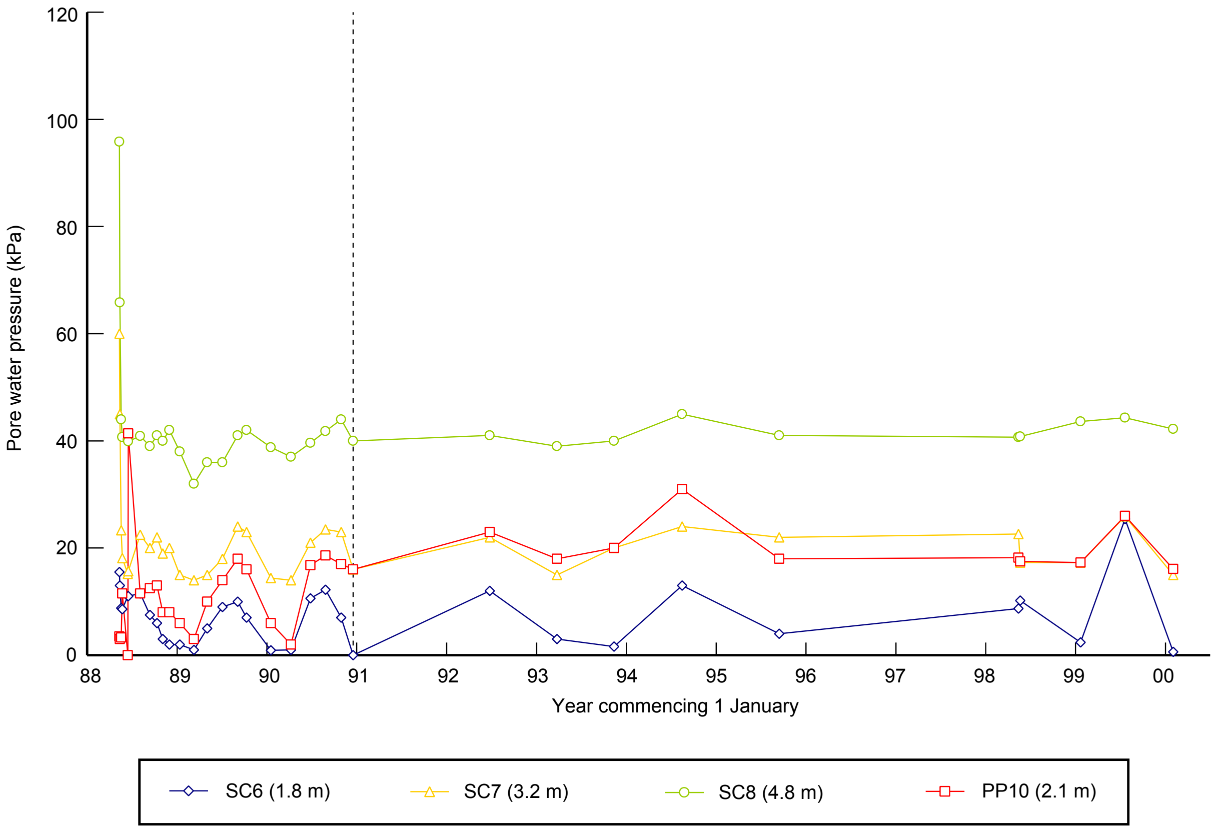


Figure 9 Variation of pore water pressure some 1.5 m in front of the wall

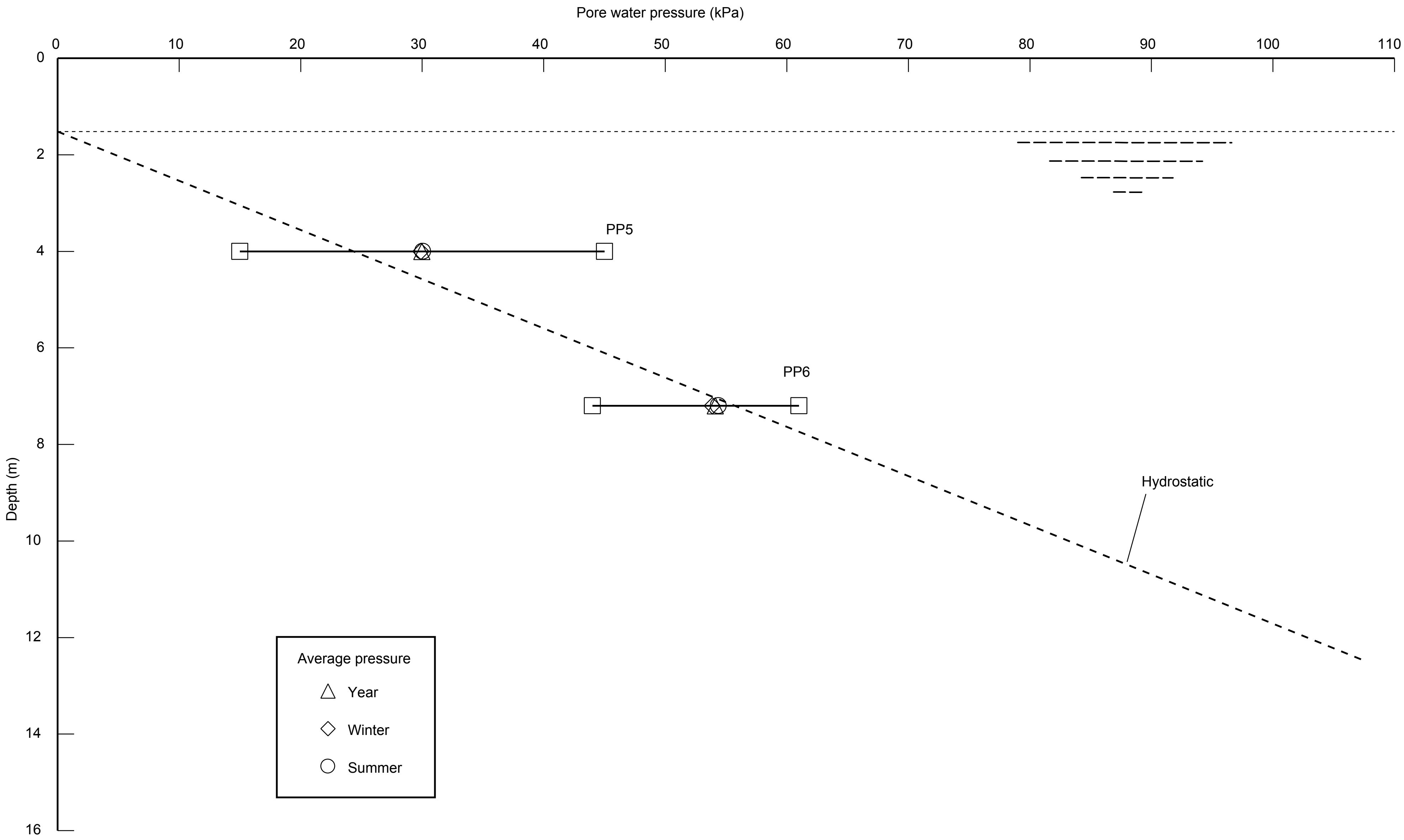


Figure 10 Distribution of pore water pressure some 19.25 m behind the face of the wall

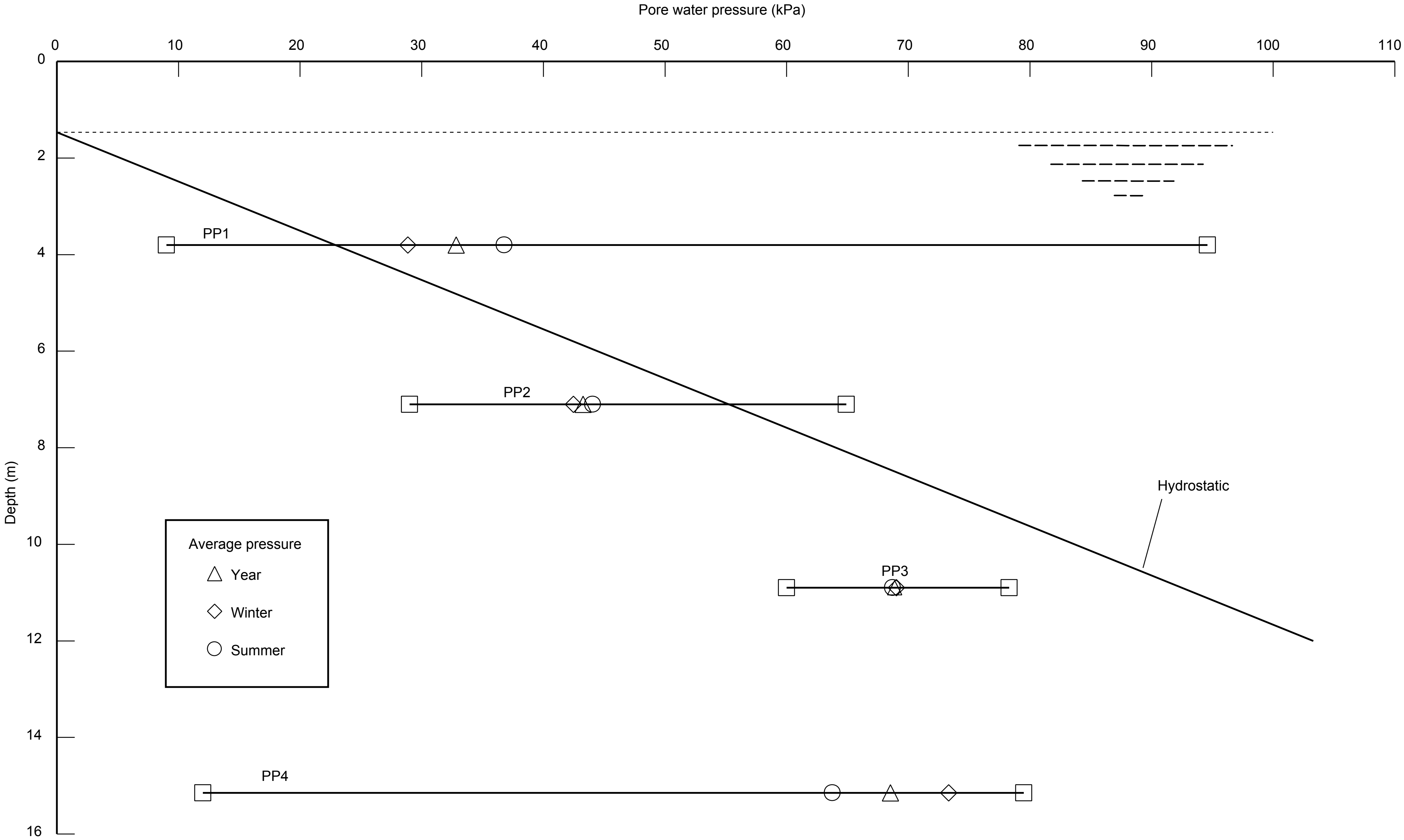


Figure 11 Distribution of pore water pressure some 6.75 m behind the face of the wall

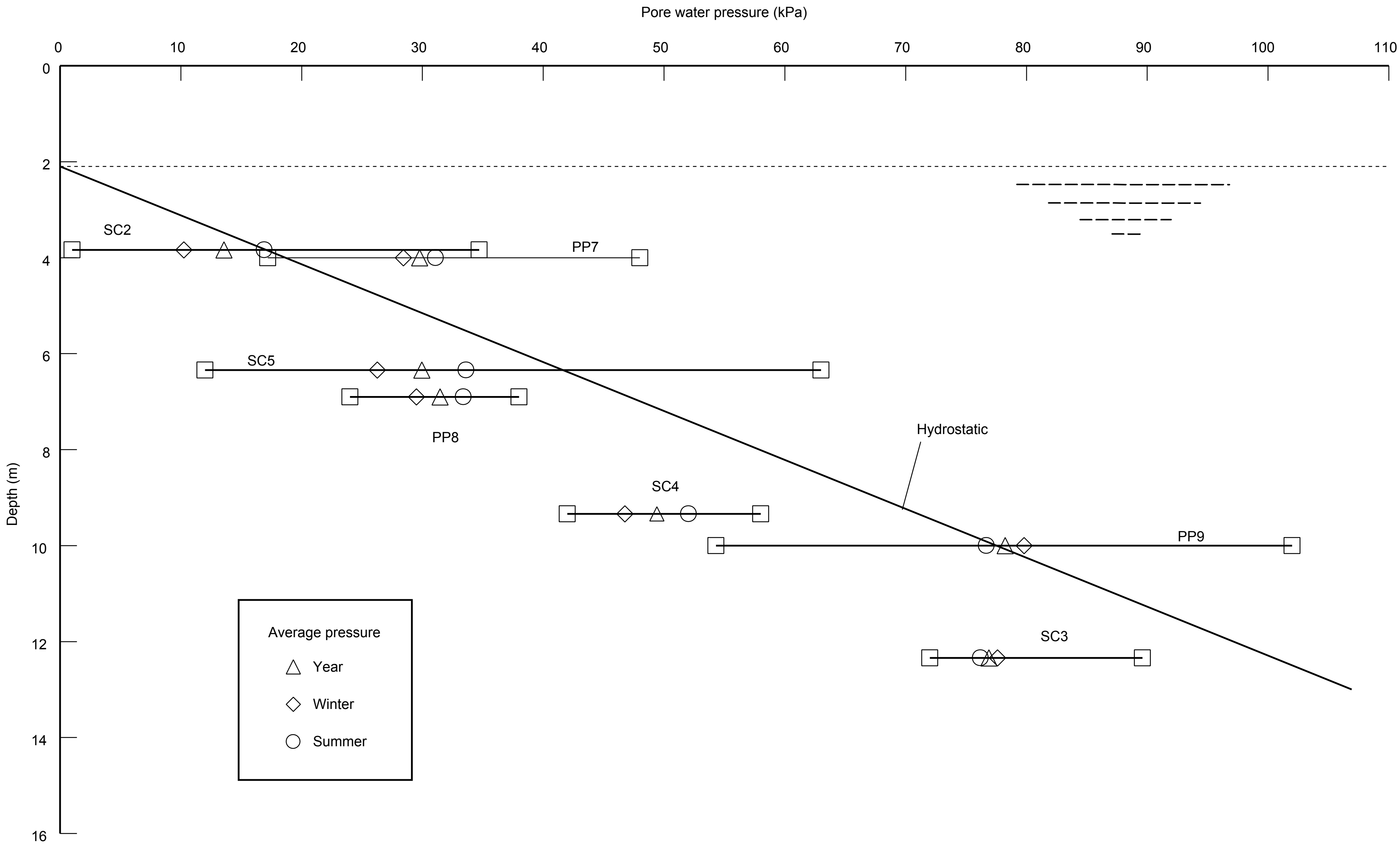


Figure 12 Distribution of pore water pressure some 1.1 m behind the face of the wall

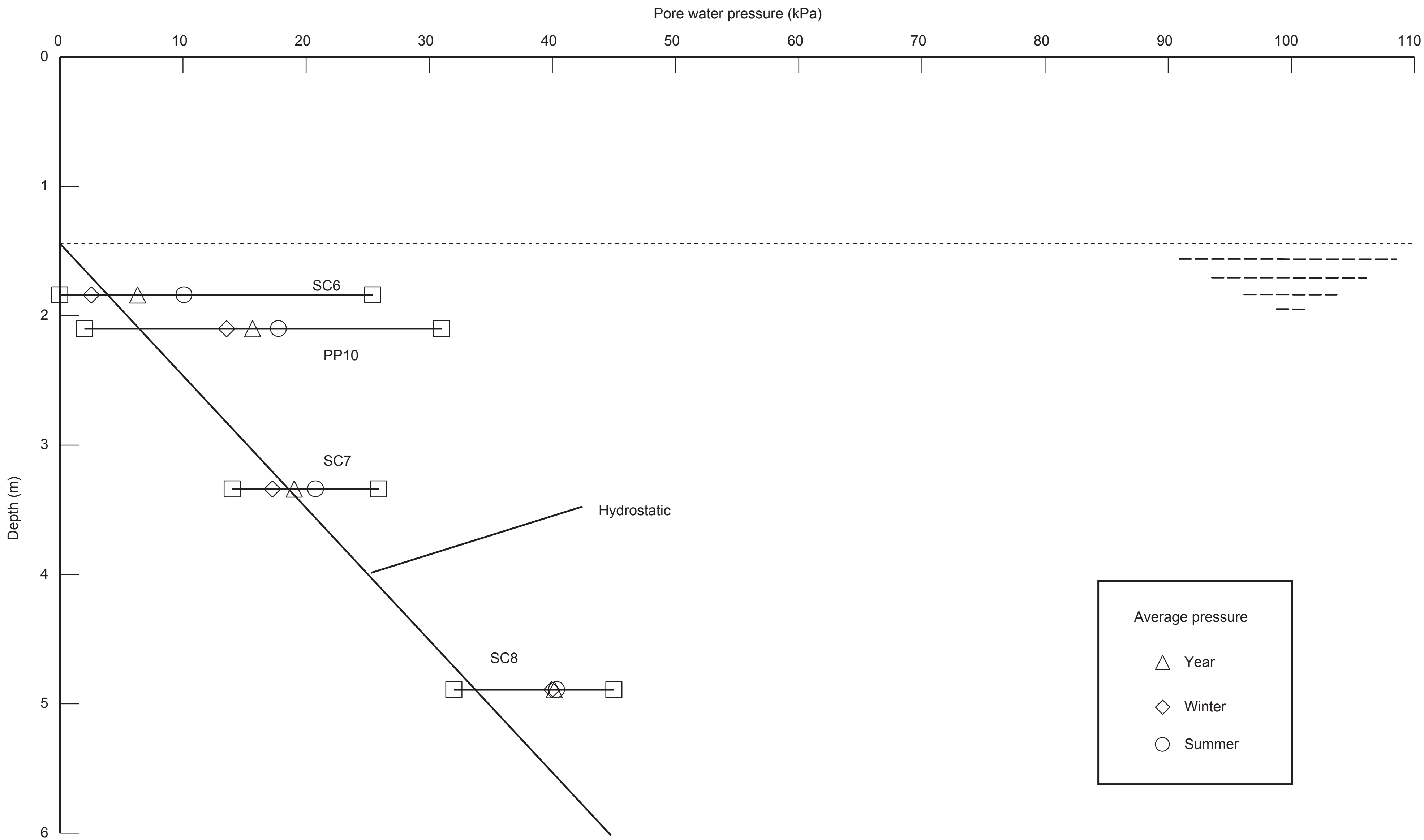


Figure 13 Distribution of pore water pressure some 1.5 m in front of the wall

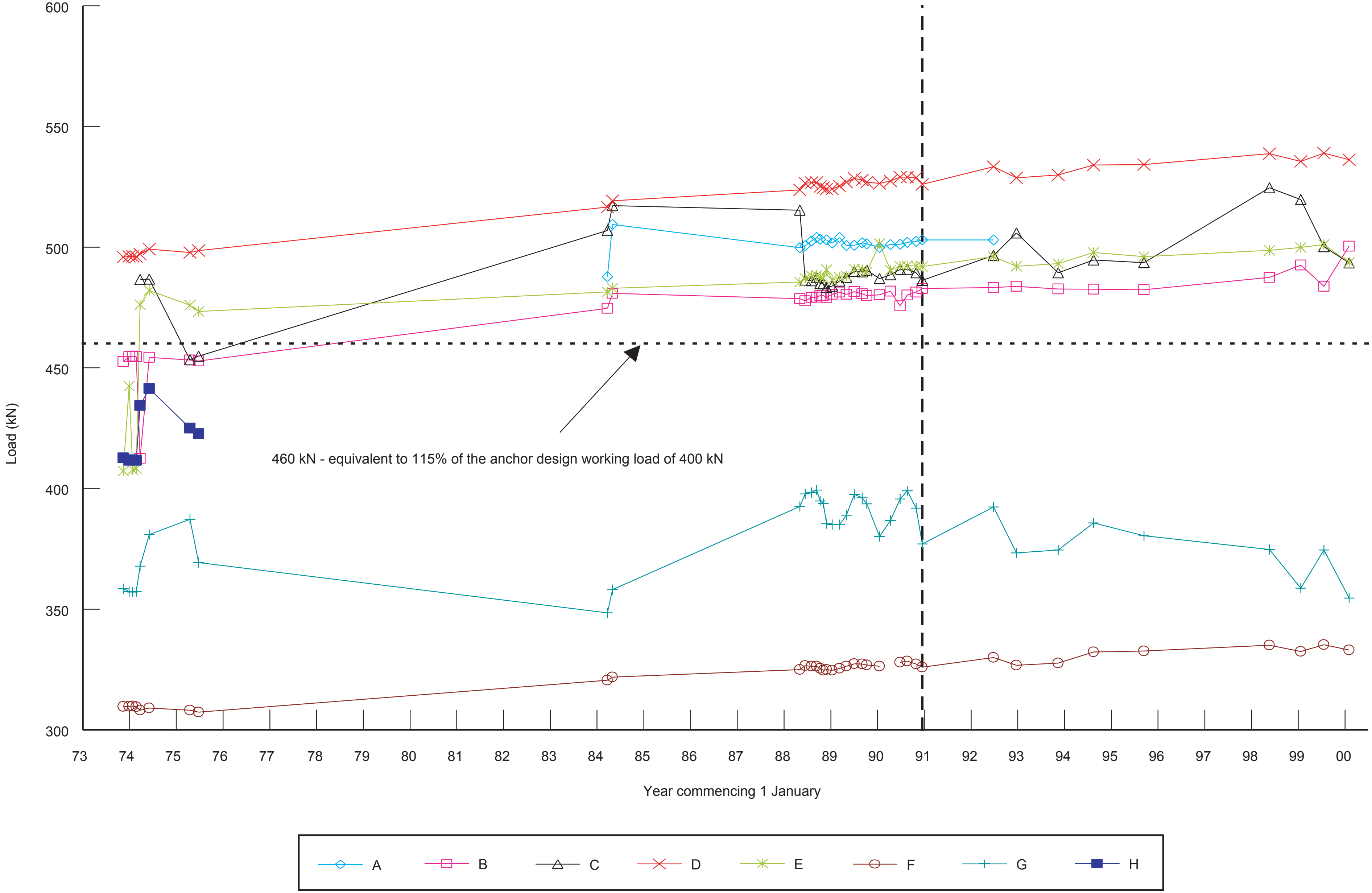


Figure 14 Variation of measured loads in ground anchorages

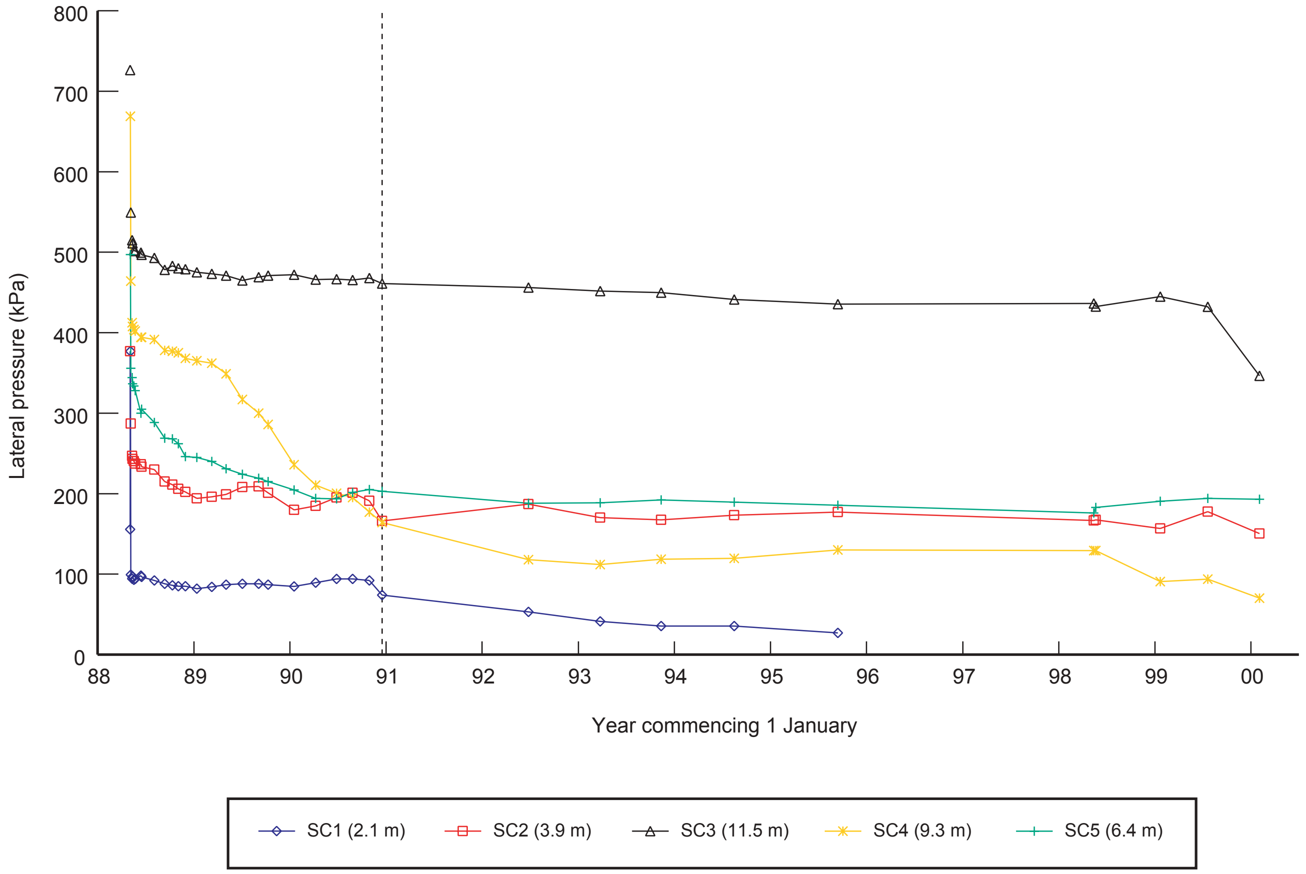


Figure 15 Variation of space cell pressures some 1.1 m behind the face of the wall

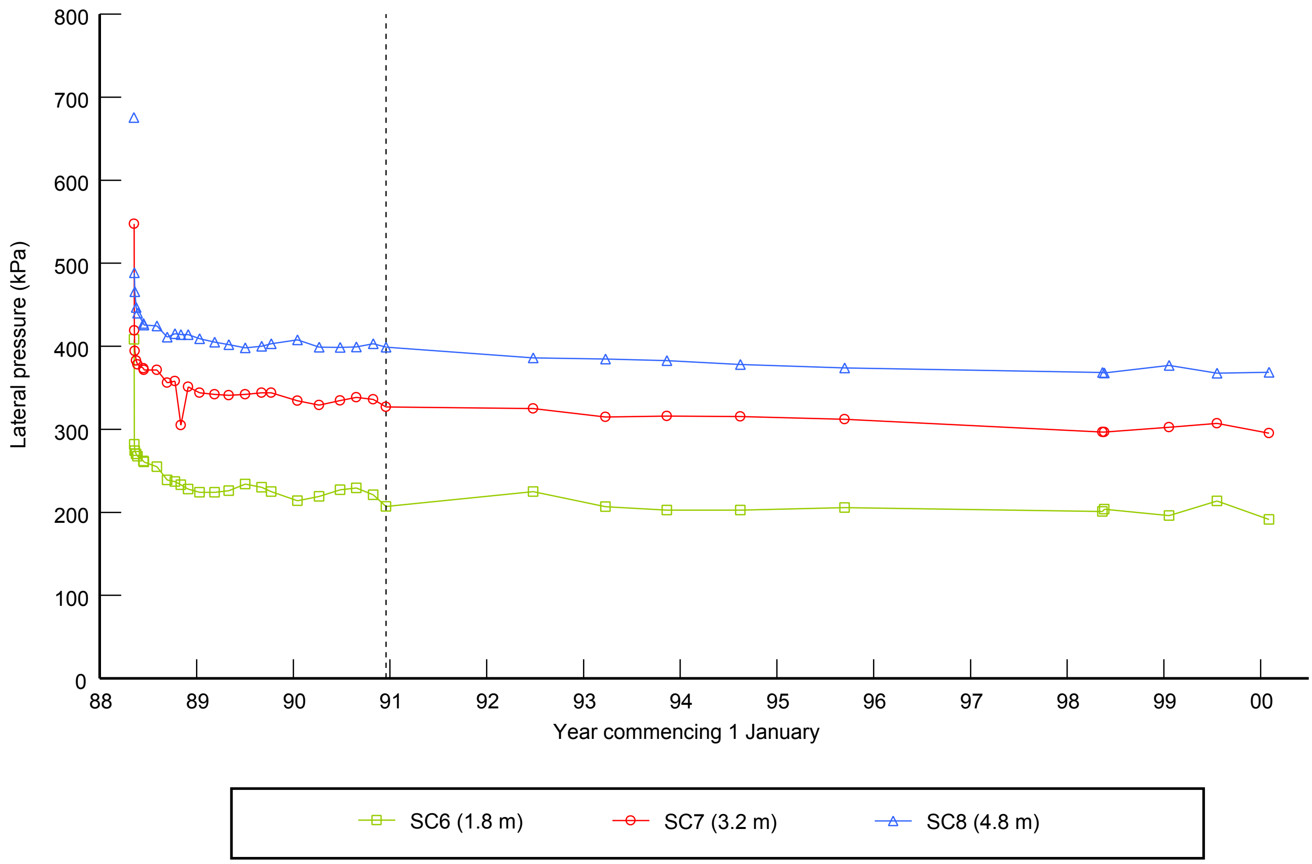


Figure 16 Variation of space cell pressures some 1.5 m in front of the wall

pressures, i.e. no correction has been applied for the registration error associated with the method of installation.

Apart from the installation phase and a small seasonal variation, the pressures in three of the five cells installed behind the wall stabilised within a few months. However SC4 showed a continuing downward trend (which was attributed to a fault in the instrument) and SC5 displayed a similar prolonged, but not as pronounced, trend. The data in Figure 15 shows that since 1991 all the pressures recorded by the spade cells have been reasonably constant, although the recorded behaviour post-1998 does appear more erratic. Figure 16 shows that all three of the spade cells in front of the wall indicate a gradual downward trend.

5.4 Wall and ground movements

Three inclinometers were installed to a depth of 13 m: these were located in the wall (I1) and at 4 m (I2) and 19 m (I3) behind the wall. The changes in deflected shape (displacement profile), calculated assuming base fixity of the inclinometer tubes, since their installation in early 1972 (i.e. prior to excavation) are shown in Figures 17 for (I1), 18 (I2) and 19 (I3).

Sills et al (1977) reported a base translation of each of the tubes of about 20 mm towards the excavation 10 months after completion of excavation (mid-1973). However, as no measurements of absolute movement were possible since that time due to loss of the reference stations, only the relative deflected shapes of the inclinometer tubes can be provided.

The responses of inclinometers I1 and I2 are quite similar, indicating a total relative movement between the top and bottom of the tubes of about 52 mm by 1990, with 14 to 18 mm of this movement having occurred since 1973. Inclinometer I3 indicated little or no relative movement between 1976 and 1990, but about 5 mm of movement seems to have occurred between 1973 and 1976. Between October 1990 and February 2000, inclinometers I1 and I2 indicated outward relative movements of about 5 mm and 1.5 mm respectively and I3 a maximum relative movement of about 3 mm (at a depth of 4 m).

5.5 Discussion

5.5.1 Pore water pressure

The pore water pressure regime does not appear to have altered much from that reported by Carswell et al (1991). Figure 20 provides values of r_u based on the computed yearly average pore water pressures and (for convenience) a bulk unit weight for the soil of 20 kN/m^3. Figure 20 also shows the line of a 100 mm diameter sandwick drain: these have been constructed at 1.15 m centres behind the face of the wall and connect through the wall to a drain under the carriageway as indicated. The lower pore water pressures measured immediately behind the wall are consistent with the presence of these drains. The pore water pressures in front of the wall appear to have remained reasonably constant over the monitoring period and are generally greater than, or at least equal, to the hydrostatic pressure.

Most of the piezometers at 6.75 m and 1.1 m behind the wall, and 1.5 m in front of the wall, indicate a seasonal fluctuation in pressure. Carswell et al (1991) also noted a seasonal variation in the piezometers installed 19.25 m behind the wall, but there is no evidence to indicate that this is the case now although seasonal fluctuation could be concealed by the relative infrequency with which the data have been acquired.

5.5.2 Lateral earth pressure

The measured lateral pressures behind the wall have remained fairly constant over the monitoring period, but those at the front of the wall indicate a small but steady downward trend. This may indicate that stresses generated as a result of the installation process are still dissipating some 12 years following the installation of the instruments. Figure 21 shows the variation in the coefficient of lateral earth pressure (K) calculated from the measured lateral stresses and pore water pressures, and an assumed bulk unit weight of 20 kN/m^3 for the soil. Corrections to the spade cell readings have been made by subtracting one half of the undrained shear strength (c_u) as determined from a series of triaxial tests, the results of which are shown in Figure 3c (after Carswell et al, 1991). Some correction is necessary because installation of the push-in type of spade cell results in an over-registration error which, as suggested by Tedd and Charles (1983), is about 0.5 c_u. The calculated magnitudes of K are typically between 1.0 and 2.0 behind the wall, and between 4.0 and 5.0 in front of it. However, as shown in Figure 21, the values of K vary considerably (particularly towards the surface) – this is mainly due to the variation in the recorded pore water pressure. The current values of K are similar, but lower, than those determined by Carswell et al (1991) who calculated values of K of about 2.0 behind the wall and 6.0 in front of the wall using this method. The spade cells also indicate the presence of small seasonal variations in the recorded pressures.

5.5.2 Inclinometer data

The data from the inclinometers indicate that movement of the wall and retained ground is still occurring some 28 years after construction. The maximum recorded movement between 1990 and 2000 is only about 5 mm, but 3 mm of movement was recorded by inclinometer I3, some 19 m behind the face of the wall.

5.5.4 Anchorage loads

Three of the anchorages (D, E & F) are instrumented with load cells in which all three vibrating wire strain gauges seem to be still functioning. As shown in Figure 14, it is evident that the loads in these anchorages have been slowly but steadily increasing over the monitoring period. Of the eight anchorages supporting the test panel, all of which have a design working load of 400 kN, five are supporting loads in excess of 460 kN (115% of the design working load) and have done so since 1984. Anchorage D is currently supporting about 535 kN which equates to 134% of the design working load: thus it would seem that the factor of safety against rupture of the tendon is about 1.73 compared to the initial design value of about 2. The anchorages also exhibit small seasonal fluctuations.

Figure 22 plots both the load in anchorage D and the

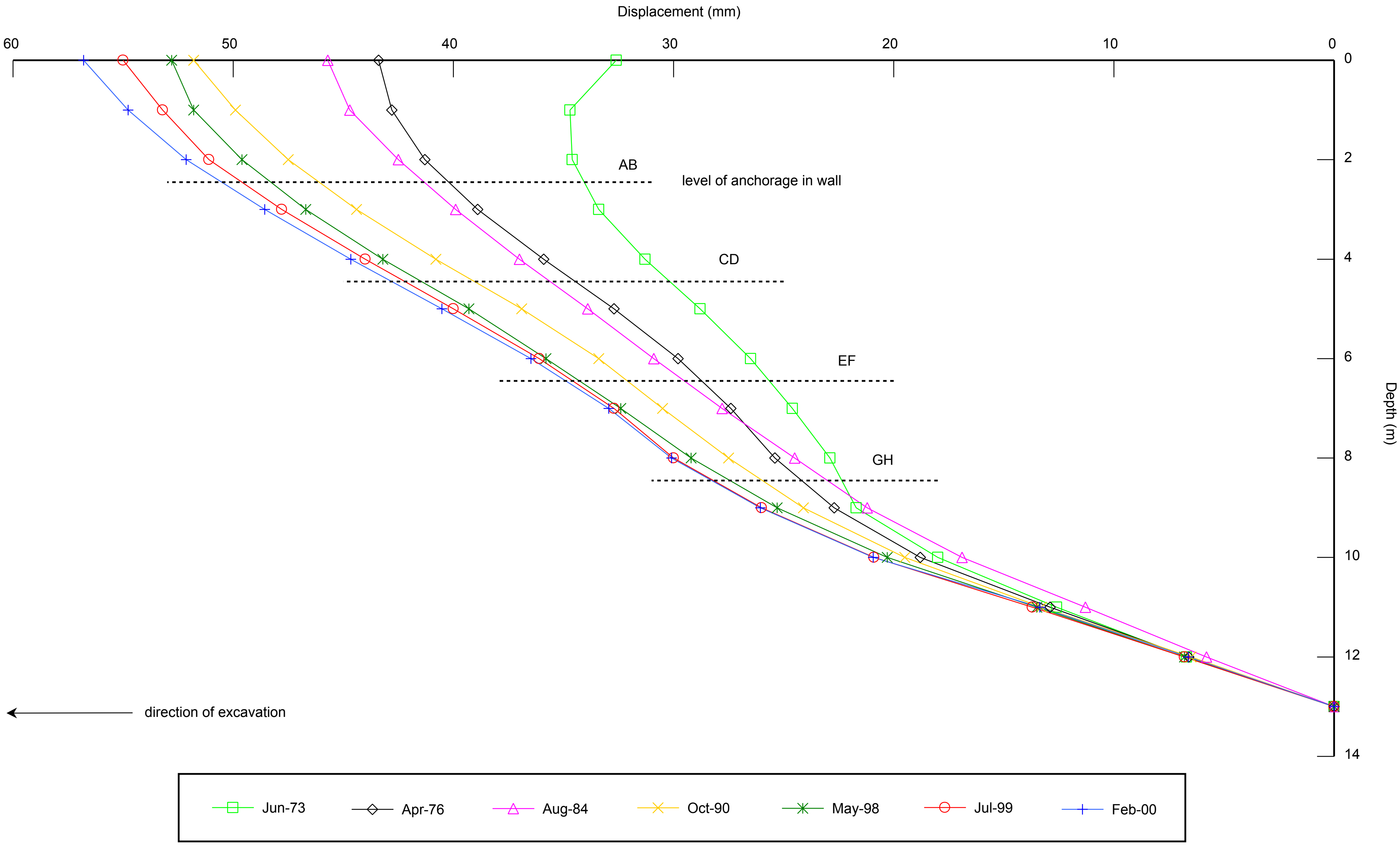


Figure 17 Subsurface lateral movements recorded by inclinometer I1 in the wall

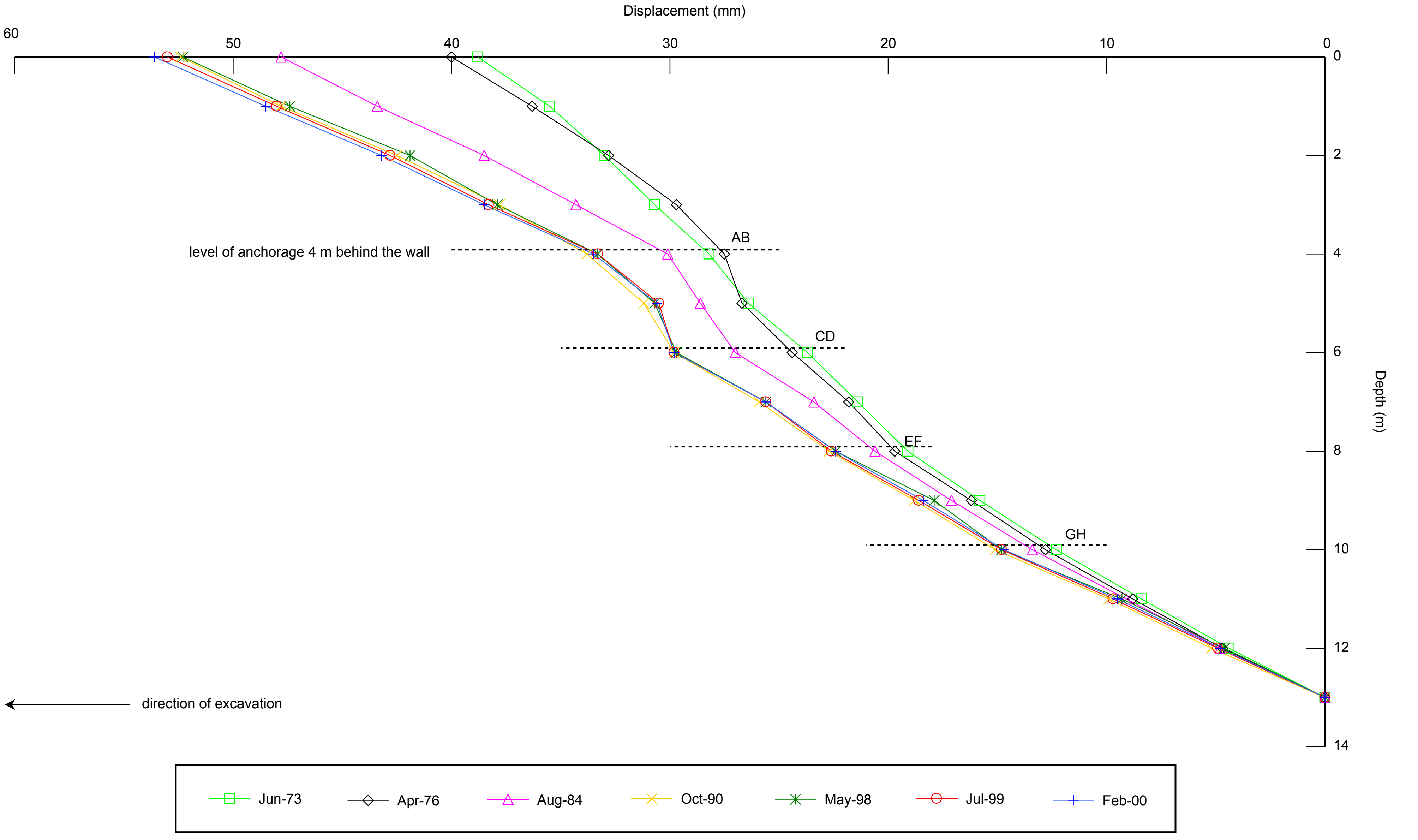


Figure 18 Subsurface lateral movements recorded by inclinometer I2 some 4 m behind the face of the wall

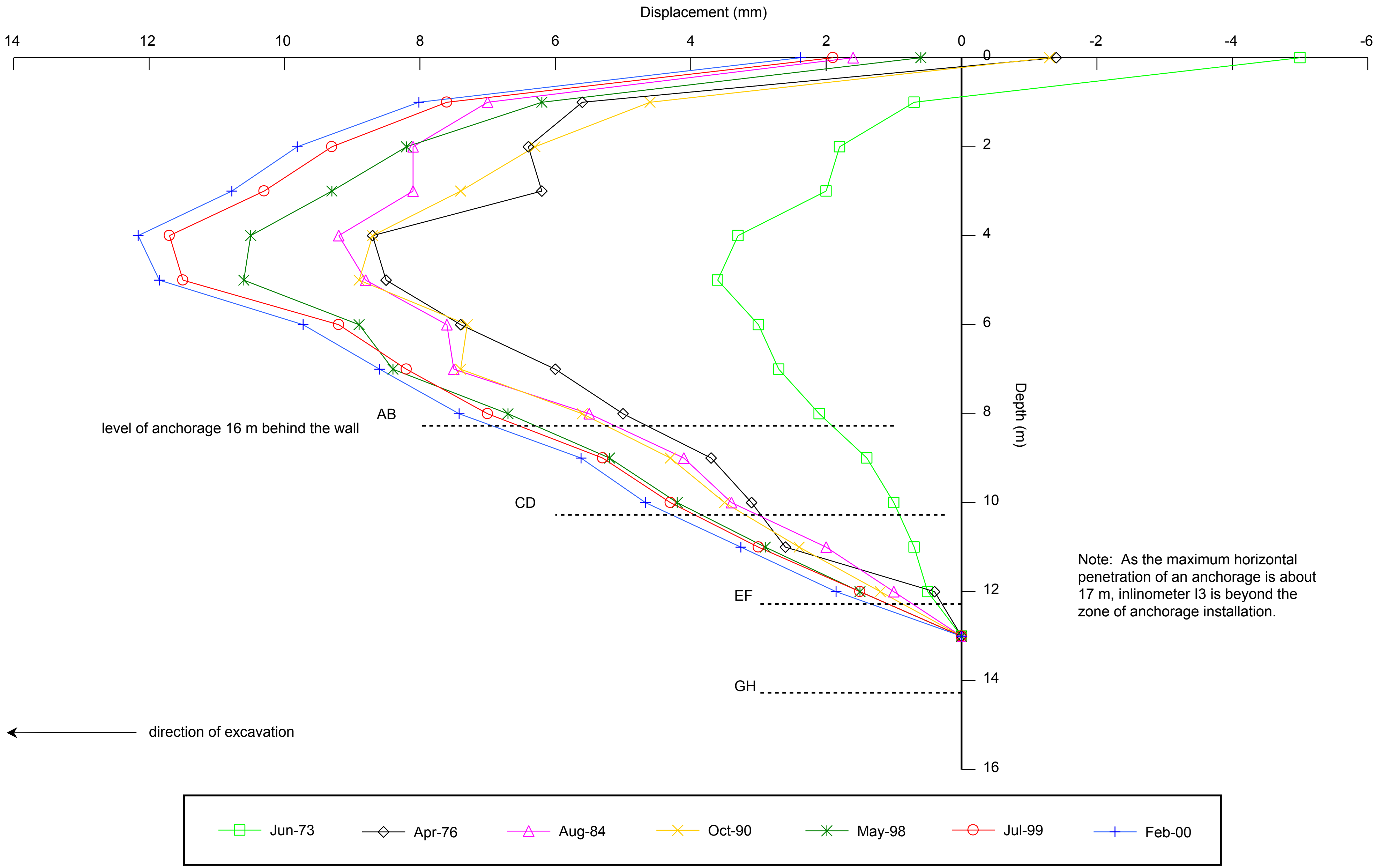


Figure 19 Subsurface lateral movements recorded by inclinometer I3 some 19 m behind the face of the wall

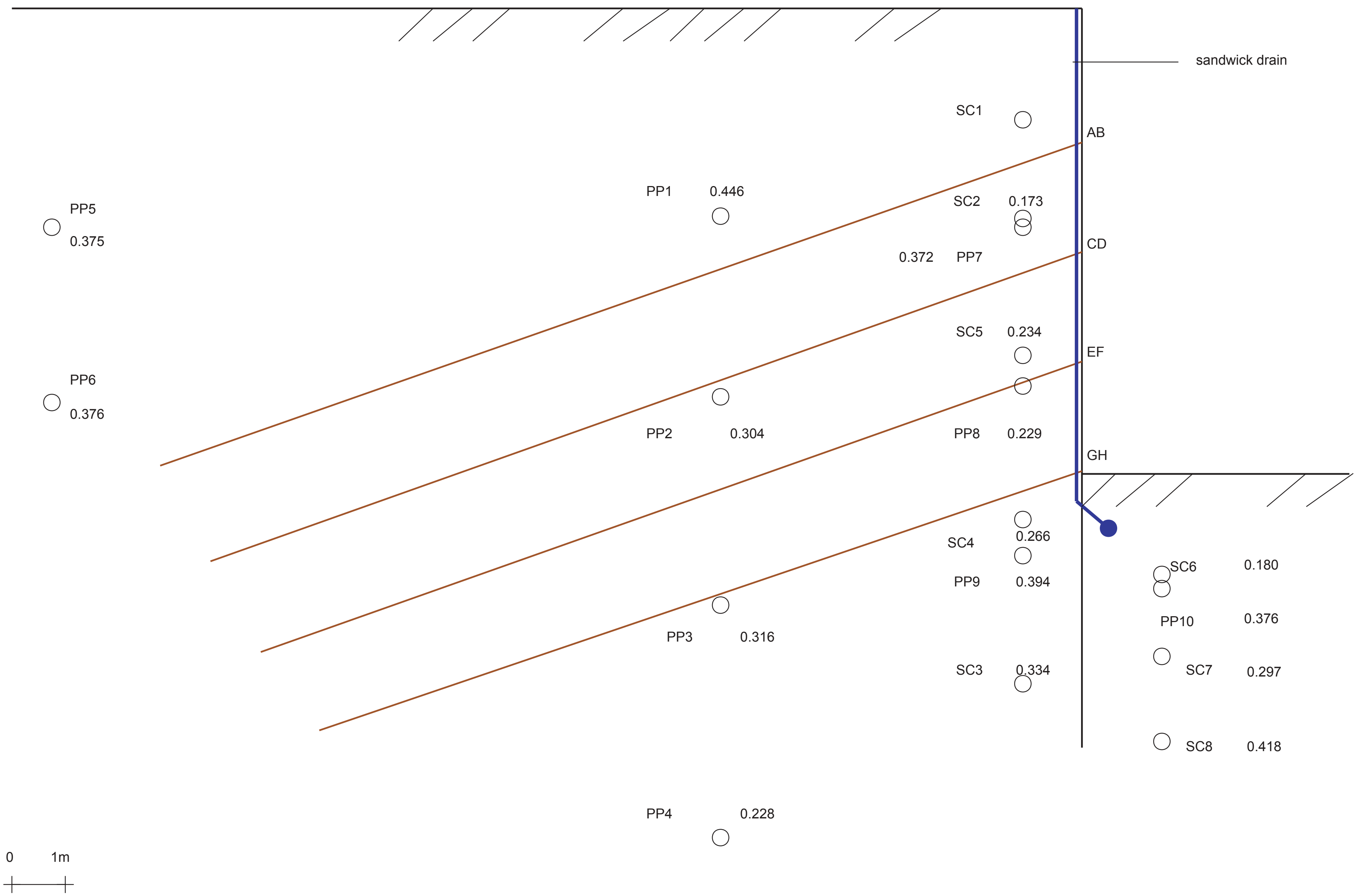


Figure 20 Neasden retaining wall - r_u values based on yearly pwp average

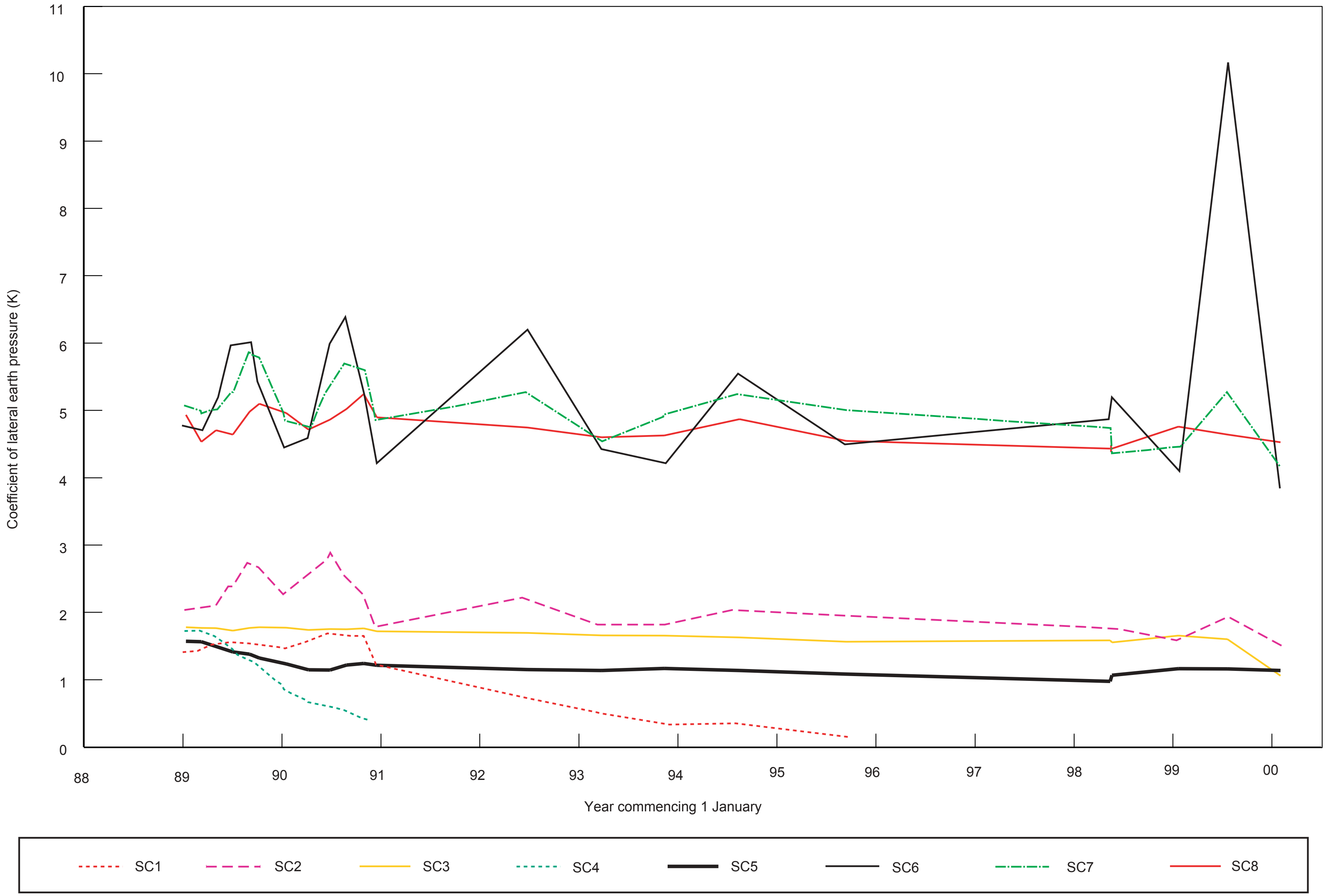


Figure 21 Variation of coefficient of lateral earth pressure (K) with time

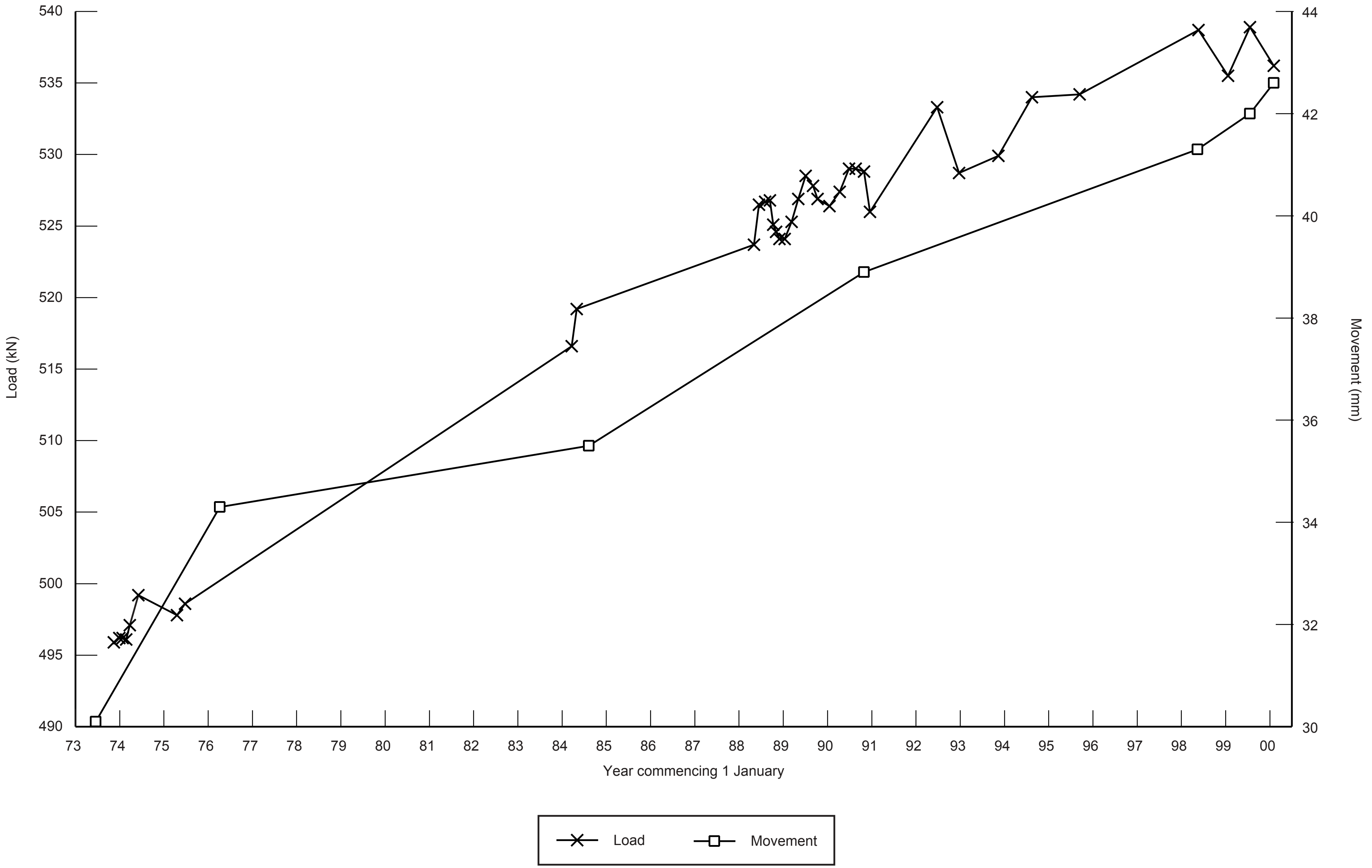


Figure 22 Anchorage D — variation of load and wall deflection with time

movement of the wall (at that depth) as determined from inclinometer I1 as a function of time. The figure shows that a change in load of about 40 kN has been accompanied by a relative movement of the top of the inclinometer tube to the bottom of about 12.5 mm. By assuming a value for Young's Modulus (E) of the steel tendon, it is possible to calculate the extension due to the increase in load. A typical value for E for a bar or rod of steel is about 210 kN/mm^2. However, according to Appendix H of BS 8081:1989, the computed value of E can vary and for a 15 mm diameter strand it quotes values for E_s (of the individual strand) of 183 to 195 kN/mm^2 compared to values for E_t (for the tendon) of 171 to 179 kN/mm^2. For the problem at hand, such a difference has little effect on the computed extensions. Assuming a value for Young's Modulus for the tendon of 200 kN/mm^2, the computed extension of the tendon at this site falls between about 2 and 5 mm (depending on the assumed distribution of load along the anchorage – which has a much greater effect on the calculated extension than the variation in E). Thus there is an apparent discrepancy of about 7 to 10 mm. Similar calculations for anchorages E and F reveals discrepancies of between about 3 and 6 mm. If the base of the wall and inclinometer tube have remained fixed over the period, this suggests that the additional movement has occurred due to creep of the anchorage. However there is no means of checking this assumption and it is possible that, for example, the relative movement has been generated as a result of backwards rotation of the base of the wall.

5.5.5 Summary

The results of the instrumentation indicate that the pore water pressure regime appears to have reached an equilibrium. Recorded lateral pressures behind the wall also appear to have equilibrated but those in front of the wall seem to be still reducing. Although a number of the load cells have ceased to function, where seemingly reliable data have been acquired they suggest that the loads in the anchorages are, some 28 years after construction, still increasing. Lateral movements recorded by the inclinometers also indicate that the wall and retained ground are still deforming. A number of the instruments indicate the presence of seasonal variations. Infrequent monitoring of systems subject to seasonal variations can make the identification of long-term trends difficult and it would be beneficial to increase the monitoring frequency at Neasden (as was done between 1988 and 1990) to identify any longer-term changes that may be occurring.

The results of a simple analysis suggests that the anchorages have pulled-out of the retained ground by about 5 to 10 mm over the past 25 years or so: the rate of pull-out seems to be near-linear. However it was assumed in this analysis that the toe of the wall was fixed and this may not have been the case. Such movements, and the increase in pull-out force should not give cause for concern. Such changes are consistent with the softening of the soil on the passive side of the wall. However, it would be worthwhile considering the installation of additional instrumentation to (a) better establish the pore water pressure regime and (b) determine the fixity of the toe of the inclinometers. Such instrumentation could be done at relatively little cost. The additional data would help define the pull-out of the anchorages which should in turn provide confidence in the use of anchorages in clayey soils.

It is uncertain whether additional expenditure would be justified on undertaking numerical analyses of the performance of the structure: indeed such analyses would probably not be particularly useful without additional site data.

6 Conclusion

The anchored diaphragm wall at Neasden Lane Underpass was constructed in 1972. During construction, instruments were installed including vibrating wire strain gauge load cells on the anchorages and three inclinometers. In 1988, spade cells and piezometers were installed by the (then) TRRL. The following conclusions can be drawn from the data:

1. The pore water pressure regime does not appear to have altered since last reported on by Carswell et al (1991) and seems to have attained a state of equilibrium. The regime exhibits small seasonal fluctuations.
2. The lateral stresses measured by the spade cells installed in front of the wall have gradually but steadily reduced since their installation in 1988. This reduction may be due to the relief of stresses generated by the installation process.
3. The load cells installed on three of the anchorages seem to have provided reliable data since their installation in 1972. Data from these indicate that the loads on the anchorages have gradually but steadily increased since 1973 and that the factor of safety has reduced from an apparent design value of about 2.0 to a worst case of about 1.73 for an individual anchorage.
4. No measurements of absolute wall or ground movements are available, only the relative movements of the top of the inclinometer tube to the bottom can be reported. However, the three inclinometers indicate that movement of the wall and retained ground is still occurring some 28 years after construction and as far as 19 m behind the face of the wall. The maximum recorded movement over the past ten years or so has been about 5 mm.

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Abstract

In 1972, as part of the upgrading of the North Circular Road (A406) in Neasden, an anchored diaphragm wall was constructed in heavily over-consolidated London Clay and instrumentation was installed to monitor the performance of the wall. In 1988, additional instrumentation was installed to better investigate the longer term performance of the wall. The results of the study have been reported by Carswell et al (1991).

Although a number of the instruments have failed in-service, the remaining operating instruments include load cells to measure the loads in the ground anchorages, spade cells to measure lateral stresses in the ground, inclinometers to monitor ground movements, and piezometers to measure pore water pressures. Following the work of Carswell et al (1991), the instruments have been read sparingly throughout the 1990s but more regular monitoring recommenced in 1998. This report provides an update of the performance of the anchored wall.

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