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Publisher’s version / Version de l'éditeur:

Canadian Geotechnical Journal, 9, 2, pp. 206-218, 1972-10-01

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THE PERFORMANCE OF TIED-BACK SHEET PILING IN CLAY

BY

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Reprinted from
CANADIAN GEOTECHNICAL JOURNAL
Vol. 9, No. 2, May 1972
p. 206–218

RESEARCH PAPER NO. 511
OF THE
DIVISION OF BUILDING RESEARCH

OTTAWA
October 1972

Price 25 cents

NRCC 12368

LE COMPORTEMENT DES PALPLANCHES HAUBANEES ENFONCEES DANS L'ARGILE

PAR

G. C. McROSTIE, K. N. BURN ET T. J. MITCHELL

SOMMAIRE

En 1969, dans un chantier d'Ottawa, on a érigé un mur en palplanches haubanées afin de fournir un appui temporaire sur l'un des côtés d'une excavation d'une profondeur de 12 mètres, traversant des dépôts de la Mer de Champlain et atteignant un tuf schisteux. Le mur fut conçu de façon à minimiser le fléchissement afin de protéger un important poste de transformateurs adjacent.

Afin d'évaluer le comportement de ce mur, on a mesuré, au cours de l'excavation, les mouvements verticaux de la surface adjacente au mur, les déplacements horizontaux du mur, les charges sur les haubans et les pressions de la nappe aquifère.

Une série d'essais triaxiaux a effectuée en laboratoire afin de déterminer la forme et l'ampleur des déformations du sol sous l'effet des changements de contrainte semblables à ceux qu'on a mesurés sur place. On obtient une correspondance convenable lorsqu'on utilise les résultats de ces essais pour évaluer les déplacements du sol sur place. Enfin, les auteurs comparent les charges sur les haubans déterminées sur place à celles que fourniraient les méthodes courantes de calcul.

The Performance of Tied-Back Sheet Piling in Clay[1,2]

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Received August 3, 1971

Accepted December 17, 1971

In Ottawa in 1969 a tied-back sheet pile wall was installed to provide temporary support in one side of a 12 m deep excavation through Champlain Sea deposits to shale bedrock. The wall was designed to permit as little yield as possible in order to safeguard the vital operation of an adjacent transformer building.

To assess the performance of this structure, measurements of vertical movements of the surface adjacent to the wall, horizontal displacements of the wall, tendon loads and ground-water pressures were made as the excavation progressed.

A series of triaxial tests was carried out in the laboratory to determine the form and magnitude of soil deformations under stress changes approximating those derived from the field measurements. Reasonable correlation is obtained when the results of these tests are used to estimate soil displacements in the field situation. The measured tendon loads are compared with those that would be expected using current design methods.

Introduction

The use of tieback systems to replace external bracing for temporary and permanent earth-retaining structures has greatly increased during the past 5 years. The prediction of the movements within the soil mass adjacent to the retaining structure is often regarded as the most difficult problem associated with these systems. It therefore seems worthwhile to present case histories where movements have been measured in some detail, where comparisons can be made between observations and the predictions of current design methods, and where an attempt to predict the movements has been made.

The opportunity for such a case study arose in Ottawa, Canada, when excavation for an office building to a depth of 40 ft (12 m) was begun close to an electric utility transformer building. The building was only 15 ft (4.6 m) from the excavation, and was founded on spread footings on clay at a depth of about 7 ft (2.1 m). In addition, nine large cooling units, rigidly connected to the building, were only 5 ft (1.5 m) from the excavation. Since damage to the coolers or transformers would have halted electric power service to a large section of downtown Ottawa, including the Houses of Parliament of Canada which were in session at the time, special measures were required.

Tieback System Design and Sequence of Construction

The tieback system was designed by the engineering staff of the specialized piling subcontractor engaged to provide satisfactory support for the sides of the excavation. A system using steel sheet piling with two levels of wales to distribute the load in tieback tendons placed at 45° under the adjacent property and into the underlying shale rock was adopted. The design assumed fully drained conditions during construction, a trapezoidal distribution of earth pressures plus surface surcharge allowance, and an at-rest lateral soil pressure coefficient of 0.6. It was intended to approach

[1]Paper presented at the 24th Canadian Geotechnical Conference, Halifax, Nova Scotia, September, 1971.
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the values of the lateral soil pressure at rest by prestressing the tendons to the required loads so that the sheet piling would not move toward the excavation. The resulting system was about twice as strong as would have been needed merely to provide normal retention of the sides of the excavation in the active pressure situation.

Figure 1 shows a plan of the site; Fig. 2, a typical section of the sheet pile wall and tieback system as well as the instrumented section of the sheet piling. A photograph of the site after the excavation was completed is presented as Fig. 3. The elevation of wales varied across the west wall to facilitate construction of ramps leading to the parking floors.

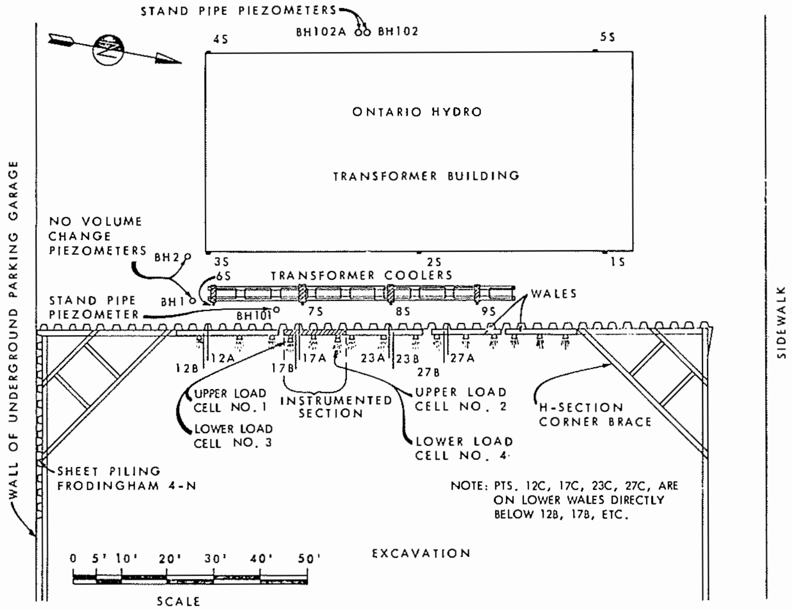

FIG. 1. Plan view of tied-back sheet pile wall and adjacent transformer building.

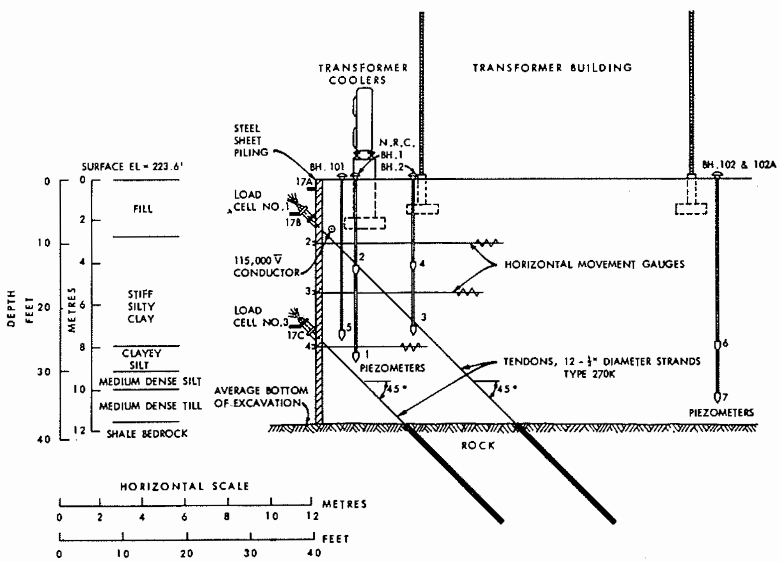

FIG. 2. Section through wall showing instrumentation.

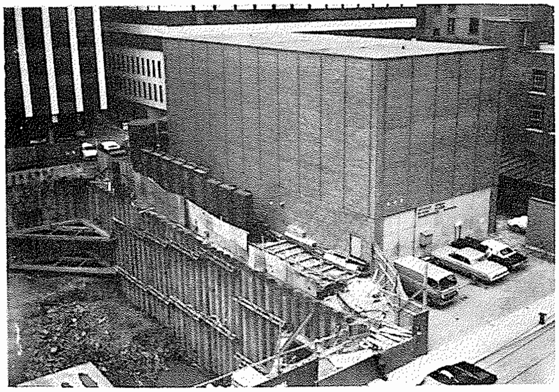

FIG. 3. View of tied-back sheet pile wall and adjacent building when excavation was almost complete.

Sheet piling was driven to refusal, penetrating the upper few inches of shale. Construction followed the usual pattern of excavating to the upper tieback levels, placing and stressing these tiebacks, excavating to lower tieback levels, and final excavation after placing and stressing the lower level of tiebacks. There were two unusual occurrences; the first was a planned retensioning of the upper tieback tendons about 3 weeks after the initial tensioning and the second was the unplanned removal of rock adjacent to and below the base of a short section of the sheet pile wall. The retensioning was carried out when it was observed that the tendons had lost about one third of the prestress loads and that the adjacent building had settled about $\frac{1}{2}$ in. (1.3 cm). The undermining resulted in an immediate downward and outward movement of the sheet piles involved and caused changes in the tendon loads at the instrumented section about 40 ft (12 m) distant. In the analysis of observations the effects of the undermining have not been considered typical. Their occurrence is, however, of interest when judging the over-all factors of safety needed in this type of construction work.

Geotechnical Profile

Samples of soil were obtained by borings before construction and as blocks from three levels during excavation.

In July a borehole was made only a few feet from the southeast corner of the transformer building. Continuous samples were obtained between 9 and 25 ft (2.7 and 7.6 m) in the profile using a thin-walled fixed piston sampler of 55 mm diameter. Included in the geotechnical profile, Fig. 4, are the results of laboratory tests and descriptions of the deeper materials supplied from an earlier boring at the site and verified during excavation.

The surface material is a rubble fill varying in thickness from 6 to 10 ft (1.8 to 3 m) beneath which there is 23 ft (7 m) of Champlain Sea deposits commonly known as Leda clay. The upper 16 ft (5 m) is a stiff silty clay and below this depth the soil becomes much more silty with occasional very thin seams or pockets of coarse silt and sand.

At a depth of 33 ft (10 m) a deposit of till is encountered which varies from 3 to 10 ft (1 to 3 m) in thickness from the north to the south across the site. It is primarily a silt but contains boulders up to 6 ft (2 m) diameter. The till in turn rests on a black shale of the Billings formation on which several large buildings, including those immediately east, north, and south of the site, are founded. The rock dips gently southward with its surface elevation varying across the site from 184 to 194 ft (58 m).

The classification tests indicate that the soil between the depths of 10 and 26 ft (3 and 8 m) is a silty clay of high plasticity with water content near the liquid limit. Sensitivity values between 10 and 40 were obtained using a laboratory fall cone. The undrained strength was nearly constant in the clay layer with an average value of 0.65 kg/cm^2 measured in confined triaxial compression of tube samples.

Results of consolidation tests plotted on Fig. 4 indicate an apparent preconsolidation pressure P_c' of about 2.5 kg/cm^2.

Field Instrumentation

In order to monitor soil movements and stress changes resulting from construction activities several settlement points, horizontal movement gauges, piezometers, and load cells were installed. The positions of these instruments, shown in Figs. 1 and 2, were selected so that detailed information might be obtained on that section of wall supported by the tendons through the second set of wales from the southwest corner of the site. In addition, several reference points on this wall section and elsewhere were established for optical measurements of displacements.

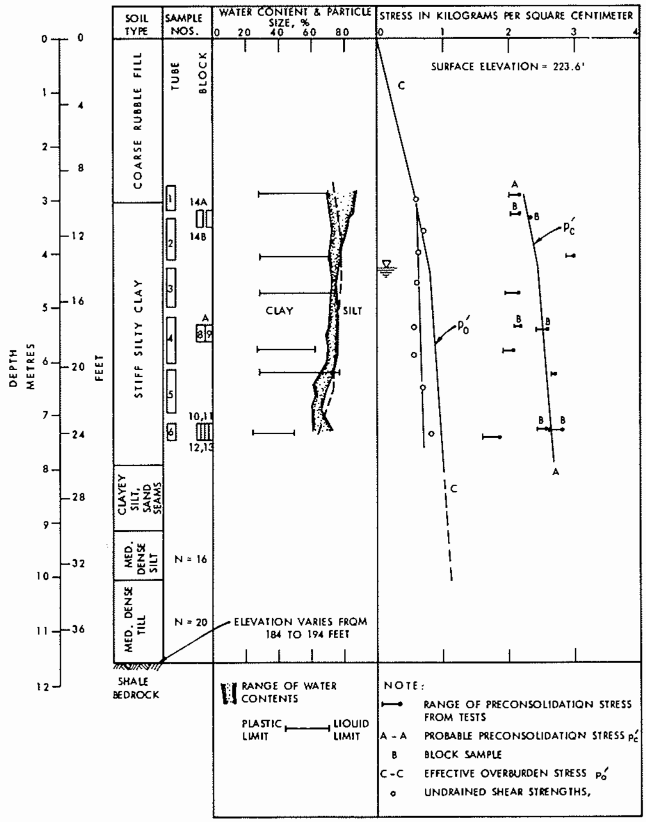

FIG. 4. Geotechnical profile.

Four Geonor no-volume-change piezometers were installed in pairs at distances of 6 ft and 16 ft (2 and 5 m) from the wall and two open stand-pipe piezometers were installed at distances of 5 ft and 60 ft (1.5 and 19 m) from the wall. The no-volume-change piezometers, accurate to a pressure head of 0.2 ft (6 cm) were selected because of their rapid response and capability of measuring transient pore water pressures. With open stand-pipe piezometers it was possible to determine pressure head to 0.1 ft (3 cm).

Each wale was restrained by three tendons. The wales in the instrumented section were 14.4 ft (4.4 m) long with the tendons on 4.8 ft (1.5 m) centers. Load cells were placed on both of the outer tendons at each level to measure the time variation of the tendon loads. These cells are described by Lacroix (1966). Loads were measured to an accuracy of about 0.5 t (1 kip).

The horizontal movements of the sheet pile wall were measured using horizontal movement gauges installed through holes cut in the wall and by optical measurements on horizontal scales attached to the wall at various points. Each horizontal movement gauge consisted of two concentric pipes with an auger attached to the inner one. As the gauge was jacked horizontally into the soil a machined bearing surface prevented soil from entering the space between the inner and outer pipes. After the gauge had been jacked a horizontal distance equal to the vertical distance to the base of the wall, the anchor was advanced about 2 ft (0.6 m) into undisturbed soil by rotating the inner pipe. Readings of the relative movement between the wall and the inner pipe were made with a dial gauge bearing on a machined ring welded to the wall and were easily reproducible to 0.002 in. (0.005 cm). The positions of these gauges are shown on

Figs. 1 and 2. Optical measurements involved the reading of horizontal scales intersecting a vertical reference plane using a theodolite. Readings were made to the nearest 0.01 ft (0.3 cm).

Settlement points (S in Fig. 1) on the building and on the transformer coolers between the building and the edge of the excavation were surveyed using a precise level and with reference to a bench mark on an adjacent building founded on rock. These measurements were made to the nearest 0.01 in. (0.025 cm).

Field Performance

Using an undrained strength of 0.65 kg/cm², as measured from tube samples, the critical height of a vertical slope would be predicted to be about 40 ft (12 m). The fact that the contractor excavated all materials below 3 m (10 ft) over most of the site in one lift using a drag line and maintaining a slope of about 80° indicates that this high undrained strength can be mobilized in practice for several days. It is interesting to note that the value of c_u is about equal to $P_c'/2$ as found for some other Leda clays.

Continuous readings typical of the data collected from the instrumentations described above are presented and discussed below.

Readings of horizontal wall movements and tendon loads are presented, together with a diagram showing major construction sequences, in Fig. 5. The close agreement between measurements taken optically and those taken with the horizontal movement gauges indicates that there was little horizontal strain in the soil beyond the anchors provided for the latter instruments. The decreases in tendon loads appear to reflect the wall movements although this aspect will be discussed in more detail in a later section of this paper.

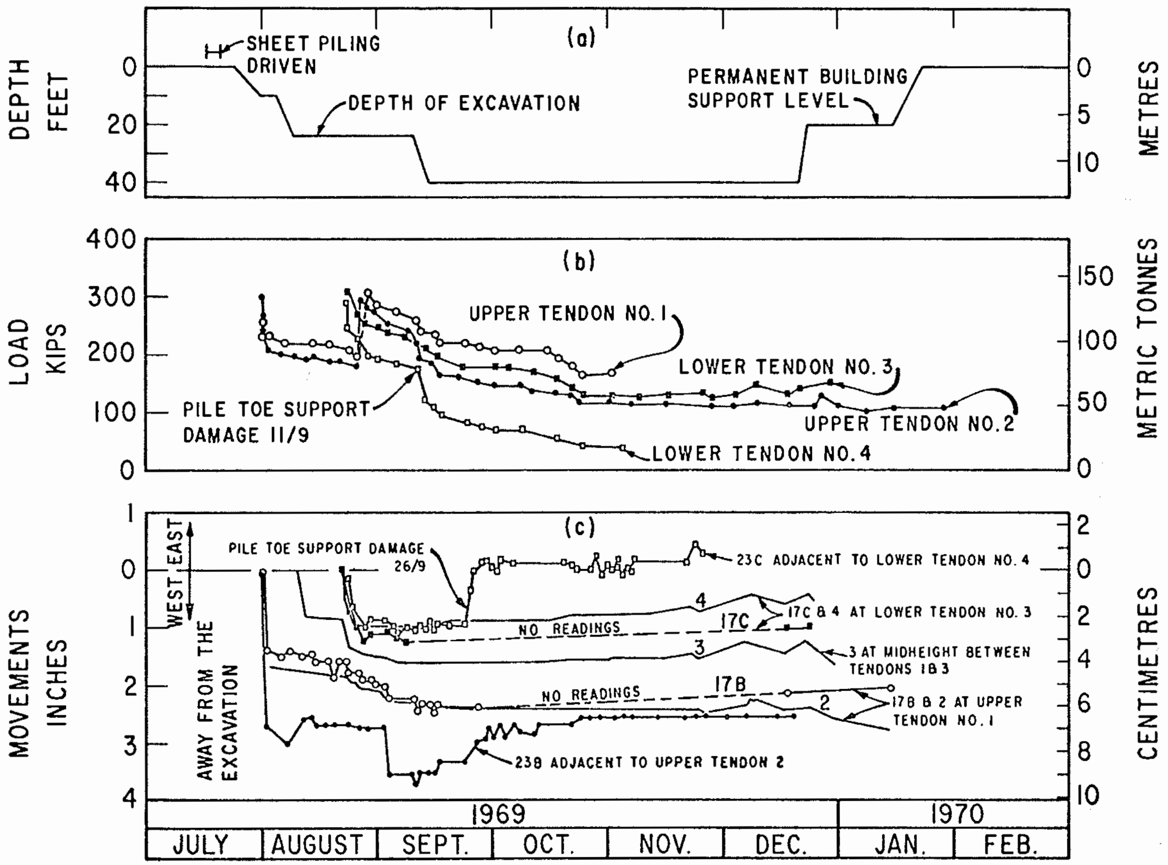

FIG. 5. (a) Construction progress. (b) Tendon loads with time. (c) Horizontal wall movements with time.

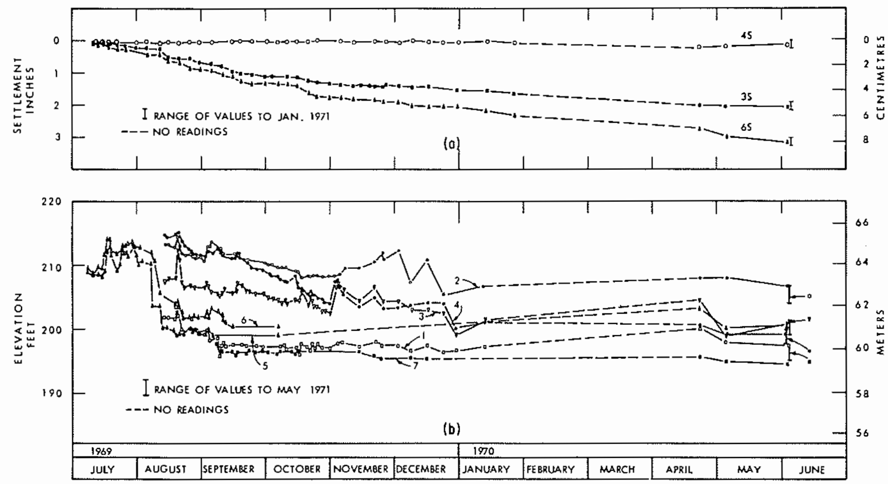

FIG. 6. (a) Vertical movements of ground surface with time. (b) Piezometer levels with time.

It is apparent from Fig. 5 that the undermining of certain sections of the sheet piling resulted in undesirable movements and decreases in tendon loads.

Vertical settlements and piezometer readings are plotted in Fig. 6. The no-volume-change piezometers apparently registered the immediate changes in pore water pressure due to excavation and to prestressing of the wall. The sensitivity of these instruments is demonstrated by the fact that they registered peaks due to a prolonged rainfall on August 19. The open stand-pipe piezometers give accurate readings only under steady state conditions and became dry probably due to the decrease in pore water pressure around the porous tips, which were situated at a depth that was close to the more pervious silty layers. Although it is difficult to estimate the time required to establish a steady state flow regime in the clay layer, the closed piezometer readings are considered to represent the equilibrium drawdown. Vertical settlements shown in Fig. 6 appeared to develop continuously with the ground-water drawdown.

The deflected shape of the flexible wall can be approximated, at various times, frpm the horizontal wall movements as plotted in Fig. 7. It is appropriate to note, for later discussions, that the base of the wall did not appear to move relative to the rock at the instrumented section and that the inward movement near the top of the clay layer (upper tiebacks) is about twice the inward movement at the bottom of the clay layer.

Vertical movements of ground and buildings adjacent to the excavation as plotted in Fig. 6 are considered representative of a considerably larger group of measurements made along the wall and around the adjacent buildings. The maximum movements were about 50% larger than those shown on Fig. 6,

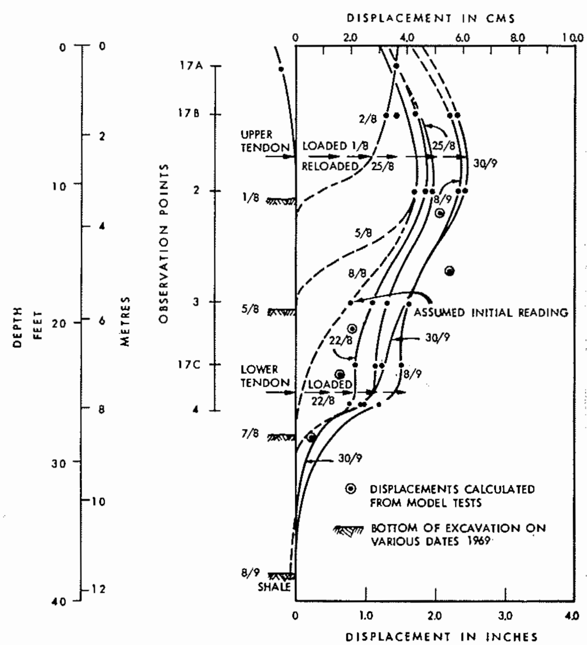

FIG. 7. Displacement of sheet pile wall at instrumented section.

but were, in all cases, affected by unplanned construction events. A comparison of data from Fig. 6 with observed vertical movements adjacent to excavations summarized by Peck (1969) is shown on Fig. 8. The movements observed at the Ottawa site fall in Zone I and tend to support the previous data presented by Peck. Zone I represents the range of vertical movements observed in sand and soft to hard clays for average workmanship. Even with the inclusion of the maximum movements at the Ottawa site the points would still remain in Zone I. No attempt has been made in this comparison to distinguish between settlements due to changes in groundwater conditions and those due to soil deformation under applied horizontal stress.

It can be seen from Fig. 8 that the efforts to minimize settlements at this site have been more successful than at the sites on which the lower boundary of Zone I is based. Even with great care and excess strength in the tieback system, however, this project has again demonstrated that significant settlements will be induced in adjacent ground and buildings unless specialized trenching or slurry wall techniques are used.

Prestress loads placed in the tendons and the retensioning of the upper level of tiebacks created the maximum observed lateral soil pressure conditions. The tendon loads decreased with time as the soil deformed inward (in the passive sense) under these prestress loads. The nearly constant tendon loads remaining after an interval of 2 to 3 months from the initial tensioning are considered to be most appropriate loads to use in a comparison with the loads calculated from normal design procedures. This comparison, with the design pressure diagram suggested by Peck (1969) for calculation of strut or tendon loads in a clay soil, is shown in Fig. 9. The equivalent pressures, shown by points, have been calculated by equating each observed tendon load to a uniform stress acting on an area defined by the midplanes of adjacent supports in the vertical and horizontal directions. This simplification is part of the technique required in using the design method. The uniform pressures distributed in a trapezoidal fashion are also shown in Fig. 9 for comparison with the design envelope.

The agreement between observed long-term tendon loads and the upper range of the diagram in Fig. 9 (0.4 γ H) is satisfactory, indicating that the diagram might be used for predicting long-term loads. The diagram does not consider lateral pressures that might be created by prestressing in an attempt to minimize the movements of adjacent ground. Finally, a pressure diagram without a decrease in pressure at the bottom of the wall is likely to be more appropriate where a tieback system is used rather than struts. This suggestion is discussed further in a later section.

Laboratory Testing of Block Samples

At each of the three elevations where block samples were obtained the following series of triaxial compression tests was carried out:

(a) Constant mean normal stress tests and constant stress ratio tests under fully drained conditions.

(b) Fully drained tests where the applied stress changes were designed to simulate the over-all stress changes in the soil adjacent to the sheet piling.

It was originally intended that the tests noted in (a) above would provide data for

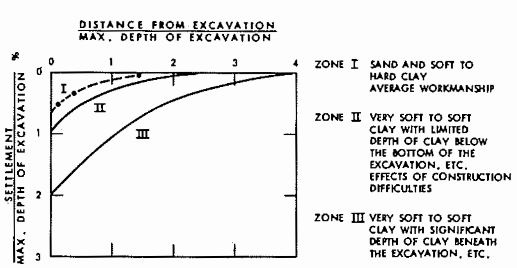

FIG. 8. Comparison of observed settlements to settlements adjacent to open cuts (after Peck 1969).

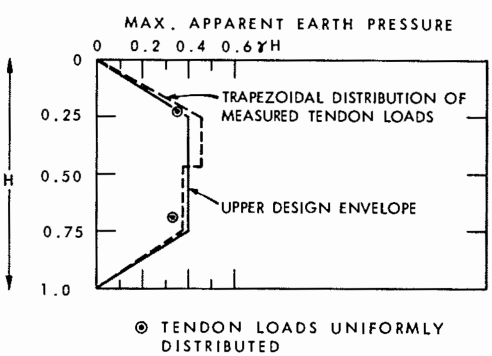

FIG. 9. Comparison of observed tendon loads with design envelope suggested by Peck (1969).

use in a linear or multilinear (elastic) finite element analysis of the stresses and deformations within the soil being retained by the wall. Although the data are considered suitable this analysis has not been completed to date. The data are, however, of significance to later discussions in this paper and will be presented, in part, in later figures.

The tests noted in (b) above were used in a simple 'stress path' analysis of the field performance. Using simplified assumptions regarding the over-all stress changes, the measured deformations in the tests were used to predict deformations in the clay layer being retained by the wall. The following section outlines the procedure and assumptions.

Model Testing and Analysis of Soil Movements

Assuming that all of the significant deformations would develop within the clay mass between 3 and 9 m (10 and 30 ft) depth and bounded in the interior by a line extending through the toe of the wall at 30° to the horizontal it is possible to represent this mass by 38 square elements as shown on Fig. 10. The initial effective stress in each of these elements was calculated in the usual manner assuming $K_0 = 0.6$ for the clay.

Changes in stress *in situ* due to tendon loads will in general depend upon many factors including the deformation properties of the wall and adjacent soil layers, the sequence of construction and the degree of drainage in the soil. If it is assumed that the clay behaves linearly elastic the problem is simplified to one of estimating the initial and final stress conditions. From the tendon loads (Fig. 5) and the no-volume-change piezometer readings (Fig. 6) it is considered that primary deformation was complete by September 30, 1969. All stress changes and deformations beyond this date are considered to be a result of soil and anchor creep or unplanned construction activities (as noted earlier). The final stress conditions were estimated from field readings on this date. Piezometers 6 and 7 located on the west side of the building were not considered in drawing the flow net in Fig. 10 because their tips were located in the lower, more pervious silt.

The final vertical effective stress was calculated, with reference to the steady state flow net drawn on Fig. 10 from the no-volume-change piezometer readings, by assuming that changes in vertical stress resulted only from changes in the ground-water regime. Integration of the final total horizontal stresses in the soil adjacent to the wall over the appropriate area will then give a force which must be in equilibrium with the loads recorded in the tendons on September 30, 1969.

In the absence of established procedures for calculating lateral pressure distributions adjacent to a tied-back flexible wall a trial and error process of comparing deflected shapes of the wall section with measured wall displacements (Fig. 7) led to the distribution shown on Fig. 10. The 30 t reaction, R, would indicate a coefficient of sliding friction of 0.25 between the wall and the rock. Use of a trapezoidal pressure distribution would not alter the lateral pressure in the clay layer significantly. The final total horizontal stresses in elements adjacent to the

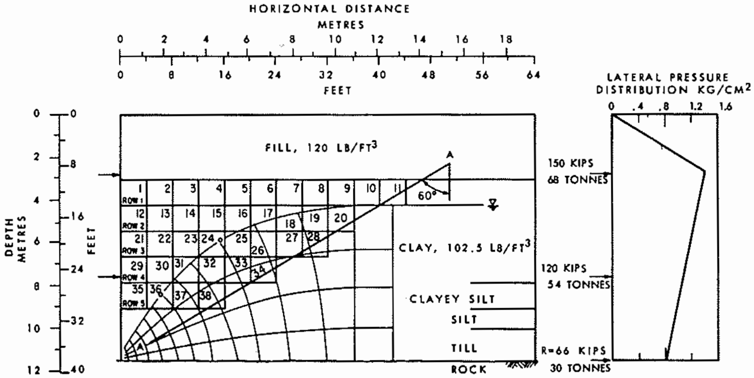

FIG. 10. Elements and equilibrium stress conditions used for stress path analysis.

wall were obtained from the lateral pressure distribution drawn in Fig. 10. The final total horizontal stresses in the other elements were estimated by a simple linear reduction of the stresses adjacent to the wall to balance with the initial *in situ* total stress at the right-hand boundary of the last element in each row of elements. Final effective horizontal stresses were then calculated with reference to the flow net making adjustments where necessary to satisfy total horizontal stress equilibrium on the line A–A on Fig. 10.

It is quite apparent that the general stress equilibrium equations have been violated in the above simple approach to estimating stress changes. In addition, it is assumed that the vertical and horizontal directions are axes of principal stress in order to use the deformations measured in triaxial tests to predict deformations *in situ*. Strain continuity within the soil mass is not ascertained. Since only five triaxial stress path tests are required for the 38 elements, however, it seems worthwhile to carry out these simple calculations.

One triaxial test was conducted for each row of elements beginning at equilibrium under the initial effective stresses for the corresponding depth. Stress increments corresponding to the estimated stress changes at the center of each element were applied to the specimen starting at the last element in each row and working towards the first element (closest to the wall). Each increment was extended several hours beyond primary equilibrium. The data from three of these tests are plotted in Fig. 11 and refer to rows 2, 3, and 4 as indicated. The test data for row 1 and row 5 were similar to the respective adjacent rows and are not plotted. Element numbers are also indicated at the appropriate stress points on Fig. 11(a). Volumetric and distortional strains resulting from the applied stress changes are plotted in Fig. 11(b) and 11(c) respectively. The principal strains can be calculated from the data in Fig. 11 using the equations given below. Since the soil testing was carried out in the triaxial apparatus and the *in situ* deformation is assumed to be plane strain it is necessary to predict the deformations which would occur under plane strain boundary conditions. The following procedure was adopted:

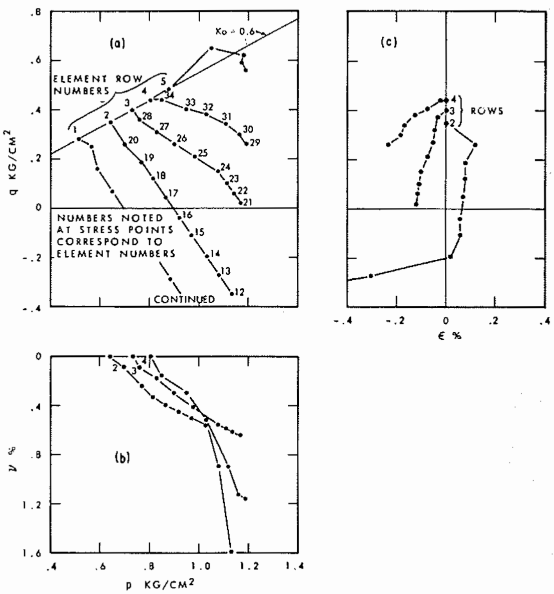

FIG. 11. Data from stress path tests. (a) Stress paths. (b) Volumetric strain. (c) Distortional strain.

Since $\delta\nu >> \delta\varepsilon$ (see Figs. 11(b) and 11(c), $\delta\varepsilon_1 \doteq \delta\varepsilon_3$

Then $\delta\nu$ (triaxial) $= 3\delta\varepsilon_1$ and $\delta\nu$ (plane strain) $= 2\delta\varepsilon_1$ giving $\delta\nu$ (plane strain) $= \frac{2}{3}\delta\nu$ (triaxial)

Also $\delta\varepsilon$ (triaxial) $= \frac{2}{3}(\delta\varepsilon_1 - \delta\varepsilon_3)$ and $\delta\gamma$ (plane strain) $= (\delta\varepsilon_1 - \delta\varepsilon_3)$ giving $\delta\gamma'$ (plane strain) $= \frac{3}{2}\delta\varepsilon$ (triaxial).

Using the plane strain values of $\delta\nu$ and $\delta\gamma$ for each increment of stress change the values of vertical and horizontal strain increment were calculated as:

$$\delta\varepsilon_v = (\delta\nu + \delta\gamma)/2 \quad \text{and} \quad \delta\varepsilon_h = (\delta\nu - \delta\gamma)/2$$

These strains were applied to the field elements and the total boundary movements were calculated. The horizontal wall movement adjacent to any row of elements was calculated as the sum of the horizontal change of dimension of each element in that row. The vertical settlement above any column of elements was calculated as the sum of the vertical change of dimension of each element in that column.

Calculated wall movements are plotted on Fig. 7 and are found to compare favourably with the measured movements as of September 30, 1969. The vertical settlement calculations varied from 1.25 in. (3.2 cm) in the first column of elements (nearest the wall) to 0.25 in. (0.64 cm) in the fourth column (corresponding to the east side of the Transformer building). These predictions are only about one half of the measured settlements but do not include settlements in the fill (due to vibrations) or settlements in the lower 8 ft (2.5 m) of silt and till. Because most of the predicted soil movements occurred in elements close to the wall, the violations of equilibrium and continuity are not considered as serious in this problem as they might be in other problems and the simple stress path approach appears to be helpful in predicting soil movements.

It is interesting to note that a one-dimensional consolidation calculation based on *total* drawdown of ground water to the base of the clay layer together with the consolidation test data would predict a vertical settlement of only 0.5 in. (1.3 cm). The increase in horizontal stress above the *in situ* K_0 value is then predicted to increase the vertical settlement above that due to ground-water drawdown (increase in vertical effective stress). Since this is contrary to the usual situation and reflects on the practice of prestressing the tendons above the at-rest pressure it is reasonable to investigate this behavior in more detail. The results of the fully drained constant mean normal stress and constant stress ratio tests mentioned earlier offer a plausible explanation, as outlined in the following section.

Stress-strain Behavior of the Clay

Following Mitchell (1970) the data from drained triaxial tests were plotted as exemplified in Fig. 12. Figure 12(a) shows void ratio changes in vertically oriented and horizontally oriented specimens tested under a constant effective stress ratio (*i.e.* the effective stress ratio was constant at the end of each increment of loading). A point of volumetric yield for each test is defined as shown in Fig. 12(a). Figure 12(b) shows stress-strain curves for one vertical and one horizontal specimen tested on a constant p stress path. These two tests are typical of many similar ones and it may be noted that the specimens

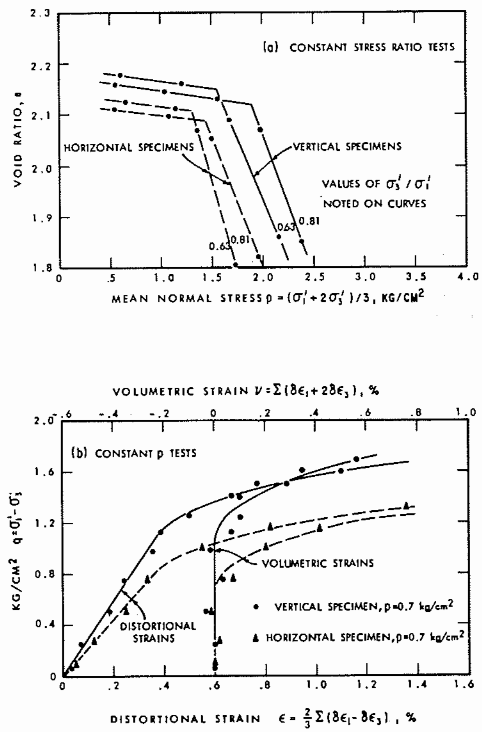

FIG. 12. Triaxial compression test data.

exhibited no significant volume change prior to yielding (yielding being defined, in this case, by the limit of the linear relationship between deviatoric stress and distortional strain). It may also be noted that both yield stress and slope of the linear portion of the stress strain curve are larger for vertically oriented specimens than for horizontally oriented specimens although the strain, ε, at yield is about the same.

Other test data from the clay at a depth of 6 m were combined with the data plotted in Fig. 12 to produce Fig. 13; in Fig. 13(a) the yield data from vertical specimens are plotted in the upper quadrant and the data from horizontal specimens, in the lower quadrant. The yield curve shown in Fig. 13(a) is drawn in a (σ_v', σ_h') space in Fig. 13(b), and the estimated initial and final stresses (in elements adjacent to the wall) are shown to lie within the yield curve. Because of the uncertainty of the estimated final field stresses it was not considered appropriate to discuss further assumptions possible in transposing this yield curve from the triaxial stress plane to

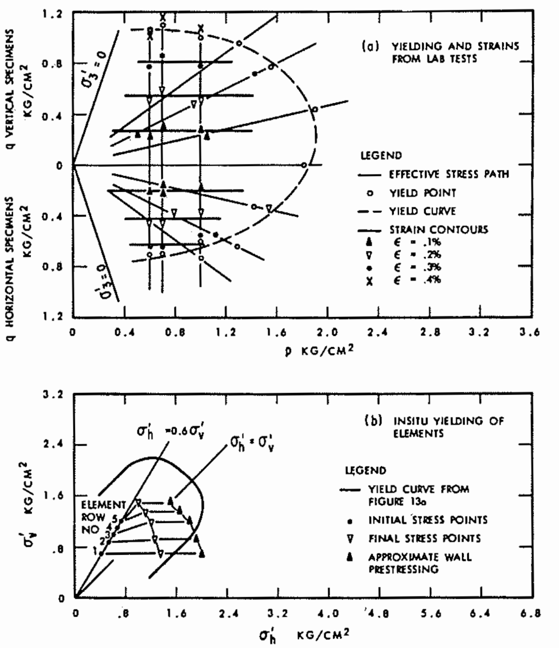

FIG. 13. Yielding due to increased horizontal stresses.

one for plane strain conditions. Despite the assumptions necessary in calculating the maximum *in situ* stresses Fig. 13(b) demonstrates that caution should be used when prestressing tieback walls in cemented clays. Large irrecoverable strains (yielding) can be initiated just as readily by increasing the horizontal stresses as by increasing the vertical stresses. If the horizontal stress is increased so that it exceeds the vertical stress in any element (such as elements adjacent to the top of the wall) the soil may yield and the resulting volume changes will cause large vertical settlements as well as large inward movements of that part of the wall. For example, had the prestress loads been maintained by continued retensioning of the tendons, the final horizontal stress in the soil adjacent to the wall would be approximated by the solid triangles drawn on Fig. 13(b). Significant yielding of the upper part of the clay layer would be predicted.

It may be noted that the triaxial stress path test for row 2 (Fig. 11), whose stress path became extensional, did yield at a point corresponding to the estimated stresses for elements 14. The resulting strains were quite large and account for a large percentage of the predicted wall movement adjacent to row 2 and of the predicted settlements above columns 1 and 2 of the field elements. The yield stress from this test lies within the yield curve of Fig. 13(b). To define yielding more accurately for this problem (and many other field problems) plane strain tests should be conducted.

In summation it is considered that the soil adjacent to the sheet pile wall approached very close to a yield condition due mainly to the prestress load applied at the tendons. Some small amount of volumetric yielding would explain the large observed vertical settlements near the wall even though the wall moved in the passive sense. It is possible that greater prestress loads could have resulted in larger settlements as well as larger wall movements into the soil. With the sensitive cemented clays, which exhibit large volume changes upon yielding, it is not always possible nor advisable to consider pushing the ground surface behind the wall upwards by pushing in on the wall.

Discussions and Conclusions

While the instrumentation was limited due to site conditions and construction activities, one aspect could have received more attention: a detailed record of possible vertical movements of the sheet piling would have been desirable even though the piles were seated on rock. A single sheet pile about 10 ft (3m) south of the instrumented section was found to have moved downward 0.03 ft (1 cm) during the period between August 30 and September 30, 1969. This amount of movement could not, however, explain the large changes in tendon loads over the same period and there is no certainty of the actual amount of vertical movement, if any, occurring at the instrumented section.

The tendon loads shown in Fig. 5 contain the details of the load losses during September and October, 1969. For the tendon sections and lengths involved, a 100 kip change in load would correspond to 3.5 cm (1.4 in.) of horizontal movement at an upper tendon and 1.8 cm (0.7 in.) at a lower tendon. Only one of the four measured tendons had a total load loss equivalent to the total elastic shortening of the tendon. The other tendons lost more load than could be explained by the wall movements, and the additional load losses are possibly due to creep in the grout or to creep in the rock mass itself (Coates and Yu 1970). Creep is understandable at this site since the shale rock has weak bedding planes and the grout was only allowed to cure a few days before loading the tendons. At another site, where a tendon was grouted into sound limestone well in advance of tensioning, a 30-day test showed little tension loss.

The large movements of the upper part of the wall (Fig. 7) tend to be in agreement with the suggestion that some yielding of the soil occurred in the upper portion of the clay layer. It would seem desirable, in order to prevent yielding, to keep the final stresses in the soil adjacent to the wall within a certain region of stress space defined, say, by the lines $\sigma_h' = \sigma_v'$ and $\sigma_h' = K_0\sigma_v'$ (see Fig. 13(b)). This could best be accomplished by a reduction in the prestress loads in the upper tiebacks with, perhaps, some small increase in prestress loads in the lower tiebacks. The resulting lateral pressure distribution would then approach more closely to a triangular distribution, and the wall movements (for an elastic soil) would be more uniform with depth. Triangular lateral pressure distributions have been calculated for uniform wall movements by Morgenstern and Eisenstein (1970).

From the measured performance of the tied-back sheet pile wall discussed in this paper the following conclusions are drawn:

1) Even with the special effort made to prestress the soil adjacent to the excavation, substantial and potentially damaging vertical movements occurred. The consequences of such movements must be considered at the planning stages of braced excavations.
2) The prediction of the amount of vertical settlement adjacent to a braced excavation in this naturally cemented clay can be aided by correlation with observed data published by Peck (1969). The observations at the Ottawa site agree with this correlation even though it is based chiefly on observations in softer clays.
3) The agreement between tendon loads calculated from the pressure diagrams also published by Peck (1969) with those actually observed is satisfactory for long-term conditions. The Ottawa results fall at the upper end of the range of values suggested in the method. The effect of an initial high prestress, if desired to reduce future wall movements, must however be considered as a separate increase in the designed tendon loads.
4) There is evidence of important loss of tendon tensions due to movement at the rock anchorage end of the tieback tendons at the Ottawa site. This could be due to creep in the shale rock, or creep in the grout in the rock socket. The loss of tension with time would have serious implications for permanent prestressed rock anchors, and is therefore being studied further at other Ottawa sites.
5) Agreement between the observed movements and movements predicted from a simple stress path approach was acceptable at the Ottawa site. This agreement may arise because elements close to the wall, where the stresses were reasonably well known, approached a yield condition and accounted for most of the predicted movements. For the general case more refined methods of predicting soil deformations should be explored.
6) While prestressing of tendons in a tied-back system is desirable, detailed laboratory testing described in this paper indicates that the prestress loads should be limited by the yield properties of the clay. This is particularly true for the sensitive naturally cemented Champlain Sea deposits. It is further suggested that the prestress loads should be increased with increased depth of the tendon location so that a triangular lateral pressure distribution is more closely approximated and there is less risk of causing yielding in the soil adjacent to the top of the wall.

Acknowledgments

The authors acknowledge the assistance provided by Prof. J. Hurtubise of Ecole Polytechnique, who made the load-measuring cells available on loan to the project. Acknowledgment is also made of the cooperation and assistance of the excavation bracing sub-contractor, Western Caissons Limited; and of the interest and support of Mr. D. G. Donaldson, Stations Engineer of the Ottawa

Hydro Electric Commission. This project was carried out cooperatively by the consulting firm of the senior author and the Division of Building Research.

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