Full-Scale Lateral Load Test of a Retaining Wall Foundation

Essai en vraie grandeur d'un mur de soutènement soumis à une charge latérale

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Summary

This paper reports the results of a full scale test to ascertain the load-movement relations for a typical pile-supported retaining wall. Two test sections, each supported on two batter piles and one vertical pile, were built back to back so that they could be loaded simultaneously by means of the same loading system. The results indicated that the conventional design procedures were quite adequate and led to a factor of safety of about 2·0.

Sommaire

Cet article rapporte les résultats d'un essai en vraie grandeur effectué pour vérifier les relations entre charges et déformations pour un mur de soutènement fondé sur pieux de type classique. Deux sections d'essai, chacune supportée par deux pieux inclinés et un pieu vertical ont été construites dos à dos de sorte qu'elles pouvaient être soumises simultanément à la même charge. Les résultats ont montré que les procédés classiques de calcul sont très satifaisants et conduisent à un coefficient de sécurité de 2,0 environ.

Introduction

Although considerable effort has been expended over the last century in attempting to determine the earth pressures that may act against ordinary retaining walls, few investigations have been directed toward the resistance that a conventionally designed wall and its foundation offers to these earth pressures. Therefore, the results of a full-scale test to ascertain the load-movement relations for a typical pile-supported wall may be of interest. The test, possibly by good fortune, indicated that the conventional procedures were quite adequate and led to a factor of safety of about 2·0.

The test was carried out on two test blocks, each representing to full scale the lower portion of a typical retaining wall. Each section was supported by two batter piles and one vertical pile. The sections were built back to back in such a manner that they could be loaded simultaneously by means of the same loading system.

The section of the wall was designed to meet the specifications of the Department of Highways, Bureau of Bridges, State of Ohio. The prototypes were walls with a height of 22 ft. from base level to top of the horizontal backfill. An additional surcharge of 2 ft. was considered to act above the ground surface. The corresponding dimensions of the prototypes and the principal calculated forces are shown in Fig. 1 a.

Each test wall was to be loaded by a single vertical reaction to replace the vertical component of the earth pressure on the stem and the weight of the soil above the heel of the wall, and by a single horizontal reaction acting at the center of gravity of the calculated earth pressure. The horizontal reaction was to be applied by means of a hydraulic jack. Therefore, it was unnecessary to extend the stems of the walls above the elevation at which the jack was to act. Consequently, the height of the test walls was approximately 9 ft. Otherwise, the external dimensions of the test walls correspond closely to those of the prototype. The pile pattern for the prototype was repetitive at 8-ft. intervals. However, since the pile loads in the test walls would be unaffected by the length of the test section, the model walls were made only 7 ft. long. The test wall is shown in Fig. 1 b.

The horizontal component of earth pressure corresponding to design conditions was approximately 90 kips. Since it was anticipated that the walls would be tested under loads equal at least to twice the design load, the reinforcement in the test walls designed for double the calculated earth pressure.

Arrangements were made to conduct the tests in an area adjacent to a foundation for which piles were at the time being driven. An excavation was made into a bank of fill and miscellaneous overburden to provide an area in which

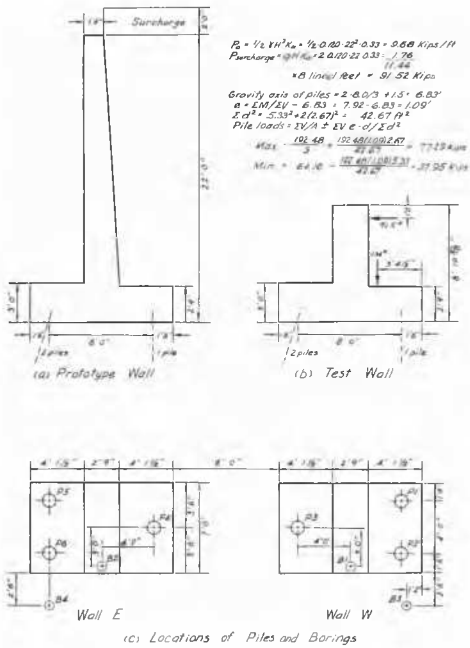

Fig. 1 Dimensions and Location Plan.
Dimensions et plan de masse.

the walls could be constructed. The piles were driven on March 26, 1958. The footings were poured on April 8 with high early strength concrete. The load test was carried out on April 23.

Soil Conditions

A general plan showing the location of the test walls and borings made at the site is indicated in Fig. 1. Borings 1 and 2 were located respectively in the west and east foundation areas. These borings were made by the Raymond Concrete Pile Company using the standard Gow procedure. Boring 1 extended to a depth of 65 ft. and boring 2 to a depth of 76·5 ft. below the elevation of the ground surface on which the footings were cast.

The borings indicated comparable but not identical soil conditions at the two locations. For a depth of about 5 ft. below footing level the subsoil consisted of a moderately stiff clay. Below that level, sandy and silty materials were encountered. Groundwater level was at a depth of about 30 ft. The results of laboratory descriptions of the samples are assembled in Figs. 2 and 3. In addition, the number of blows in the standard penetration test has been plotted for both borings.

After completion of the tests, two additional shallow borings were made, designated as Nos. 3 and 4. Boring 3, at the west test wall, extended to a depth of 8 ft.; boring 4 was carried to a depth of 6·5 ft. Samples were obtained as continuously and as carefully as possible in thin-walled steel tubes 2 inches in diameter. These samples were examined in the laboratory and were subjected to unconfined compression tests, to determinations of natural water content, and to determinations of occasional values of liquid and plastic limits. The results of the laboratory tests are summarized in Fig. 4.

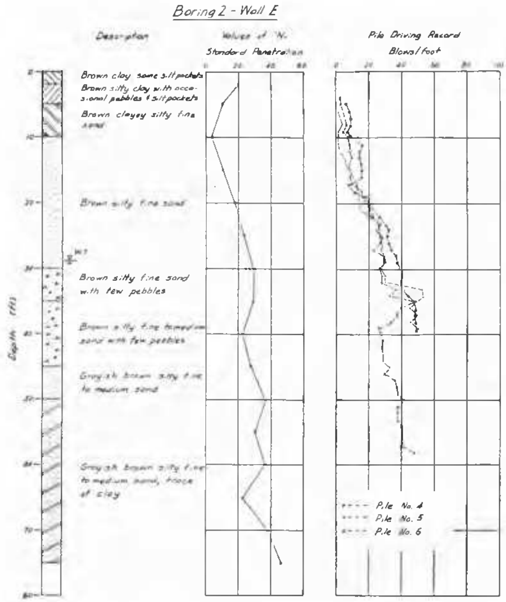

Fig. 3 Boring and Pile Driving Record, Wall *E*.
Résultats de sondages et de battages, mur *E*.

It may be noted that the unconfined compressive strengths of the clays in the upper 6 ft. below footing level ranged between about 1 and 3·5 tons per sq. ft. The natural water contents were approximately constant with depth at a value of about 20 per cent. This value is close to the plastic limit. The unconfined strengths at boring 3 were generally somewhat

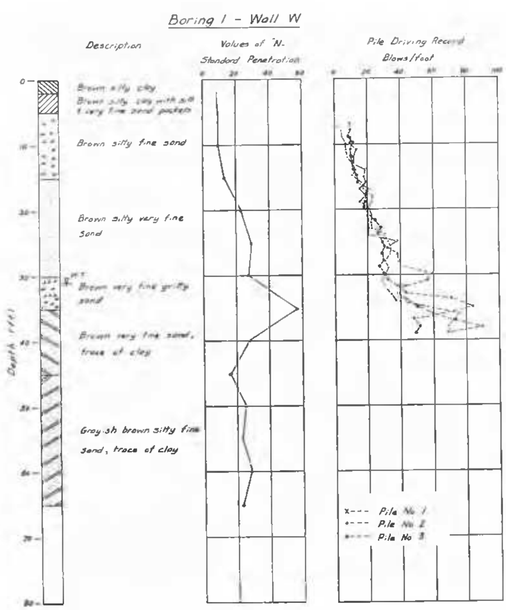

Fig. 2 Boring and Pile Driving Record, Wall *W*.
Résultats de sondages et de battages, mur *W*.

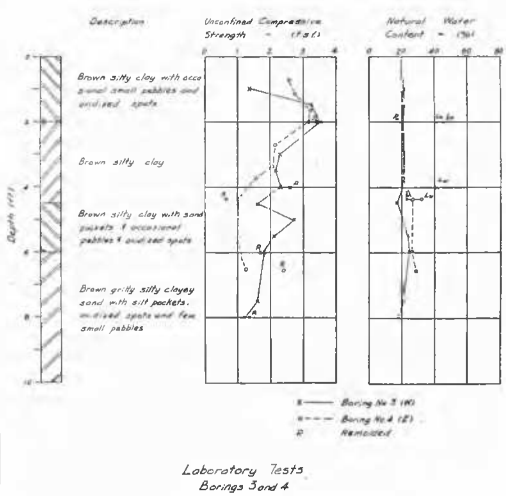

Fig. 4 Laboratory Soil Data, Upper Strata.
Résultats de laboratoire, sols de la couche supérieure.

in excess of those at boring 2. The remolded strengths were of the same order of magnitude as the undisturbed strengths, suggesting that the clay is relatively insensitive to disturbance.

Pile Foundations

The piles in footing *W*, the westerly of the two footings, each consisted of five 8-ft. Raymond step-taper sections. The lowermost section had a tip diameter of 9 ½ inches. All three piles were driven to a penetration of 39 ft. The butt diameter of the piles was 13 3/8 inches. The piles were driven with a 65 C Raymond hammer, of which the rated energy is 19 500 ft. lb. Piles 1 and 2 were driven on a 1:3 batter and were not reinforced. Pile 3, the tension pile, was reinforced with two No. 10 bars the full length of the pile.

In the east footing, designated as Footing *E*, the two batter piles Nos. 5 and 6 were of the same dimensions as the piles in Footing *W* and similarly extended to a depth of 39 ft. These piles were reinforced by means of four No. 6 bars with No. 3 hoops at 8-in. centers. The tensile pile in this group, designated as No. 4, developed a driving resistance on the order of 26-28 blows per ft. at a penetration of 39 ft. This was judged to be inadequate, as the other piles had all developed a resistance of more than 48 blows per ft. at a depth of 39 ft. Therefore, pile No. 4 was driven to a depth of 58 ft. As a consequence, it consisted of seven full units of corrugated step-taper shells each 8 ft. long and a short section of an additional shell. The butt diameter was 16 3/8 in. This pile was reinforced with two No. 10 bars extending full length and five No. 6 bars with No. 3 hoops at 6-in. centers extending to half the length.

A graphical record of the resistance to pile driving is shown for Footing *W* in Fig. 2 and for Footing *E* in Fig. 3. These records may be compared directly with the results of the standard penetration tests made during the test boring.

Loading Arrangement

The general method of loading is indicated in Fig. 5. The vertical reaction representing the weight of the earth on the heels and the vertical component of the earth pressure was supplied by means of steel ingots acting through a series of distributing beams, which in turn transferred the vertical reaction to the heels of the walls through rollers. The total vertical reaction was kept constant throughout the test at the calculated value corresponding to the design earth pressure. This amounted to 134 kips on the heel of each abutment.

The horizontal load was applied approximately 8 ft. above the base of the footings by means of a strut and horizontal jack of 400 kip capacity. The ingots were stacked in such a manner as to provide a free space through which the strut passed without contact with the weights providing the vertical load. A piece of plywood was used to transmit the reaction from the strut to the concrete in order to avoid crushing the concrete. The test setup is shown in the photograph, Fig. 6.

The system of observations was devised to permit measurement of lateral and vertical movements in some detail, to provide checks on the accuracy of the observations, and to detect significant distortions of the structure itself. The locations of the reference points are shown in Fig. 5. The vertical movement of the base slab of each wall was measured at three points on the toe and one on the heel along two lines in an east-west direction, one over the north batter pile and the other over the south batter pile. These measurements were established to permit detection of any bending distortions or cracking of the base slab. In addition, the vertical movement was measured near the top and the bottom of the stem, again on both the north and the south sides of the walls. Horizontal movements were measured near the top

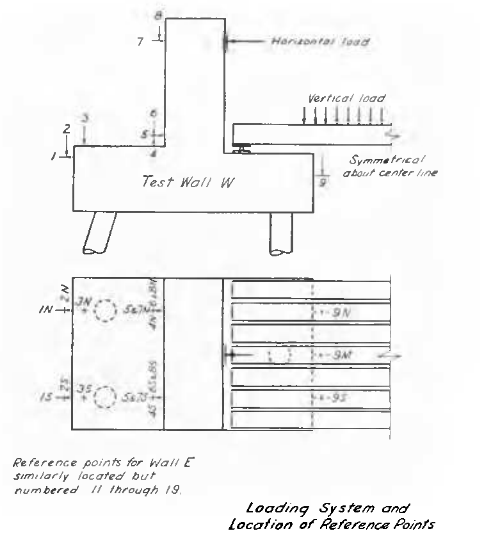

Fig. 5 Loading System and Location of Reference Points.
Moyen d'application des charges et position des repères.

and bottom of each wall and on the front and back side of each footing, in two separate lines, one near the north and one near the south side of each footing. The reference points were provided with dial indicators reading to 0·001 in. The movements were measured with respect to a separate steel reference framework supported by angles driven into the ground several feet from the test blocks. The arrangement of the reference system and of the reference points and dial indicators is shown in the photograph.

In addition, a separate set of eight reference points, four on each footing, located near the corners of the footings was established and observed by means of level and transit to detect movements of the reference system. The results indicated that the reference frame was stable.

Test Loading

All the reference points were observed prior to the addition of any vertical load, after the addition of 14 ingots, and after the addition of the 40 ingots constituting the full vertical load. Three sets of initial readings were taken under the full vertical load prior to the application of horizontal loads. The loading then took place in accordance with the following schedule.

Elapsed Time (minutes)	*Load* (kips)
0	0
9	90
20	0
29	70
36	110
51	0
64	140
76	160
86	180
114	0
240	180
262	200

Fig. 6 Photograph During Test.
Photographie prise pendant l'essai.

Results of Observations

A study of the data for the individual reference points indicated that each wall moved essentially as a unit and did not experience structural distortions large enough to permit detection. Moreover, each wall moved approximately parallel to its original position. Therefore, the results of the observations can be summarized with sufficient accuracy by diagrams indicating the average east-west movements of each wall. Such diagrams are shown in Fig. 1.

The character of the movements of the walls under the design lateral load of 90 kips can be seen in Fig. 8*a* where the original position is represented by solid lines and the position under the 90 kips loading by the dash lines. The movements are exaggerated 24 times. The magnitude of the lateral movement of the footings was approximately 0·08 in. for Footing *E* and approximately 0·12 in. for Footing *W*.

Under a load of 180 kips, equal to twice the lateral design load, the positions of the two walls are as shown in Fig. 8b. Again, the exaggeration of the movement is 24 times. The total movement of wall *E* in a horizontal direction was approximately 0·4 in. and that of wall *W* approximately 1·2 in. The behavior of the walls, as the load was increased to 180 kips, is shown in Fig. 7.

Under a load of 180 kips wall *W* was beginning to experience a slow plastic creep. Therefore, the load was reduced to zero to determine the permanent set under these conditions. The zero reading indicated a lateral displacement of about 0·1 in. for wall *E* and about 0·4 in. for Wall *W*. Thereafter the load was increased briefly to 180 kips and then increased to 200 kips in an attempt to produce failure. It was found that the load could barely be held at 200 kips as the deformation of wall *W* took place at a fairly constant rate. Therefore, after the movement of wall *W* reached approximately 3 in., the test was discontinued. At this time the movement of wall *E* was somewhat greater than $\frac{1}{2}$ in.

When the load was reduced to zero after the large deformations of wall *W* had led to termination of the tests, wall *E* recovered about 3/8 in. of its lateral displacement and returned to within about $\frac{1}{4}$ in. of its original position within a period of about 1 hour. As soon as the load was reduced to zero, wall *W* began to move energetically backward and within the same period of time recovered about 2 in. of the 3-in. displacement it had experienced. Most of the recovery occurred within a few minutes. The base of the wall slipped with respect to the underlying soil; this behavior suggested that the recovery must have been the result of restoring forces supplied by the piles themselves.

After completion of the tests, piles 1 and 2, the batter piles beneath wall *W*, were excavated to a depth of about 2 ft. to permit inspection. The external appearance of the

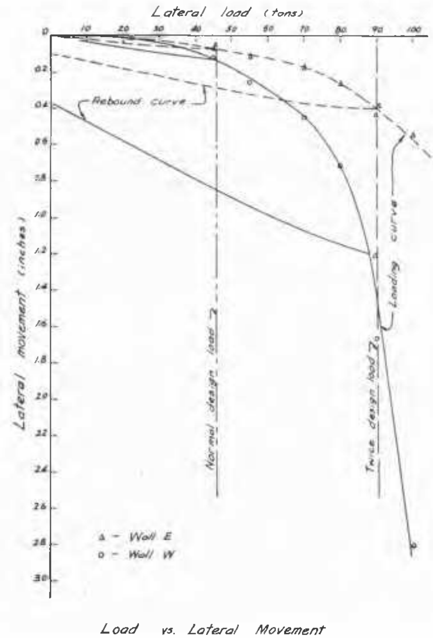

Fig. 7 Load vs Lateral Movement.
Charge en fonction du mouvement latéral.

piles indicated no signs of distress, and no cracks were apparent in the concrete near the pile heads. The entire monolith was examined with care to detect possible cracks. None was observed. However, a space was found on the side of the piles toward which the motion took place under horizontal loading. It was apparent that the upper 2 or 3 ft. of clay was displaced by the pile in the direction of motion, but when the pile straightened under removal of load the uppermost portion of the soil did not completely follow it back to its final position.

Discussion of Results

In connection with earth pressure observations against model retaining walls in the laboratory, as well as observations of the movements of retaining walls and abutments in the field, it has been noted that a lateral movement on the order of 0·1 per cent of the height of the wall seems to be necessary to develop the active earth pressure. Movements of this magnitude have been observed under many circumstances in the field and are considered unavoidable and not detrimental. Since the prototypes of the test walls would have been 22 ft. in height, the lateral movement to develop active pressure would then have been expected to be on the order of 0·02 ft. or approximately $\frac{1}{4}$ in. Under the design load of 90 kips, wall *W* moved approximately 1/8 in. and wall *E* approximately 1/15 in. Therefore, both walls must be considered to have exhibited satisfactory performance, even excellent performance, under the loads for which they were designed.

Plastic movement was not yet noticeable with respect to wall *E* even under twice the design load of 180 kips but had become significant under that loading for wall *W*. Since the test was performed rapidly, it is possible that plastic movement would have been of greater importance for both walls under smaller loads than 180 kips had lateral loads of this magnitude developed in the field and been exerted for a long period of time. Under any circumstances, however, it appears that wall *W* experienced considerably more motion than did wall *E*.

Because the tension pile No. 4 did not develop a driving resistance as great as that of the other piles at a depth of 40 ft., the length of the pile was increased as indicated previously. Consequently, at a shallow depth such as 5 or 10 ft. beneath footing level, the cross-sectional area of the piles below wall *E* was substantially greater than that at the same depths below wall *W*. Various calculations have been made to attempt to evaluate the additional resistance to lateral displacement offered by the larger and longer pile in footing *E*. It has not been possible to arrive at a satisfactory conclusion, but all the studies indicate that the added resistance, due to the larger and longer pile, should be substantial. This may account for the greater resistance of wall *E*. Since the pile was, in addition, heavily reinforced, it could also provide appreciable bending resistance.

It is usually required that the factor of safety of a retaining wall against sliding should be at least 1·5, even when the passive pressure of the soil in front of the footing is disregarded. In the test walls the passive pressure was actually zero. At the design earth pressure of 90 kips, the required cohesion of the clay or the adhesion between base and clay would, therefore, be $90 \times 1{\cdot}5/11 \times 7 = 1{\cdot}75$ kips per sq. ft. At base level the shear strength of the soil was, according to Boring 3, about 1·3 kips per sq. ft. beneath wall *W*. If this value were representative of the average conditions the sliding resistance would provide a factor of safety of only 1·12. Moreover, the toe pressure, calculated according to the usual procedures, would have been 5·95 kips per sq. ft. The value may be compared with the ultimate bearing capacity of the clay, on the order of 10 kips per sq. ft., or a factor of safety slightly less than 2. Under twice the design earth pressure that resultant force would have fallen outside the base. In design of an actual wall, these deficiencies might have been overcome by increasing the base width or by using piles. The latter alternative was adopted for the test walls.

The calculated pile loads, on the assumption of axial

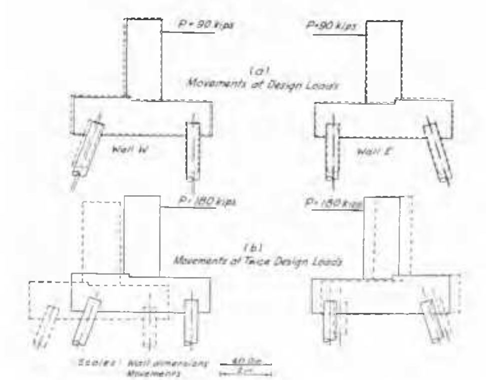

Fig. 8 Wall Movements at Design Load and Twice Design Load.
Mouvements du mur soumis à la charge de calcul et à deux fois la charge de calcul.

loading, were 77 kips for each battered pile and 37 kips for the vertical pile. Under twice the design earth pressure these values became 109 kips and — 26 kips respectively. The piles were driven to a minimum resistance of 110 kips according to the Engineering News formula.

It is apparent that the conventionally designed pile foundation behaved in an entirely satisfactory manner and that it, in fact, provided a factor of safety of almost exactly 2 against failure. Nevertheless, it is equally apparent that the action of the foundation was much more complex than was assumed in the conventional calculations. The marked sliding of wall *W* toward its original position upon unloading, for instance, demonstrated that the piles actually carried much if not most of the lateral load in bending.

Acknowledgment

The tests were performed by the Raymond Concrete Pile Company under the supervision of Mr. R. O. Marsh, District Manager, Pittsburgh. The writers designed the instrumentation, directed the conduct of the tests, and made the observations. Dr. George E. Triandafilidis, Asst. Prof. of Civil Engineering at the University of Illinois, reduced the data and digested the results.