FIELD MEASUREMENTS ON INSTRUMENTED PILES UNDER AN OVERPASS ABUTMENT

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The usual design procedure for piled bridge abutments over thick deposits of soft compressible soils has been to consider only those forces acting above the footing level. Several structures have experienced large horizontal movements resulting in damage to the structure because of the shortcomings in the conventional design procedures. For this reason, a bridge abutment on Interstate 80 in New Jersey was instrumented to determine its behavior and to provide a clearer definition of the problem. The instrumentation installed on a pile-supported abutment is described, and suggestions are made regarding some improvement for possible future field research. Qualitative results were in accordance with the anticipated behavior of the soil-pile system. The results obtained from these field measurements have confirmed that abutments supported on piles driven through soft compressible soils have the tendency to tilt toward the backfill when the shearing strength of these soils was exceeded because of superimposed embankment loads. These findings permit the tentative establishment of limiting values for stresses imposed on compressible layers with relation to the backward tilting of such conventionally designed abutments, and they suggest various remedial measures.

•THE CONVENTIONAL design procedure for bridge abutments supported on piles driven through thick deposits of soft compressible soils is to consider only those forces acting on the abutment above the footing level. These forces usually include only the girder reactions, the weight of the abutment, and the conventionally determined active lateral earth pressure. In some cases, an estimate of the effect of the consolidating strata on the piles in bending is considered (1). The inherent shortcomings of this procedure has resulted in damage to several structures as the abutment tilts back toward the retained fill. In some cases large horizontal movements (as great as 10 in.) have developed, although to the writers' knowledge no catastrophic failures have been recorded (2, 3, 4).

The adequacy of the conventional design procedures was studied by instrumenting a bridge abutment to determine its behavior during and after construction. The instruments were installed on the west abutment of the bridge carrying the eastbound lanes of Interstate 80 over the Lehigh and Hudson Railroad near Allamuchy, New Jersey (Fig. 1). This bridge was chosen because it could safely be designed by conventional methods and, at the same time, provide measurable movements yielding data that would result in a clearer definition of the problem.

A profile including the soil stratigraphy at the Allamuchy site is shown in Figure 2. In general the profile includes approximately 30 ft of embankment over 14 ft of medium, compact silty sand and 44 ft of soft varved clay over very compact sand and gravel. Both the vertical and battered (1:3) piles penetrated approximately 35 ft into the sand

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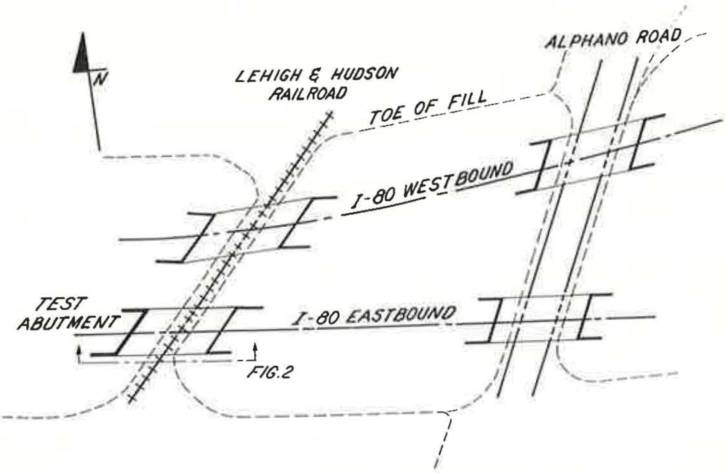

Figure 1. General plan of the Allamuchy site.

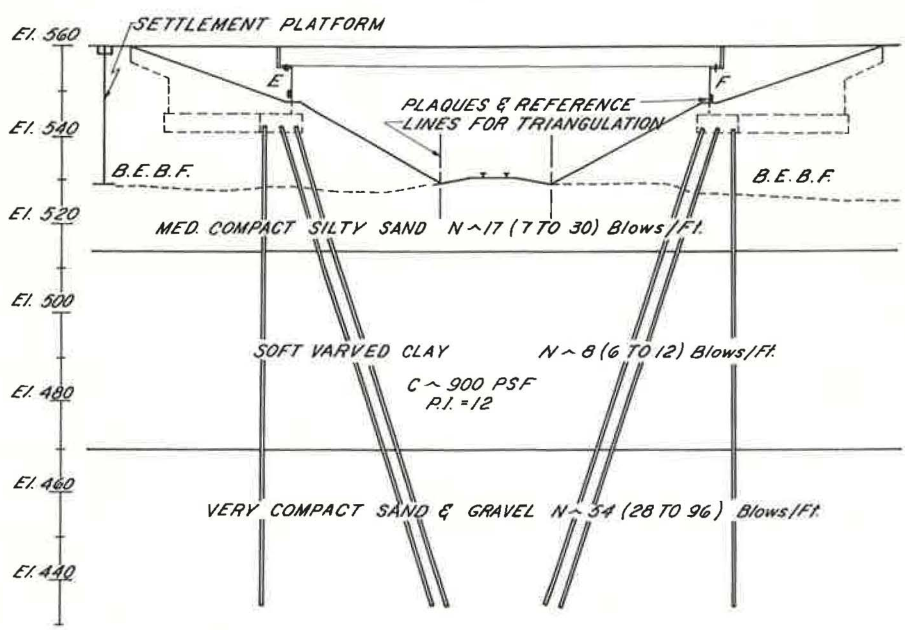

Figure 2. Profile and soil stratigraphy of the Allamuchy site.

and gravel. Steel H-piles (12 BP 53), using a design load of 28 tons per pile, were selected to minimize remolding of the varved clay (sensitivity = 3 to 4).

CONSTRUCTION PROCEDURES

Construction at the Allamuchy site started in May 1969 with the placement of settlement platforms and proceeded through July 1970 when the embankment construction was completed to subgrade elevation. The placement of the subbase and pavement has not yet begun. Additional measurements are planned after these operations are completed. For convenience, the significant events during the construction process have been divided into the following stages (Fig. 3):

0. May 1969, installation of settlement platforms on original ground;
1. July-August 1969, placement of bridge excavation borrow foundation, completion of pile-driving operations, completion of pile load test, and installation of Slope inclinometer guide casings;
2. September 1969, installation of stress and strain meters and pouring of abutment footing;
3. October 1969, completion of construction of abutments;
4. November 1969, placing of approximately 10 ft of embankment and erection of steel girders;
5. December 1969, completion of approximately 14 ft of embankment;
6. April 1970, pouring of bridge deck; and
7. July 1970, completion of embankment construction including all the fill at the east abutment of the test structure.

INSTRUMENTS AND MEASUREMENTS

Six of the piles, 3 vertical and 3 battered, in the test abutment (Fig. 4) were modified to permit measurements of their deflection under load. A 5-in. diameter steel pipe was fillet-welded to each pile through 3-in. slots cut at 1-ft intervals through the web at the neutral axis. The pipe was then cut between each fillet weld into 1-ft segments by means of a carborundum saw and an acetylene torch, so that each 1-ft segment was free to form a tangent to the slope of the deflected pile.

The openings in the pipes caused by slotting were filled with a mastic to prevent soil from entering during driving. The pipes terminated 5 ft above the H-pile tips to eliminate disturbance of the sand and gravel stratum into which the piles were driven. Disturbance of the varved clay, because of the increased cross section, was minimized by jetting with a 2-in. diameter jet pipe inside the 5-in. pipe, with the jetting discontinued when each pile had penetrated a few feet into the supporting sand and gravel stratum. Aluminum casings (3.18-in. outer diameter), supplied by Slope Inclinometer of Seattle, were installed inside the 5-in. pipes after the piles were driven. Ottawa sand filled the annular space between the casing and the pipe. Slope inclinometers (model 200 for

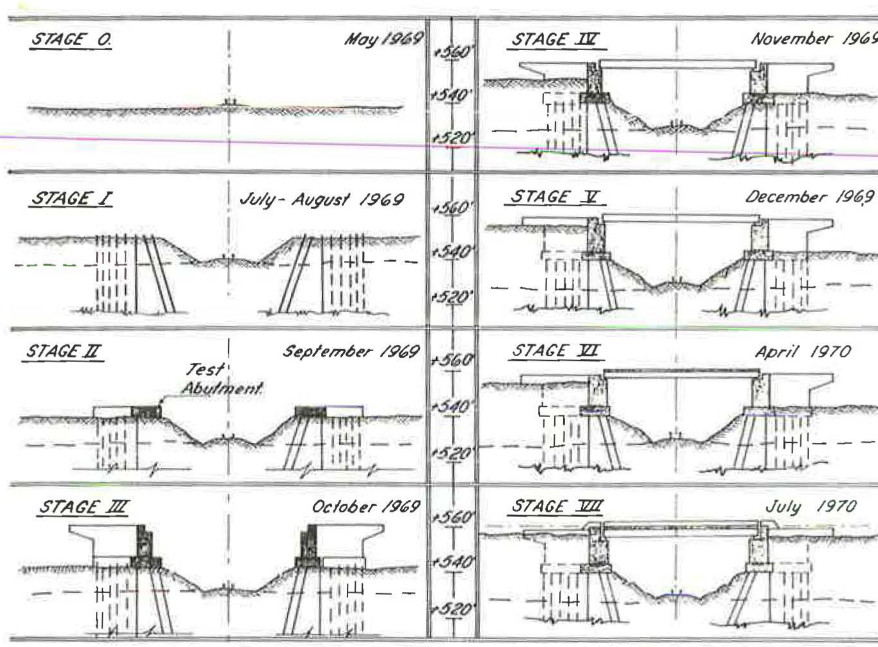

Figure 3. Construction stages.

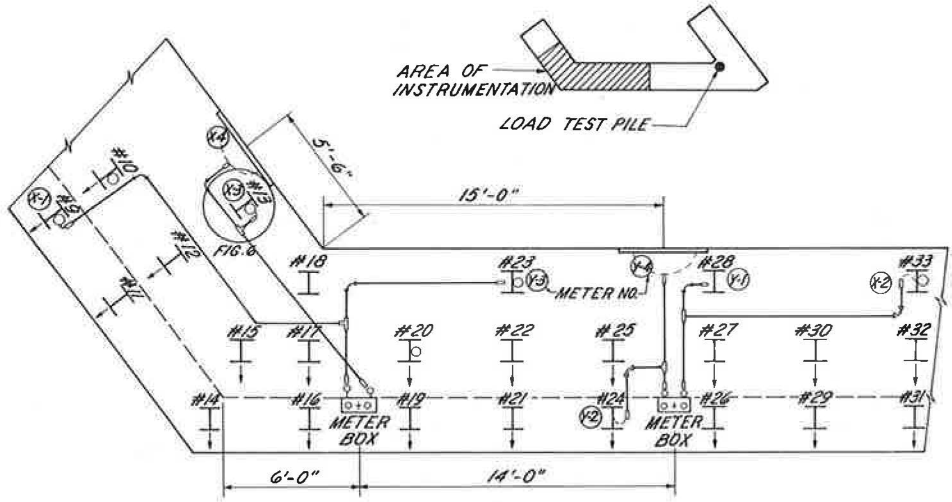

Figure 4. Plan of instrumentation layout.

the vertical piles, and model 225 for the battered piles) were then used to measure the deflection of the piles after they were driven and during their construction.

Each of the 6 piles in the test abutment (Fig. 4) were instrumented with 4 Carlson SA-10 strain meters, one at each edge of each flange. Two of these piles, 1 vertical and 1 battered, did not have Slope inclinometers. The purpose was to determine whether the jetting of the inclinometer-instrumented piles had diminished the pile support capacity of the in situ soils. All strain meters were installed prior to the placement of reinforcement steel.

The stress meter assemblies, consisting of 4 Carlson stress meters (Model PE-10) each, were installed at the back of the abutment and wing-wall pile cap (Fig. 4) to measure the lateral earth pressures. The 4 meters were held in position by a template fastened to the form while the pile cap was poured; and, after the forms were stripped, a 1-in. thick steel plate, measuring $2\frac{1}{2}$ by 4 ft, was placed flush against the surface of each group of 4 meters. Each plate was then suspended by wires fastened to bolts cast in the concrete cap. The plates were otherwise unfastened and free to swing against or away from the stress meters.

All cables from the Carlson stress and strain meters were run through Greenfield and $2\frac{1}{2}$-in. rigid metal conduit within the abutment and wing-wall footing and stem to protect the cables from damage during concreting operations. These cables were terminated in meter boxes, mounted on the face of the abutment, wherein the cables were fastened to Leeds and Northrup decade switches. The stress and strain meter measurements were made by connecting a Carlson testing meter assembly to the Leeds and Northrup switches.

Nine settlement platforms were installed behind the test abutment at approximately the original ground surface elevation, after topsoil and other organic matter were stripped. Settlements of the platforms were measured to 0.001 ft throughout the construction of the bridge by using 3 wire-leveling methods. Readings will be continued after the highway is open to traffic by means of protection boxes placed in the pavement and shoulders into which the settlement platform pipes have been extended.

In addition, several points on each west abutment at both the Lehigh and Hudson Railroad and Alphano Road were surveyed to observe settlements of the structures. Readings were begun on the test abutment immediately after it was constructed, while measurements to obtain correlative data were performed on the other abutments some time after they were constructed.

The horizontal distances between 2 opposing pairs of monuments (Fig. 2) embedded in the faces of the test abutment and the east abutment (eastbound) were measured by triangulating with theodolites reading directly to 1 second. One reference line and then a second one was established along the railroad tracks (Fig. 1) from which the triangulation was performed.

It was observed that noticeable movement had occurred at the test abutment rockers a short time after the steel girders were placed. Subsequent measurements were performed on the rockers at the west abutments of the Lehigh and Hudson Railroad and Alphano Road structures to determine the magnitude of horizontal movement occurring at the rockers. Measurements consisted of determining the horizontal distance between the rocker key centerline and the vertical plane passing through the rocker pin centerline.

TEST RESULTS

To aid in the reduction of the myriad of field measurements, 3 computer programs were written for this research project: Slope inclinometer data, which output deflection versus depth; Carlson strain meter data, which output values of strain and corresponding stresses; and Carlson stress meter data, which output values of pressure acting on each meter and a coefficient of lateral earth pressure.

By using slope indicator measurements, the as-driven shapes of the 6 inclinometer-instrumented piles were computed and plotted. For the vertical piles, there was very little drift in the direction of their strong axes. However, there was considerable deviation from the vertical, i.e., up to 5 ft in a 100-ft long pile, in the direction of weak axes of the piles. These deviations were similar to, and about the same order of magnitude as, those recorded by Hanna (5) for steel piles. In the batter piles, the same general trend to deviate from the design location was observed, but of smaller magnitude. The design batter of 1:3 was never achieved, probably because the piles tended to bend under their own weight, as noted by Bjerrum (6).

Measurements of the 6 piles were made at the various stages of construction described earlier, and the deflections were computed by using the as-driven shape as a "zero deflection" reference. Piles 20, 23, and 33 in the abutment indicated deflections occurring under increasing abutment and embankment loadings in the direction of the strong axes of the piles, away from the embankment. There was virtually no deflection under load in the direction of their weak axes. Piles 9, 10, and 13 in the wing wall indicated that deflections occurred about both strong and weak axes with the resultant of these deflections acting normal to the railroad. Figure 5 for pile 13 shows the deflections computed for the instrumented wing-wall piles. It was hoped that by measuring the deviations of the piles under load, from their as-driven shapes, deflection curves could be obtained from which the coefficient of lateral earth pressure, K, acting against the piles could be computed. Difficulties were encountered in selecting from these deflection curves the end conditions and the effective lengths of the equivalent beams of the piles on which such a coefficient would depend.

The strain meter measurements were used to compute the compressive and tensile stresses acting in the piles immediately below the point of embedment in the footing. These stresses indicated that bending occurred only about the strong axes of the H-piles in the abutment and about both axes of the H-piles in the wing wall. Figure 6 shows the stresses acting in pile 13, which is typical of the wing-wall piles. The bending moments of fixation, computed from the strain meter measurements, for the vertical piles averaged 41 kip-ft. The stresses induced by bending moments of similar magnitude will depend on the types and dimensions of the piles. However, it is evident that piles with low bending strength properties should be avoided.

Figure 7 shows the variation of the coefficient of lateral earth pressure with the construction stages. Before completion of state 4, the coefficient of lateral earth pressure computed from the readings of the abutment stress meter assembly was 0.32 and from those of the wing-wall assembly, 0.58. This high value was attributed to possibly abnormally high compactive effort adjacent to the wing-wall assembly. At the completion of stage 4 of construction, the K-value at both locations was 0.27. With no further increase in load for approximately 6 months, the coefficient of lateral earth pressure decreased to 0.13 and 0.22 for the abutment and wing-wall respectively.

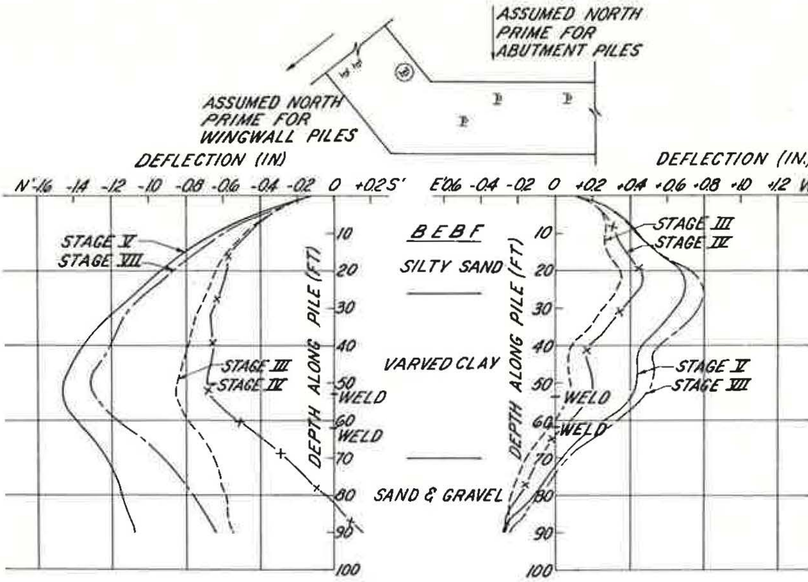

Figure 5. Deflections with time for vertical pile 13.

Increasing the embankment height to final subgrade in stage 7 resulted in further reductions in the K-value to 0.12 and 0.19 for the abutment and wing wall respectively. These latest reductions in K are quite small, probably because the varved clay consolidated under the weight of the embankment and abutment loads that remained relatively unchanged from November 1969 through April 1970 with a corresponding decrease in void ratio and increase in shear strength.

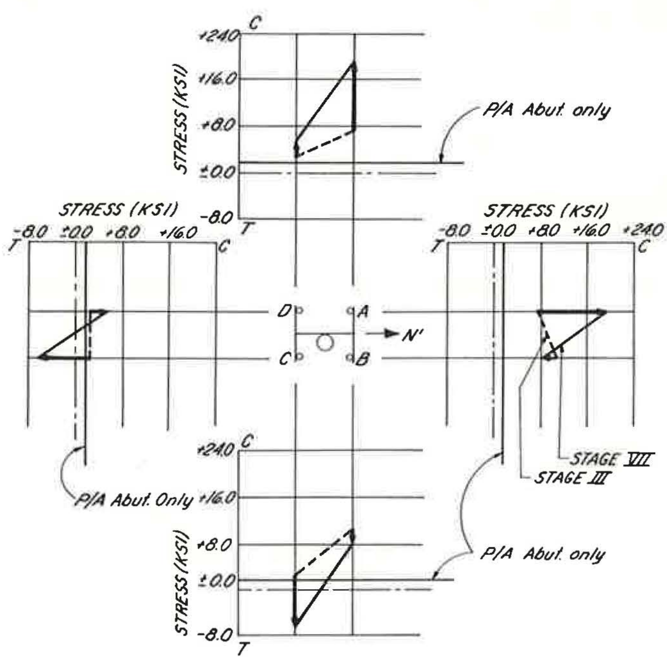

Figure 6. Bending stresses with time for vertical pile 13.

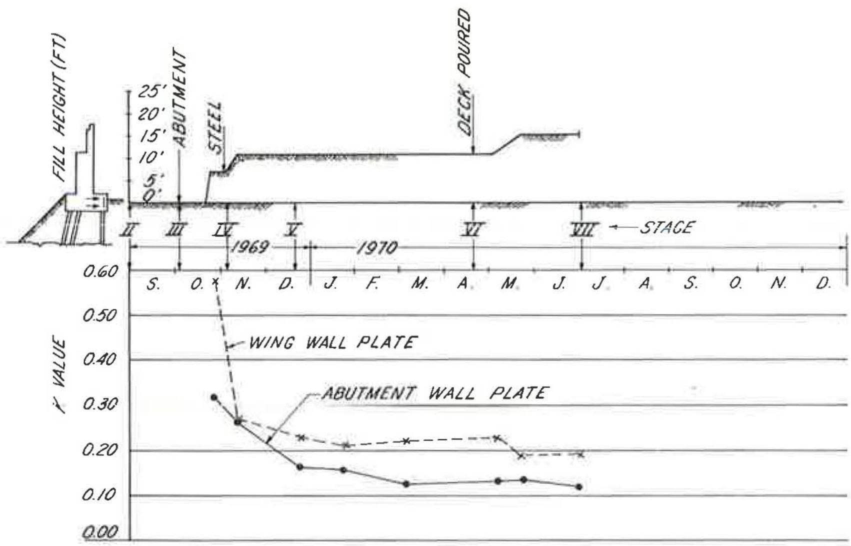

Figure 7. Coefficient of lateral earth pressure with time.

The settlement platforms behind the test abutment showed settlement of the original ground surface as indicated by the settlement curves shown in Figure 8. Several of the settlement platform pipes were accidentally disturbed, and this resulted in unreliable values for the total settlement although the trend of the settlement values remained valid.

Settlement readings on the top of the test abutment were much more conclusive. Plots of settlement for points 13 and 14 at the end of each wing wall at the test abutment, also shown in Figure 8, yielded curves corresponding to the settlement plots for the adjacent settlement platforms. Readings taken on the other abutments showed similar qualitative settlements.

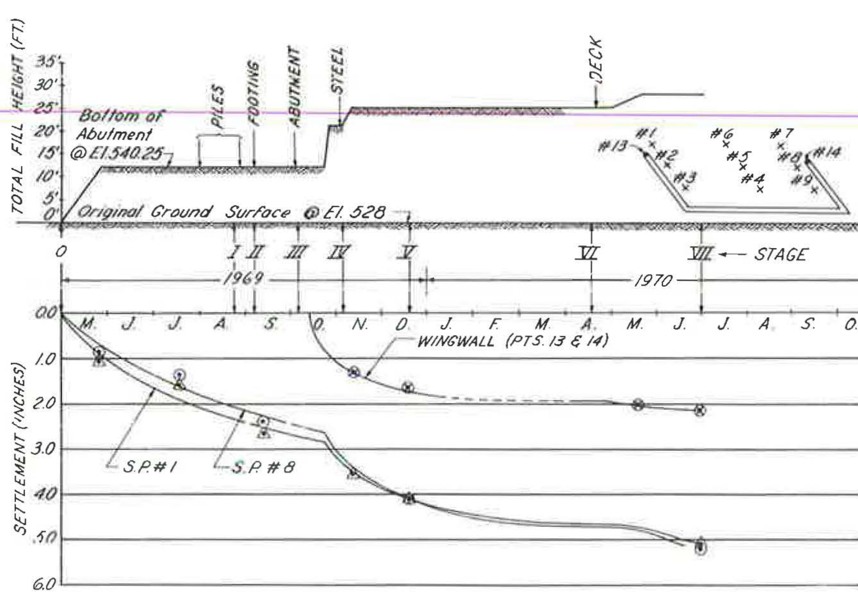

Figure 8. Settlements with time.

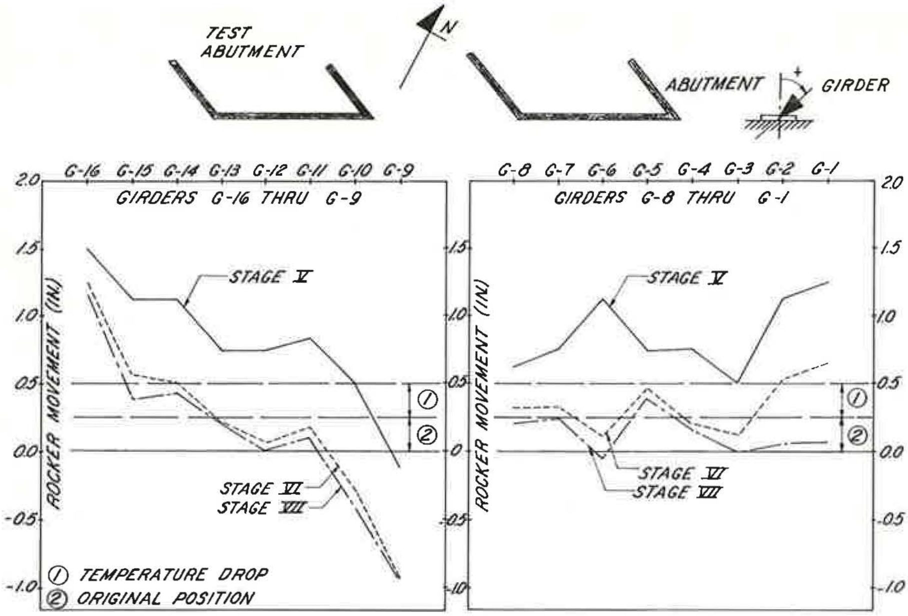

Figure 9. Rocker tilts with time.

Results of the triangulation between the abutments to determine the horizontal movement were disappointing. A small amount of tilting of the abutment back into the embankment was recorded before one of the reference line monuments used for these measurements was disturbed. Subsequently, 2 new reference lines were established with more monuments only to be destroyed by someone unfamiliar with the work.

Measurements of the expansion rockers of the west abutment at the Lehigh and Hudson Railroad are shown in Figure 9. Parts of the tilting shown is a result of contraction caused by thermal changes and the initial tilt of the rockers when set. The remainder of the tilting is attributed to movement of the abutments in both the eastbound and the westbound bridges. Because the abutments had fixed bearings, any movement at these abutments would be reflected as tilting at the expansion rockers of the west abutment. Examination of data shown in Figure 9 reveals that there was rotation of each abutment about a vertical axis. The portion of each abutment closer to the median, where there was an appreciable height of embankment, moved away from the embankment. The portion of each abutment closer to the outside embankment slope tilted back into the embankment. Movement at girder G16 (Fig. 10) at the test abutment was the maximum tilting observed.

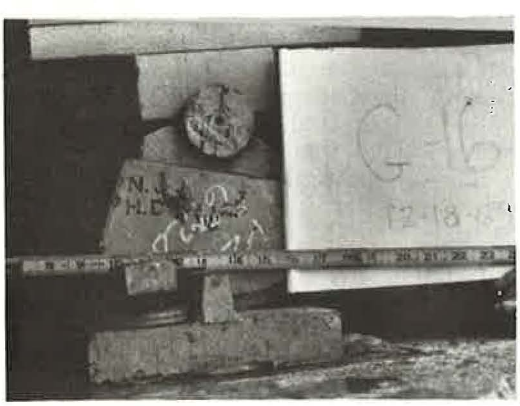

Figure 10. Rocker tilt at girder G16, December 1969.

DISCUSSION OF RESULTS

Results of Present Investigation

A decrease in lateral earth pressures against the abutment and wing wall was observed with a corresponding decrease in the lateral coefficient of earth pressure from 0.32 to 0.12 and 0.58 to 0.19 for the abutment and wing wall respectively. The decrease of these pressures was concurrent with the increase in settlement of the backfill behind the abutment. In addition, this rotation of the abutment about a vertical axis caused the stress meters to move toward the embankment. If the abutment had not rotated in this manner, it is likely that a larger decrease in the coefficient of lateral earth pressure would have been observed.

The vertical H-piles under the heel of the abutment were subjected to bending stresses due to lateral pressures of the consolidating clay layer, the tilting of the abutment, and the rotation of the abutment about a vertical axis.

The dual effect of the decreased lateral pressure against the abutment and of the increased lateral pressure against the supporting piles could be visualized as a couple acting in a vertical plane, which induces a tilting of the abutment toward its backfill. This is confirmed by the excessive tilting of the rockers supporting the bridge girders and by the large relative settlements of the wing walls (Fig. 8 and 2, Figs. 27 and 28). The assumption is that the supporting piles at the end of the wing walls, which were not instrumented, have yielded, transferring their load to the wing-wall footings.

The beginning of the pronounced tilting of the abutment coincided in time with the construction stage when the shearing stresses within the underlying plastic clay layer, due to the weight of the backfill, started to induce localized plastic flow and deformation in the weaker zones of the clay. This apparently established a maximum limit value for the applied surface load equal to 3C (C = cohesion) to prevent backward tilting of similar abutments (2). Establishment of this upper limit is also supported by the fact that no excessive tilting of the rockers or of the abutments was detected at the adjoining bridge over Alphano Road where shearing stresses and shearing stress gradients in the underlying clay layer were smaller because of a lower, balanced, and more uniform height of embankment (2, Fig. 7).

A 5-month winter pause in the placing of the backfill behind the abutment induced a sufficient increase in the shearing strength of the clay due to its consolidation and accelerated by its varved nature so that no appreciable additional movements occurred when the filling was completed in the late spring of 1970. This emphasizes the practical importance of preloading as a design tool.

Recommendations for Future Investigations

The Carlson stress meter readings were found to be fully reliable. Provisions were made to facilitate readings of stresses due to lateral pressures on the abutment and wing wall after the project is opened to traffic. This would establish whether the decrease of the lateral pressures is permanent or will vary during the use of the bridge.

Readings on the Carlson strain meters, located at the top of the piles just below the pile caps, were reliable and consistent with the behavior of the abutment. For a more complete interpretation of the behavior of similar abutments in future investigations, strain-measuring devices should be installed along the pile length and stress-measuring devices should be installed at the lower surface of the pile cap.

Wilson inclinometers gave reasonable results that were in qualitative agreement with the strain meter readings on the H-piles. However, by comparison to the strain readings, the deflection readings were low, possibly because of the yielding and possible loss of the Ottawa sand filling the annular space between the inclinometer guide casing and the outer steel pipe. Use of miniature inclinometers and improved connections between the inclinometer casing and the pile are essential for future work.

Accurate triangulation of abutment displacements is also important in establishing the movement of the pile butts and, therefore, the true deflection curves.

CONCLUSIONS

The results obtained from these field measurements have established that abutments supported on piles driven through soft clay layers tilt toward the backfill when the shearing strength of these deposits is exceeded because of the superimposed embankment loads. An upper limit has apparently been established on the magnitude of the applied surface load (limit value of 3C) beyond which excessive abutment tilting will occur. The magnitude of the tilting has been found to be most sensitive to the shear strength of the deposit and loading rate and also dependent on the shear stress gradient and pile locations.

Preloading of soft foundation soils should always be considered, even for pile-supported abutments. The amount and time of surcharging will be governed by the consolidation and strength characteristics of the compressible layers.

Piles with adequate resistance to bending should be used to support abutments when compressible layers have not been previously stabilized. These piles should be of the nondisplacement type to minimize remolding of the clay and the resulting decrease of its shearing strength. Steel H-piles or pipe piles driven open-ended would meet this requirement.

Use of "spill through" types of abutments could be made in cases where the direction and magnitude of lateral forces acting against the abutment and wing walls are undetermined because of consolidation of the foundation soils.

ACKNOWLEDGMENT

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DISCUSSION

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The authors are to be complimented for a very lucid presentation of a complex topic. Also, I would like to thank them for their gracious acknowledgement of my connection with this research project.

A precise determination of the actual lateral pressures against the piles under the abutment did not prove possible, but the order of dimension thereof seems to emerge fairly clearly and indicates to me the need for a downward revision of the values roughly "guesstimated" by me in 1962 (1, p. 492). No such measurements on piles had as yet been made then.

The purpose of this discussion is to indicate and to justify these revisions by the measurements reported in the paper.

In 1962 I had suggested assuming a triangular lateral loading of the pile along the depth of the clay layer with a maximum pressure p_H at the center of the layer.

$$p_H = 2bk\,(\gamma H) \qquad (1)$$

where

b = width of pile;
(γH) = weight of fill behind abutment in respect to original ground; and
K = 0.4 = consolidated equilibrium lateral earth pressure coefficient.

The doubling of the width b in Eq. 1 was based on the results of model tests with sand (1, pp. 466 and 514). This does not now appear necessary in sensitive clays where some slight remolding is possible even around H-piles.

The results obtained at Allamuchy suggest the following changes to Eq. 1:

$$P_H = \beta b K \delta_z \qquad (2)$$

where

β = stress concentration coefficient, taken as 1.00 for Allamuchy;
δ_z = vertical stress at point where p_H is determined;
b = width of pile; and
K = 0.4 = consolidated equilibrium lateral earth pressure coefficient.

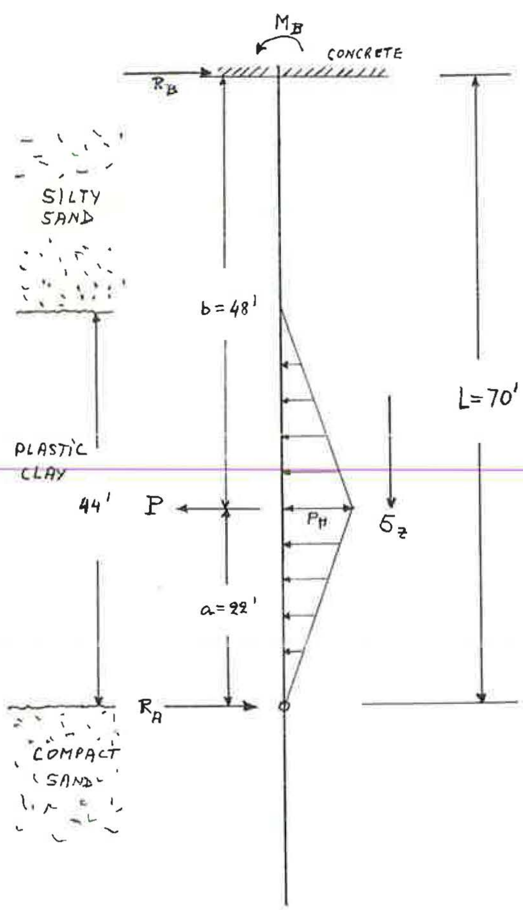

Figure 11. Dimensions for analysis of measurements at Allamuchy site.

Because at Allamuchy (γH) = 3.1 ksf and δ_z = 1.25 ksf, the use of Eq. 2 instead of Eq. 1 means an almost fivefold reduction in the numerical values of p_H.

Figure 11 shows the dimensions relevant to the following analysis of the Allamuchy measurements. If we replace the triangular loading by a concentrated force P and assume complete fixation of the pile in the concrete footing and a hinge at the upper surface of the compact sand layer under the clay, structural handbooks give the following values for the moment of fixation M_B at the upper support and for the field moment M_m:

$$M_B = -\frac{P\,a(L^2 - a^2)}{2\,L^2} \qquad (3)$$

$$M_m = +(P\,a/2)(2 - 3a/L + a^3/L^3) \qquad (4)$$

For the values, a = 22 ft and L = 70 ft shown in Figure 11, we obtain M_B = -9.35 P and M_m = +10.30 P = 1.16 M_B.

As stated in the paper, the bending moment of fixation for the vertical piles averaged 41 kip-ft. This value was computed from readings of Carlson strain meters, the centerline of which was 4.5 in. from the neutral axis of the 12-in. H-pile. The extreme fiber strain corresponding to the actual bending moment would, therefore, be 6.0/4.5 = 1.33 times larger than the one recorded. Further, because the strain meters were located about 1.5 ft below the bottom of the footing, the bending moment of fixation would be at least 5 percent larger than the one at the location measured. This gives M_B = 41 × 1.33 × 1.05 = -57.5 kip-ft and M_m = +57.5 × 1.16 = +66.7 kip-ft.

If we attempt to compute the bending moment M_m from the maximum 0.3-in. deflection along the 44-ft deep clay layer determined from inclinometer readings, we obtain a maximum value of M_m = 12.1 kip-ft, which is 4.75 times smaller than the average moment of fixation M_B of the pile at its upper end. This result cannot be reconciled at present with any logically conceivable pressure redistribution pattern, which would fit other data from this job, but may be attributed to some yielding at small deflections of the imperfectly compacted Ottawa sand that filled the space between the steel pipe welded to the H-pile and the aluminum guide casing for the inclinometers.

The following computations will, therefore, be based solely on the fixation moment M_B = -57.5 kip-ft determined from the Carlson strain meters because it cannot be too large and may even correspond to only partial fixation. The H-piles had only the customary 1.0-ft embedment in the 3,000-psi concrete of the footing. The compressive stresses in the concrete along the vertical faces of the H-pile flanges necessary to balance the bending moment M_B = -57.5 kip-ft will equal f_c = 2,400 psi at the edge or 80 percent of the 28-day compressive strength. Some yielding and plastic flow of the concrete may have, therefore, occurred.

We will use M_B = -57.5 kip-ft but, as a first approximation, compute from Eq. 3: P = 57.5/9.35 = 6.15 kips. Because an equivalent triangular load will produce a bending moment of fixation at least 10 percent smaller than the value obtained from Eq. 3 for a concentrated load, we find the ordinate p_H of the triangular load (Fig. 11) from p_H = (6.15 × 1.10 × 2)/44 = 0.307 kip = ft. Inserting this value, b = 1.0 and δ_z = 1.25 ksf (2, Fig. 29) into Eq. 2 and solving for K, we obtain K = 0.245. In view of various uncertainties involved, it will nevertheless be prudent to continue using K = 0.40 in Eq. 2.

In this connection, one should note the evidence shown in Figure 8 that piles under the extreme tips of the wing walls started to buckle. An extreme fiber stress in bending of some 30,000 psi, when added to stresses produced by axial loading, should be sufficient to initiate buckling. The piles under the skewed wing walls project a width of 1.35 b on plane paralled to main abutment. If we distribute the lateral pressure on that width to the major and minor principal axes of a pile in proportion to their moments of inertia, we find that a value of K = 0.24 corresponds to an extreme corner stress of 38,000 psi and the triangular loading of Eq. 2. These piles at the ends of the wing walls are more exposed to possible stress concentrations than piles 13 and 9 on which measurements were made under the wing walls.

Of course, the possibility cannot be excluded that further measurements and studies may reveal a much more complex and hitherto unsuspected pattern of pile-soil interaction.

A. A. Seymour-Jones, Howard, Needles, Tammen and Bergendoff

The authors are to be complimented for having presented some very interesting information on a neglected subject. It is hoped that this paper will generate further studies of this problem because it can and has occurred in many areas where compressible foundation soils exist.

The writer agrees with the authors that the best prevention for this type of problem is preloading or surcharging the abutment areas. However, there have been cases where schedules or other factors do not permit adequate treatment of the foundation areas, so the engineer has to face this type of condition.

The writer has encountered 7 cases of similar abutment movements and obtained rough measurements of the movements involved. These data have provided a basis for evaluating similar problems and are given in Table 1.

The data given in Table 1 provide a basis for estimating possible abutment rotation for similar problems provided a reasonable estimate of the post construction settlement

TABLE 1

SUMMARY OF ABUTMENT MOVEMENTS

Structure	Foundation	Fill Settlement (in.)	Abutment Settlement (in.)	Abutment Tilting (in.)	Ratio of Abutment Tilting to Fill Settlement
1	Steel H-piles	16	Unknown	3	0.19
2	Steel H-piles	30	0	3	0.10
3	Soil bridge	24	24	4	0.17
4	Cast-in-place piles	12	3.5	2.5	0.19
5	Soil bridge	12	12	3	0.25
6	Steel H-piles	48	0	2	0.06
7	Steel H-piles	30	0	10	0.33
—[a]	Steel H-piles	5	0.4	0.5 to 1.5	0.10 to 0.30

[a]Described in paper.

magnitude can be made. Based on these data, the writer would recommend a ratio range between 0.25 and 0.33.

Problems associated with abutment tilting can be provided for by the following steps:

1. Instead of rockers, use sliding plate expansion shoes with sliding plates large enough to sustain the anticipated horizontal movements;
2. Make provisions to fill in the bridge deck expansion joint over the abutment by inserting either metal plate fillers or larger neoprene joint fillers;
3. Design piles for drag forces due to settlement; and
4. Use steel H-piles for the abutment foundation.

Unfortunately there is insufficient information to calculate the bending forces induced in the piles by the abutment rotation at this time. It is hoped that further research will provide a guide in the future. Because all the abutments (Table 1) founded on piles are performing satisfactorily, bend stresses in the piles may not be a major problem. Still it would be prudent to use steel H-piles in such cases because they are capable of taking large tensile stresses without failing.

The writer has had access to the report (2) referred to by the authors. Two facts noted are of interest. The driven piles deviated appreciably from a straight line. The measured stresses in the 6 instrumented piles varied considerably from the design stresses. It is the writer's conclusion that for these 2 reasons conservative design-allowable pile loads should be used for structures of this type.