Deep Silt Consolidated by Surcharge Fill on Patapsco Tunnel Approach

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The consolidation of deep organic silt deposits under highway embankments for toll plaza and approach roads to the Baltimore Harbor Tunnel by the use of surcharge fills alone is discussed. Field and laboratory investigation, settlement computations and design studies, together with field observations and controls during construction, are presented. Predicted and actual settlements are compared in magnitude and in rate.

It is concluded that surcharged fill without sand drains is practical in some instances and that reasonably rapid consolidation may be achieved by the method. The methods of settlement analysis developed in recent years based upon Terzaghi's theory of consolidation of soil are directly applicable to practical problems and give results as dependable as the accuracy of normal sampling of the soils warrants.

• ORGANIC SILT deposits up to 60 ft deep beneath 30-ft embankments were consolidated by the use of surcharge loading on the approach to the Baltimore Harbor Tunnel. This paper presents a record of the investigation, design, and construction performed on a section of this project. Predicted and actual settlements are recorded and methods and equipment are described.

The Patapsco River tunnel project includes the tunnel proper passing beneath Baltimore harbor, and six additional design sections covering over 13 mi of approach roads. Design section D-3 extends west from the depressed ramp at the southern end of the tunnel a distance of about 1½ mi. It includes the toll plaza and six bridges. This section is elevated 25 to 40 ft above the adjacent area.

Borings showed a layer of silt 45 to 65 ft in thickness underlying surface sand and gravel beds beneath the toll plaza and much of the roadway fill. After study of various methods of treatment a 20-ft surcharge was used to consolidate the silt beds. Treatment appears to have been successful and the area has now been paved with rigid pavement.

FACILITIES AND SCOPE

Design section D-3 includes six bridges, one of which is a 2-span continuous K truss about 1,000 ft long, and a 14-lane toll plaza 1,300 ft long and 280 ft wide, all elevated about 30 ft above the surrounding area with side slopes at two to one. Earthwork on design section D-3 was about two million cubic yards of fill and almost no cut at all. The approach road and toll plaza have now been paved with rigid pavement. Many problems were encountered in placement and compaction of this fill over a period of about a year and a half. It is believed to be an excellent example of a well constructed embankment and consolidation within this mass should cause no further concern. This paper is directed entirely toward the problem of consolidation of geological marsh deposits which lie below the fill and will present no data or discussion regarding placement of the fill itself.

GEOLOGY AND SOILS

The site is on the south side of the Patapsco River, an estuary of the Chesapeake Bay. The site rests upon Pleistocene sediments, which in this area are known as the "Talbot" formation. This formation borders the Bay and its estuaries and was apparently formed by deposition of sediments at the mouth of the rivers during the last submergence of this area. The formation is composed of a heterogeneous mass of clay, sand, gravel, silt, and cobbles. It is 50 to 80 ft deep at the site. The silt deposits were probably laid down during this last submergence of the area by the Talbot sea. At the base of the deposit are definite accumulations of grassy peat covered by many feet of river silt, which in time were covered by thick deposits of sand and gravel.

Below the Talbot formation at the site lies the Potomac group which is composed of irregular alternations of sand, gravel and clay. This group rests upon the crystalline rocks, about 300 ft deep at this point, dipping steeply toward the Atlantic Ocean to the east.

INVESTIGATION

Borings and Soil Profile

About 80 rig borings were taken along the right-of-way. They ranged in depth from 30 to 150 ft and in all cases were taken into material having a penetration resistance of well over 100 blows per foot by the so-called "Standard Gow Test." Borings were made with both 2½- and 4-in. casings. Penetration tests were taken every 5 ft in conjunction with ordinary dry samples except in cases where undisturbed samples were taken.

Most of the borings were taken at the bridge sites which were spaced at frequent intervals along the section. Intermediate borings were spaced to supplement those at the bridges, which were at the beginning and end of each section of the embankment. Figure 1 shows the borings and the soil profile which was developed from them. It shows not only the original soil profile, but also the profile of the surcharge and the finished roadway. The greatest body of the silt material occurs directly beneath the toll plaza which also contains the greatest volume of embankment. The paved toll plaza has a width of 280 ft and a length of 1,300 ft. Borings at the railroad bridge, the toll booth, and the Childs Street bridge were supplemented by intermediate borings as shown in the figure.

The existence of the deep silt stratum, which had not been discovered in the preliminary report stage, was evident after the first few borings were taken in the vicinity of Childs Street bridge. The material had a depth of 60 ft beneath the west abutment of this bridge but had completely disappeared beneath the east abutment. The deposit extended westward about 4,000 ft maintaining its 60-ft depth the full length of the toll plaza and then thinning westward to about 45 ft at the railroad bridge and to about 15 ft at Frankfurst Avenue. It appeared under all three piers of the K-truss bridge and disappeared abruptly beyond. A cross-section of the deposit was traced for about 1,000 ft beneath the relocated Frankfurst Avenue and the Shell Road ramp (Fig. 2). The materials reappeared in some areas east of Childs Street in depths of 10 to 20 ft where the roadway fill slopes from the bridge downward toward the tunnel entrance. About 20 ft of sand and gravel soil overlay the silt body in most cases except for a distance of about 1,000 ft where this material had been removed commercially for sand and gravel. In this pit area removal had been made to within 5 ft of the silt or approximately to the river level, leaving a blanket of gravel covering the silt.

Sampling and Testing

Undisturbed 3½-in. diameter samples were taken throughout the depth of the silt body in at least one boring in each group. A total of 28 of these samples were taken and consolidation tests were made on 12 of them. As the progress of design demanded immediate answers, most of the consolidation tests were loaded only 8 hours per load increment. However, as the design progressed some of the tests were loaded in 24-hr increments to confirm the results obtained. Atterberg

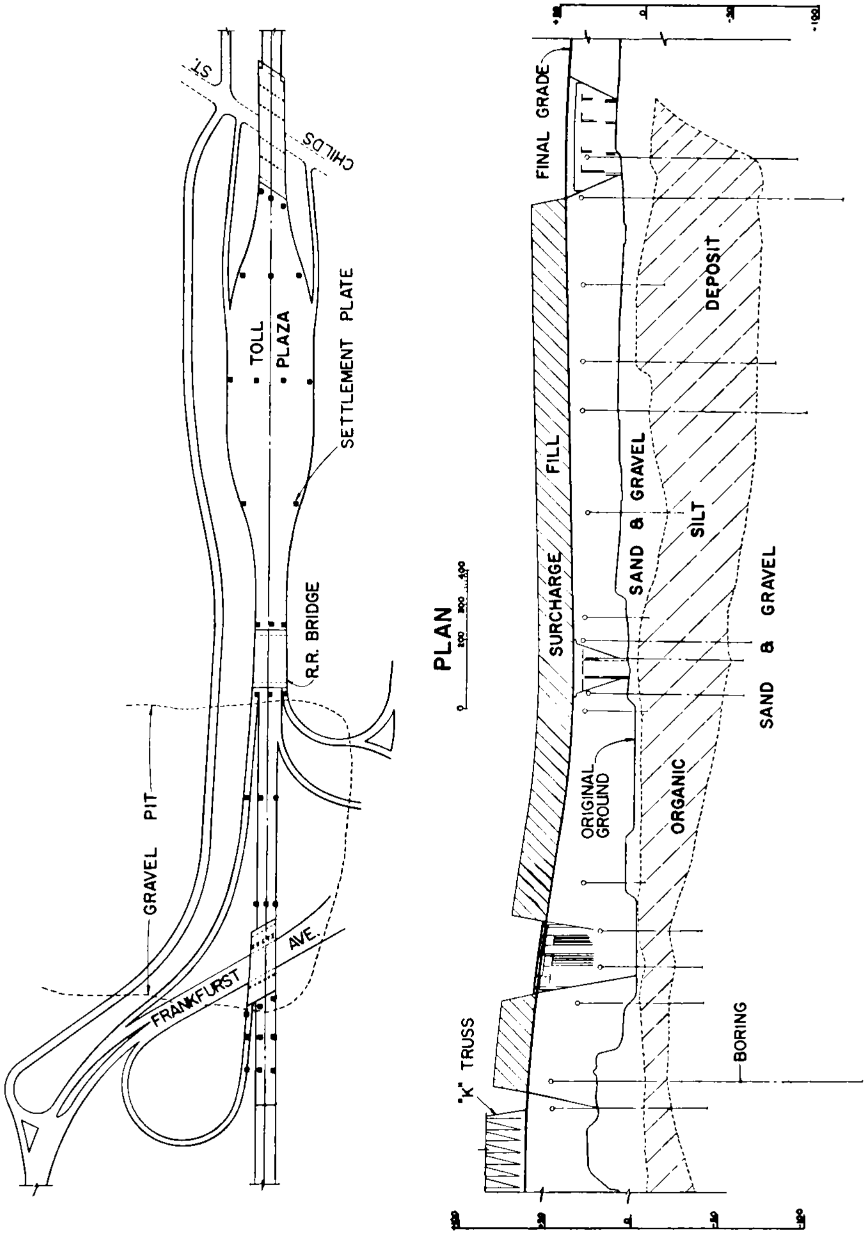

Figure 1. Soil profile and plan of Section D-3, Patapsco Tunnel approach highway.

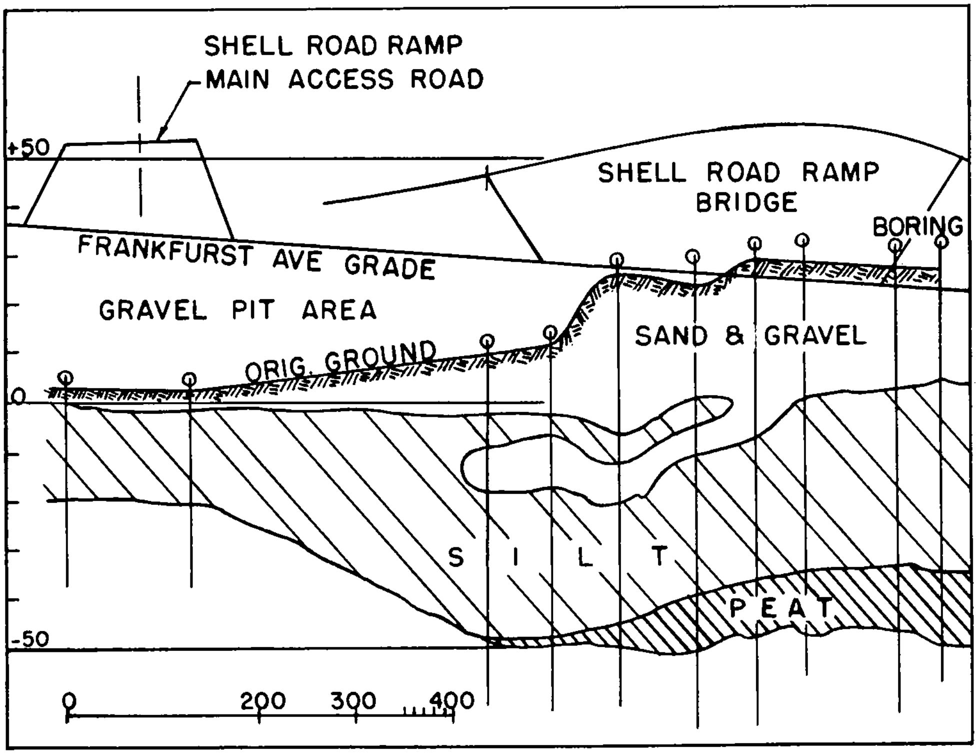

Figure 2. Soil cross-section at Frankfurst Avenue interchange.

limits and grain size tests were made on the samples including hydrometer analyses in order to compare the material as closely as possible with the normal dry samples and a few 2-in. Shelby tube samples which were obtained from the smaller diameter holes for void ratio comparisons and other tests. Standard penetration tests (blows of a 140-lb hammer falling 30 in. on a 2-in. o.d. sampler) were taken every 5 ft in all holes. In general the penetration resistance of the silt body was in the 3- to 5-blow range.

Throughout the extent of this large silt body a great similarity of material was evident. The material appeared to be definitely silty in nature, as opposed to clayey, and contained a large amount of organic material of a fibrous or grassy nature. Some lenses of grassy peat were found in the deposit. These offered a penetration resistance of 8 to 15 blows per ft. Some sandy lenses were also present.

The tests of the basic material indicate that it is a low plasticity soil having an organic content of from 6 to 9 percent. The hydrometer tests show that with a rare exception containing some sand, 100 percent of the material passes the No. 200 sieve but that only about 20 percent passes the 0.002 mm size. The liquid limit is generally between 35 and 50 and the PI is between 7 and 18. By the Unified Classification system this material falls in the OL group or by the HRB classification about A-7-5 (12). Void ratio of the material was in the range of 1.2 to 1.5 in most cases. The average preconsolidation load shown by the consolidation tests, using Casagrande's method (*1*) was about 4 tons per sq ft. Table 1 gives basic soil properties.

ANALYSIS

The already established toll plaza and roadway design required that a fill 35 ft

TABLE 1
TEST RESULTS, BASIC ORGANIC SILT DEPOSIT [1]

Organic content	6-9%
Grain size, by hydrometer test:	
Minus No. 200 sieve	100%
Minus 0.002-mm size	20%
Plasticity:	
L.L.	35-50
P.I.	7-18
Classification	Unified OL; HRB A-7-5(12)
Void ratio	1.2-1.5
Preconsolidation loading	4 tons/sq ft
Standard penetration test	3-5 blows/ft

[1] Some peat (fibrous, 8 to 15 blows); sandy silt lenses.

in depth be placed above the silt deposits previously described.

Stability

The first consideration was the stability of the material against displacement due to shear failure under the load. The material was a preconsolidated deposit having a penetration resistance of 3 to 5 blows per ft and lay generally beneath a 20-ft blanket of sand and gravel. The sand and gravel had been removed to about 5 ft in the gravel pit area, however.

In order to obtain an immediate answer so that design could proceed before testing was completed, certain conservative assumptions were made. A fill 60 ft in height above the water table was assumed to be resting on a blanket of 5 ft of gravel on 60 ft of silt, all below the water table. This would represent about 25 ft of surcharge fill above the road fill in the gravel pit area. Assuming an angle of shear of 20° (*2*) and a cohesion of 200 lb per sq ft, the factor of safety against shear failure was investigated at two points below the side slope, following very closely the Spangler method (*3*), and was found to be about 1.75. This value dropped to about 1.7 if cohesion was neglected or to about the same if $\phi = 17°$ and cohesion was assumed to be 200. Later unconfined compression tests confirmed this to be amply conservative and no further studies were made relative to stability against shear failure. The specifications required that the fill be placed in layers over the full width of the roadway cross-section. The surcharge was on the fill for nearly a year and removed with no evidence of failure of this nature ever being observed. In the adjacent contract the excavation for the depressed ramp to the tunnel portal passed through lenses of similar geologically preconsolidated silt. A 10-ft unsupported face of this silt stood beneath a layer of about 6 ft of sand and gravel for a short time during construction and showed no tendency to flow or squeeze out.

Settlement

Having established the stability of the material against shear failure the problem was then one of settlement. As soon as the first consolidation test results were received, settlement computations were initiated.

Computations were made first for three points along the line. These were at Childs Street bridge west abutment, the center of the toll plaza, and at the railroad bridge west abutment. Consolidation tests on undisturbed samples were first available from these points. Figure 3 is a diagram of the typical computation set-up. A firm layer of relatively clean sand underlay the silt body in all cases, insuring a double drainage situation. Settlement under the proposed fill was indicated at the three points, respectively, to be 1.78 ft, 1.31 ft, and 1.84 ft. The time indicated for 90 percent settlement was 1.84 years, 1.04 years, and 1.12 years, respectively. Table 2 summarizes settlement values computed and measured at several points.

The study was continued to shed further light upon these values, which ap-

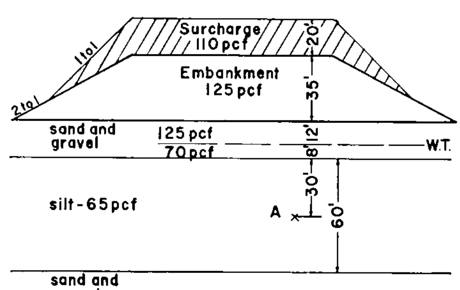

Figure 3. Typical diagram for settlement computations.

TABLE 2
SETTLEMENT OF EMBANKMENT

Location	Hole No.	Computed Settl. (ft)	Computed $T_{90\%}$ (yr)	Computed T_{20Ft} [b] (yr)	Measured [a] Plate	Measured [a] Settl. (ft)
Childs St. abut.	29	1.78	1.84	0.43	25	1.72
Av. curve	29	1.56	1.39	0.74	27	1.10
Toll Plaza, central	27A	1.84	1.12	0.54	19	1.12
					20	1.38
					21	1.45
R.R. br. abut.	22	1.31	1.04	0.44	9	1.60
					10	1.30
					11	1.88
Frankfurst br. abut.	20	1.10	0.25	0.11	1	0.93
Av. curve	20	0.92	0.21	0.09	3	1.25

[a] Some observed settlement was lost by some of these plates being broken at different times during the critical period. Values shown are minimum rather than adjusted.
[b] For 20-ft surcharge.

peared low, both in magnitude and time of settlement. Computations were made trying the probable variations of test values in an attempt to obtain maximum possible values and ranges of variability for settlement and time factors.

The proposed fill imposed an average load at the midplane of the silt body in the vicinity of 4 tons per sq ft. This is about the average obtained for the apparent preconsolidation load obtained from the consolidation test curves or, perhaps, in the vicinity of the geological load once applied to this silt layer. This would help to explain why the predicted consolidation was as small as indicated for so great a depth of silt.

The primary settlement computations were made assuming each test hole to be at the center point of the fill cross-section. They entirely neglected cross-sectional deviation and the influence coefficient for the proximity of the toe of the fill slope. Maximum values only were sought at that time. Further studies were subsequently made to determine the differential settlement across the toll plaza and at other critical points.

Surcharge

The problem of treatment had to be decided at once in order to meet the design schedule. Only 6 to 9 months were available for a consolidation period prior to paving even considering that the contractor would be asked to bid on an apparently impossible time schedule for placement of the fill. This placement period would extend through a winter and require any surcharge fill to be in place above a completed compacted fill by the following March. Sand drains and other measures to accelerate consolidation were considered. Sand drains in an area of this extent and depth are to be avoided from an economic standpoint if at all possible. The apparent small magnitude of settlement and relatively rapid drainage characteristics demonstrated by the material in the consolidation tests indicated that a simple surcharge might do the job. After computations were made to study the effect of surcharges of different heights it was decided to place a 20-ft surcharge over the entire area and, if observed settlement was not rapid enough, to place sand drains in the vicinity of the bridge abutments later. Sand drains were not used on the job.

Investigation of the effect of surcharge loads was simply an extension of the settlement computations. Settlement and time required to achieve a settlement equivalent to that predicted for the simple fill were computed for surcharges of 10, 15, and 20 ft. Surcharge depth could then be plotted against time to achieve a given settlement, in this case, 100 percent of theoretical settlement for the final embankment (Fig. 4). Twenty feet was required to keep the time of consolidation within 6 months. This quantity of material could be used as fill in another area without waste. It was, therefore, chosen as the surcharge to be

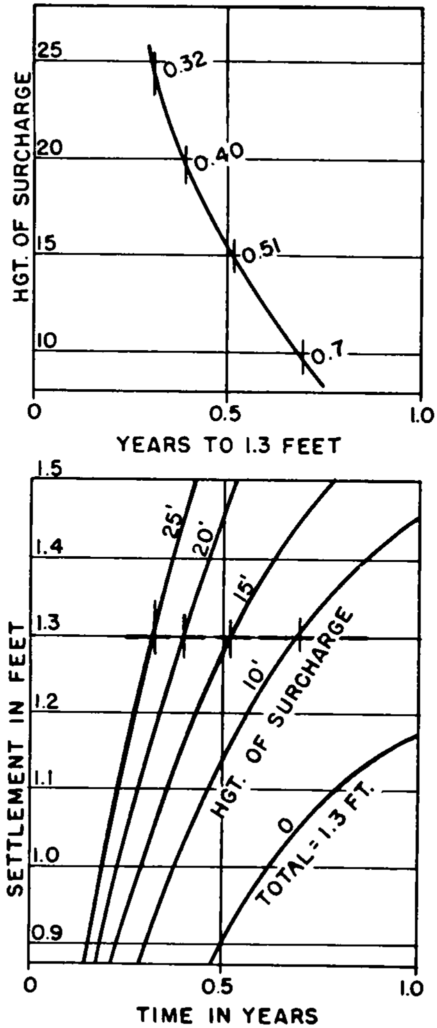

Figure 4. Time-settlement-surcharge curves, railroad bridge area.

used. Total theoretical settlement under 20 ft of surcharge averaged about 1.8 times that computed for the basic embankment above. It considerably exceeded the preconsolidation load, and settlement was greatly accelerated.

Special studies were made to determine the effect of the configuration of the surcharge slope to be used. It was desired to achieve as nearly as practicable complete settlement under the entire pavement and shoulder areas, yet practical considerations were involved. If the normal 2 to 1 slope were maintained 20 ft above the edge of the fill, 40 ft on each side of the fill would not be fully surcharged. It was proposed to make the berm of the surcharge fill immediately above the edge of the shoulder and to allow it to spill over as nearly as possible on a 1-to-1 slope. Again this required considerable additional material. Considering that the material to be consolidated was actually about 55 ft below the base of the surcharge, it appeared that the only requirement might be to place a given yardage above the fill, assuming that the weight of the supported load would spread to about the same pressure on the silt layer below. To investigate these conditions specifically, settlement computations were made for three separate surcharge schemes (Fig. 5). The results are given in Table 3. There is a definite differential in magnitude of ultimate settlement, but actually interest was in achieving a settlement equivalent to total theoretical settlement under no surcharge load. Scheme 1 would achieve this in about six months at both points whereas Scheme 2 would take

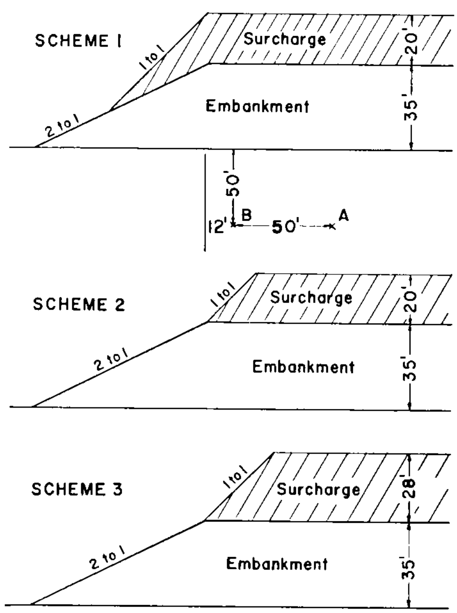

Figure 5. Surcharge configurations investigated.

TABLE 3

SETTLEMENT FOR VARIOUS SURCHARGE LOAD SLOPES, USING HOLE NUMBER 29 VALUES

Surcharge Condition	Point A [1]		Point B [2]	
	Settl. (ft)	Req'd. Cons. (%)	Settl. (ft)	Req'd. Cons. (%)
No surcharge	1.78	—	1.31	—
Scheme 1 (20 ft over berm, 1 to 1 down)	3.77	47	2.54	52
Scheme 2 (1 to 1 up from berm to 20 ft)	3.77	47	2.21	59
Scheme 3 (1 to 1 up from berm to 28 ft; 8 ft is material represented by difference between schemes 1 and 2)	3.82	47	2.36	55

[1] Point A 50 ft or more inside berm in midplane of silt.

[2] Point B 12 ft inside berm in midplane of silt layer (see Fig. 5).

about nine months. Scheme 1 was used on all normal roadway sections and Scheme 2 was used in the broad toll plaza area where its significance was much less.

Special Studies

In addition to the basic computations discussed above, various additional investigations and special studies were made. These included attempts to prove the results of the basic investigation and to determine the margin of deviation which could be expected due to variation in the material, in the samples obtained, and in the tests conducted upon these samples.

Consolidation tests were re-run in several instances loading each increment for 24 hours instead of 8 hours as before. Considering the depth of the layer of silt, the original settlement figures appeared low in both magnitude and time required. The 24-hr curves looked the same except that 90 percent settlement as determined by Taylor's square root of time fitting method (*4*) did not appear to be 90 percent of the total settlement after the curve was projected. Figure 6 is a typical curve of this nature. Computations of settlement based upon these curves are of longer duration but settlement under the 20-ft surcharge in the 6-months period available still appeared to be great enough to preclude further consequential settlement after removal of the surcharge. The amount of compression attributable to so-called "secondary settlement" is, of course, undetermined.

In order to bracket the variability of results some of the settlement computa-

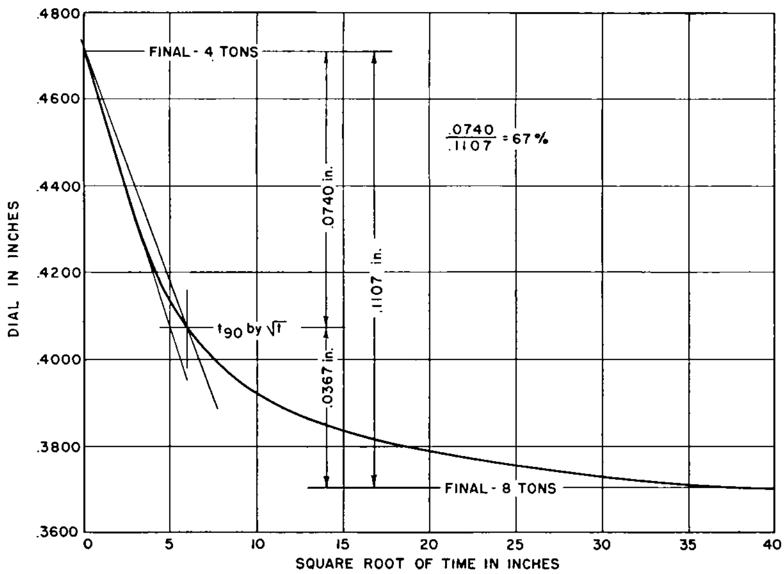

Figure 6. Consolidation curve for 24-hr 8-ton load.

tions were repeated varying consolidation test constants. This was done by neglecting a sample containing more than the average percentage of sand in one instance and in others by averaging all consolidation tests obtained, except for the high sand content one. An average test curve was obtained (Fig. 7) by averaging the results of all of the tests. Settlement was recomputed at all of the key points investigated using this curve. With the use of the data so obtained the order of magnitude of variation from the original computation could be seen. It is obvious that variations in the material, the sampling, and the testing preclude predictions to fine accuracy, but it is also shown that the order of magnitude is generally about right. Settlement over the whole area was found to vary from 1 to 2 ft. Some would read this as a possibility of 100 percent error. However, from a practical sense, the material will settle up to at least 1 ft and it will be unlikely to settle 3 ft. This information is of great value and should be read in this manner only. Obviously the entire area represented by one boring is not going to settle exactly 1.78 ft. An example of results obtained directly from the borings and from the average curves is shown in Table 4.

TABLE 4
VARIABILITY OF RESULTS, HOLE NO. 29

Hole	Normal Settl. (ft)	$T_{90\%}$ (yr)	T_{20Ft} (yr)
No. 29	1.78	1.84	0.43
Avg. curve	1.56	1.39	0.73

As the job required borrow material for almost all fills, it was essential that material used for surcharge loading be later incorporated in the work. In addition to fills for access roads which were not as high and ramps which did not traverse the silt area, it was planned to use the material in constructing the main roadway fill east of Childs Street. Some silt existed in this area but its depth was not as great, and the area had been used for many years for the storage of steel scrap in piles 20 ft or more in height. Consolidation under these loads was believed to be well advanced. One cross-sectional series of settlement computations was made in this area to determine the requirement for a slight camber in a drain line below this fill. The silt here was 18 ft deep under a 24-ft sand and gravel layer beneath a 24-ft fill. Settlement at the centerline was found to be about ½ ft and to take place in about 7 months. Settlement at the toe was about

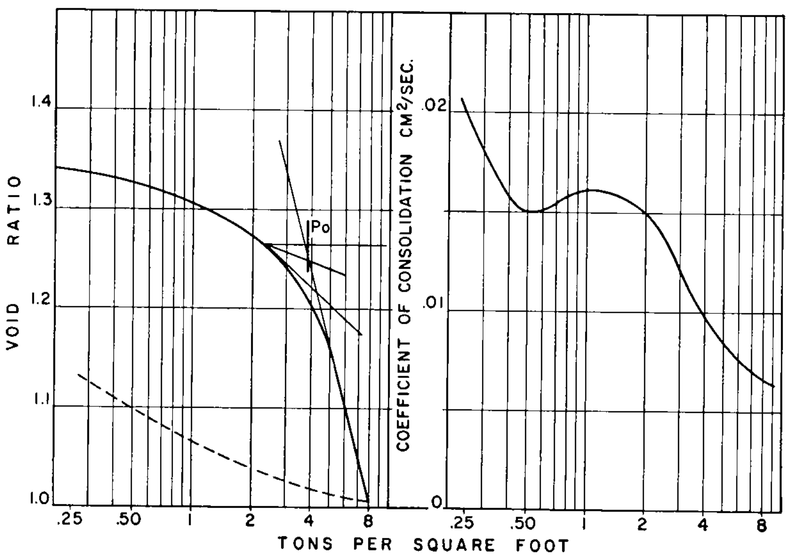

Figure 7. Consolidation test curves, average of 10 samples of organic silt.

$\frac{1}{10}$ ft and should take place in about 1 month. No surcharge was recommended or used.

Another special study was made to determine the effect of settlement caused by bridge piers and fills for ramps to the abutments of the Shell Road ramp if spread footings instead of piles were used beneath this bridge. The 24-ft layer of compact sand and gravel in which the footings were proposed was underlain by a 34-ft layer of 4- to 6-blow silt which was, in turn, above an 11-ft layer of 12- to 20-blow peat. This was underlain by very compact sand and gravel. No successful consolidation samples were obtained of the peat although attempts were made. It was treated in the computations as though it were silt. Surcharges of 10- and 20-ft were tried on the south and north ramps, respectively. Of the 7 piers, 5 were not affected by the fills. Of these, four were computed to settle about 0.05 ft and one 0.12 ft. The end piers, due to abutment fills, were computed to settle about 0.3 ft whereas the abutments would settle 0.5 and 0.75 ft. This is a rather formidable picture, considering the unknown of secondary settlement in the peat. This bridge was constructed on piles. The south abutment fill was not surcharged and north fill received a 20-ft surcharge for a very short period only, due to construction complications.

CONSTRUCTION

Initial Layer

Placing of the fill was begun in September 1955. Fill material was a mixture of sand and clay which made excellent embankment. Some good bankrun gravel also came from the borrow pit in the early stages of construction. Relatively clean sand and a very hard, dense red clay were often encountered and were mixed by consecutive and alternate placement and spreading. Placement of almost the entire fill was done with trucks hauling over public streets.

As the underlying silt body was below sand and gravel layers in all cases it presented no problem of being forced out provided all layers of fill were placed to the full width of the embankment as was required by the specifications. In no case was the fill allowed to become unbalanced. It was built from toe to toe full width and kept nearly the same height throughout its length during the entire construction period.

The lowest point of fill was in the gravel pit area (Fig. 8) a large part of which contained water 2 or 3 ft deep at river level. The contractor was required to place granular material such as sand or gravel in the initial layer where it was in water. He was allowed to place a layer about 4 ft deep by end-dumping and bulldozing ahead of the equipment. This initial layer in the pit was compacted largely with trucks and a small rubber tire roller. Except for this initial layer in the areas of the gravel pit containing water all material was placed in layers of 8-in. loose depth and rolled to 95 or 98 percent of AASHO maximum density according to the AASHO standard requirements using rubber tire rollers and heavy pad wheel compactors. As stated previously, even in the gravel pit area a layer of about 5 ft of dense gravel occurred between the base of the fill and the silt stratum. The initial layer of fill over most of the area was a normal 8-in. layer on dry surface at about 15 ft above the river and 10 or 12 ft above the water table.

Settlement Plates

Settlement plates were set below the initial layer of fill. They were 3-ft square plates of $\frac{3}{8}$-in. steel with 8-in. deep cross fins on the bottom and an extendable section of 2-in. standard steel pipe projecting from the top. Level readings were made on the top of the pipe section. As the fill rose, the pipe was extended in 4-ft lengths. Level readings were taken before and after adding each length of pipe. Elevations were read twice each week during placement of the fill and one month thereafter, after which they were read once each week until after removal of the surcharge. As the surcharge was

Figure 8. View of the gravel pit area before construction.

removed pipe sections were removed from the plates, readings being taken before and after removal. A total of 36 plates were used. They were spaced generally in groups of three across the fill at the points most nearly represented by borings and settlement computations.

Order of Construction

In the fall of 1955, the gravel pit area was filled and this and the toll plaza area were brought to about 10 ft above the original ground. Due to freezing of the soil in the fill and borrow pit, it was necessary to discontinue placement of the fill late in December. Placement was continued after March 15, 1956. Readings of settlement plates continued during the shutdown period. The actual surcharge was placed very rapidly during the latter part of April and early part of May. From a point 2 ft above finished grade the contractor was allowed to place the material in lifts up to 3 ft deep and to compact only as required for his own equipment. This method was entirely satisfactory from the standpoint of the surcharge but it allowed the material to absorb considerable moisture over the settling period, making its placement in a subsequent embankment more difficult. Figure 9 shows a portion of the surcharge in place.

The surcharge was removed between Frankfurst Avenue bridge and the railroad bridge during October and November of 1956. At this time this surcharge had been in place about 6 months, an appreciable settlement had been achieved and the curve had flattened out so that movement had apparently ceased. Rebound after removal of the surcharge was so small and rapid that it was difficult to distinguish. It may have been partially lost in breakage of pipes by construction equipment. The surcharge remained upon the toll plaza a longer period than was intended since removal was started in November but was discontinued in December due to difficulties in placing compacted fill during the winter. Removal was resumed in mid-February and was completed about the end of March. Some of the settlement plates in this area were destroyed during removal of this surcharge by pan operation, making further observations difficult. The concrete pavement was placed almost immediately upon the surface of the embankment. Levels are now being run approximately once every two weeks upon a number of points in the concrete pavement. No movement has been detected as yet.

GENERAL

The importance of achieving complete stability, particularly in the toll plaza area, in the short time allowed by the tight construction schedule cannot be over-emphasized as the vast expanse of pavement on this area was to be concrete, and settlement, if it did not injure the surface itself or interfere with drainage might disrupt connections with the toll booths and utility tunnel which, constructed under another contract, were on piles through the center of the area.

Material

The organic silt material with which this paper is concerned is not a normal recent river or swamp deposit but was deposited during a previous submergence of this area geologically known as the Talbot submergence as was the deposit of sand and gravel above it. It has, therefore, had a long period in which to consolidate under the load of this material, which probably was considerably deeper than the 20 ft found above it at the time of this investigation. This is evidenced by the apparent preconsolidation loads shown by the consolidation tests and is reconfirmed by the definite increase in rate of settlement as the surcharge was placed. The preconsolidation load was about the same load as the permanent fill would place upon the soil. Figure 7 shows the average consolidation curve and Figure 10 shows typical settlement curves. Settlement increased rapidly as the surcharge was placed.

The material was definitely silty in nature even though PI values ranged between 7 and 18. The grain size and structural characteristics indicated a high permeability. The normal grain size distribution was 100 percent passing a No. 200 sieve (exclusive of organic materials) and only 20 percent below the 0.002 mm size. Figure 11 shows typical grain size curves. Structurally the silt contained varying amounts of decomposed grasses which formed flow channels horizontally and sometimes vertically. The time-settlement relation shown by the tests was relatively rapid even to the extent of

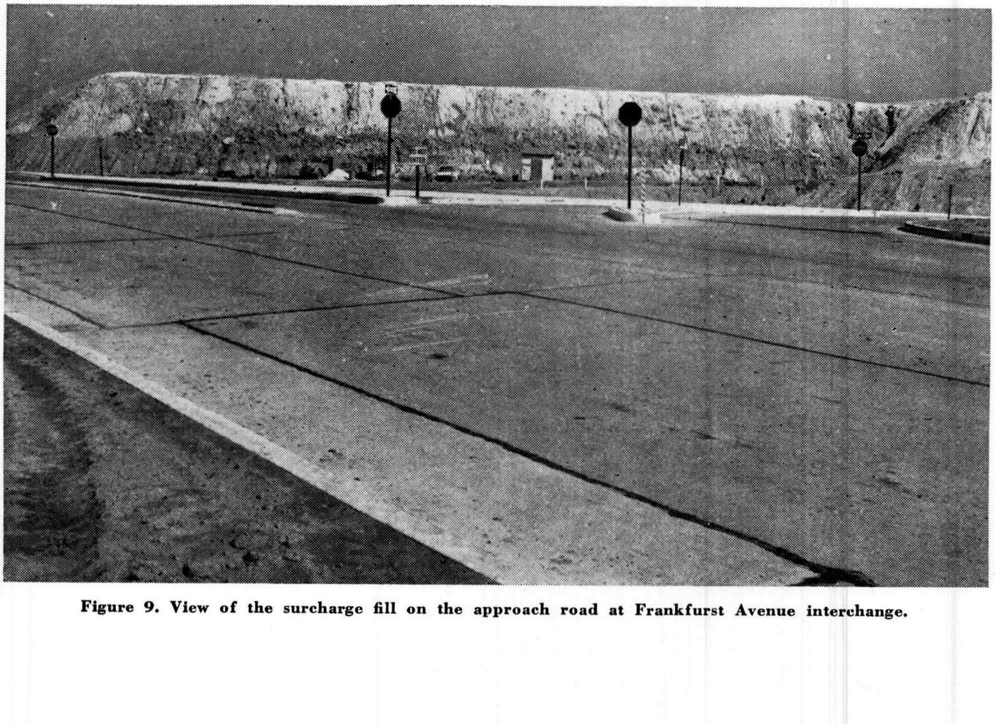

Figure 9. View of the surcharge fill on the approach road at Frankfurst Avenue interchange.

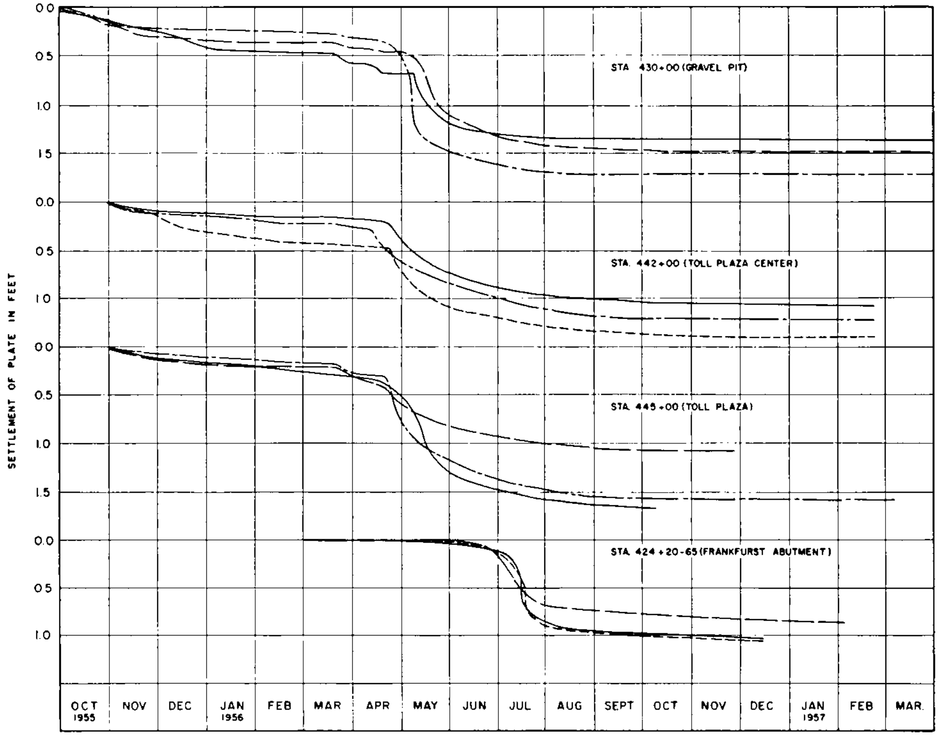

Figure 10. Typical settlement curves from field observations.

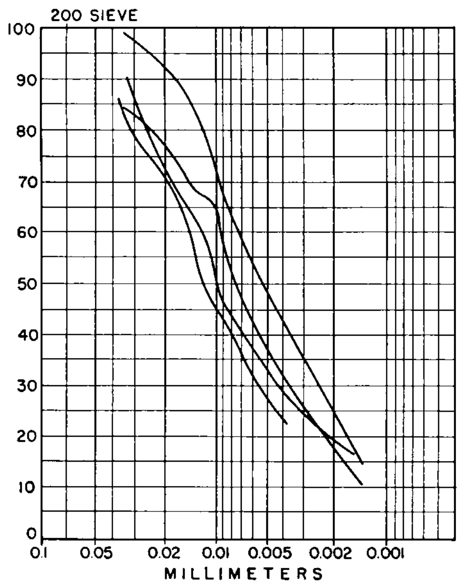

Figure 11. Typical grain size curves for the organic silt.

causing some anxiety as to reliability of the results. For this reason, further studies were made re-testing some of the same samples using longer consolidation periods between load applications. Later the observed settlement confirmed the high permeability as time of consolidation was close to the time computed.

Stability

The preconsolidation to which the material had been subjected geologically is of great importance in the problem of stability against shear failure by overloading. Twenty feet of compact sand and gravel remained in place above the silt except in the gravel pit where it had only recently been removed and under which the silt had previously achieved its total settlement. The toe of the fill was computed as 2 to 1 at all points. Actually, the gravel pit area was crossed diagonally by the relocated Frankfurst

Avenue embankment which was slightly above the elevation of the ground bordering the pit. Four ramps of the interchange were in this area, and the embankment was skirted on the north from Frankfurst Avenue past the toll plaza to Childs Street by an access road at the original ground elevation 20 ft above the silt. Therefore, the entire gravel pit area bordering the fill was refilled to the old ground level or higher well beyond the toe of the slope of the surcharged fill. Consequently the stability computations were based upon conditions far worse than actually existed.

Surcharge

The choice of surcharge without sand drains was made after consideration of the cost of sand drains against the apparently rapid settlement predicted by the tests. As mentioned previously, settlement predicted appeared more rapid than would normally be expected and some concern was felt that permeability of samples might have been affected by cutting of planes of the organic matter in the silt which would not be realized in the undisturbed deposit. This uncertainty probably dictated the use of a 20-ft surcharge instead of 15 ft. However, the quantity of surcharge was kept well within that which could later be incorporated in the embankment elsewhere.

Surcharge shoulder configurations (Fig. 5) were found to be critical on narrow embankments but of less consequence on wider fills. Surcharge slopes used were satisfactory. No difficulty was encountered in placing relatively uncompacted soil on 1-to-1 slopes. Some erosion took place and slides were threatening at some points on the toll plaza after the surcharge had been in place through the winter. It was removed, however, before any damage was done to the roadway fill or slopes.

Settlement Observations

The settlement observations were made by running levels on 2-in. pipes extended from settlement plates. These did not prove entirely satisfactory due to frequent damage by equipment. The contractor was charged with protection and maintenance of the plates, but with about 30 trucks operating on double shifts on the narrow fill, particularly during the surcharge placement, protection with flags, lights and continuous threats and warnings was not enough to prevent many broken pipes. Pipes were dug up, levels re-run, and the pipes were re-extended so that nearly continuous records were maintained for most of them. It was found possible, with a small pipe with a welded hook to feel out joints about 12 ft below the fill to confirm position of recovered pipes and hence recover or adjust for settlement lost by a few days loss of a pipe. In spite of these obstacles, quite a number of settlement curves were obtained which are believed to be very close to what actually occurred. Figure 10 shows some of them. Figure 12 shows three settlement plate curves, the embankment height, and predicted settlement with and without surcharge on each of 2 cross-sections of the fill. Some settlement occurred under the early fill placed in the fall of 1955. This curve started to flatten before resumption of operations in the spring, then bent downward relatively slowly until the level of the finished grade was reached. As the surcharge was placed, the curve turned sharply downward, almost dropping, until most of the predicted settlement had occurred within the first month. It then flattened rapidly and soon leveled off. As far as late observations show, it is now flat. Readings on points on the concrete will continue.

It is believed that the treatment applied on this project has been successful. Its success is largely attributable to the type of material and to the condition of preconsolidation. However, the methods used here certainly have many applications. In the regions along the Chesapeake Bay and similar tidal estuaries surrounded by and founded upon similar alluvial formations, old swamp and river deposits of this nature are not uncommon. In this day of super highways with

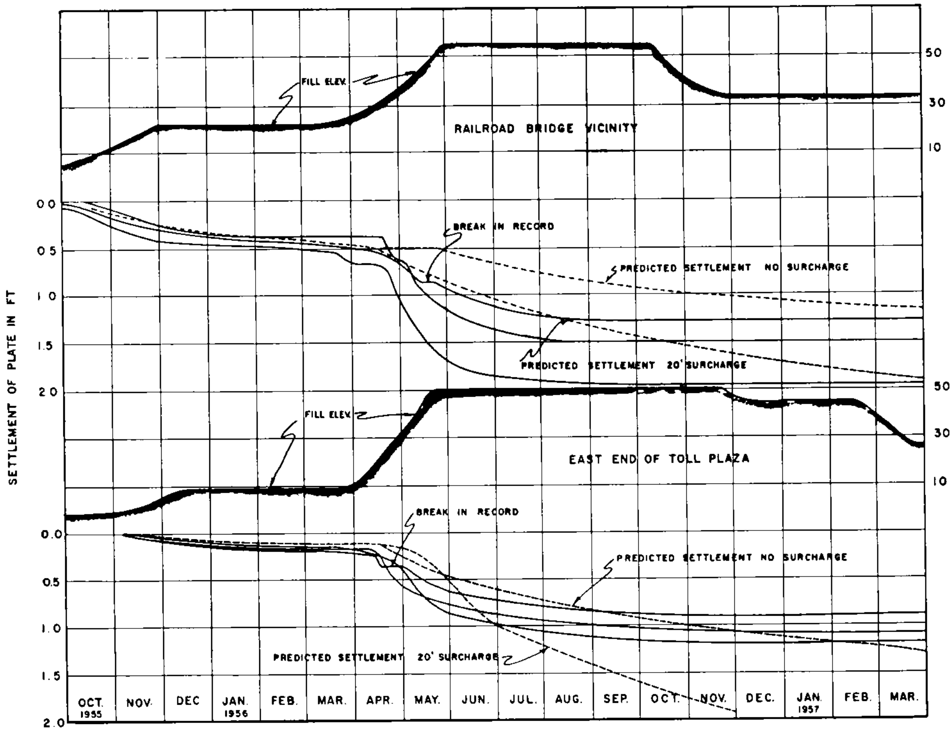

Figure 12. Settlement curves from field observation and predicted settlement for the same points.

intricate interchanges high embankments are frequent. On another project now under construction in the Baltimore Harbor area, which has five such interchanges, very similar material occurs, as well as some areas of recent river silt deposits. Consolidation of the older deposits by the use of surcharges on local areas of high fills appears to have been successful. Some of the recent deposits, too fluid to be consolidated in the required construction time, have been displaced by surcharge loadings, rather than consolidated.

CONCLUSIONS

It is concluded from the experience gained on this project that:

1. Certain types of organic silt deposits of considerable depth and extent, in particular, geologically preconsolidated deposits, may be consolidated sufficiently for safe construction of pavement on relatively high embankments by simple surcharge loadings.

2. That such consolidation can be accomplished within reasonable construction time without the use of special drainage devices such as vertical sand drains.

3. That methods of settlement analysis developed in recent years based upon Terzaghi's theory of consolidation of soils are directly applicable to practical problems and can be relied upon to give results fully as dependable as the accuracy of normal sampling of soils warrants.

ACKNOWLEDGMENT

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tion was supervised by Whitman, Requardt and Associates, under whom this paper has been prepared. C. J. Langenfelder was the contractor.

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DISCUSSION

RICHARD J. WOODWARD, *Woodward, Clyde, Sherard and Associates, Consulting Engineers, Oakland, Calif.*—Mr. Janes' paper presents a very interesting example of the use of a surcharge fill to largely complete the consolidation of a silt deposit that would otherwise occur upon the placement of the design load.

The principle of partial preconsolidation by means of surcharge loads has been successfully used in the San Francisco Bay Area for several years. The principal differences between these experiences and the example described in the paper are in the permeability and the previous geological history of the deposits.

San Francisco Bay occupies an older depression, the most recent flooding of which probably occurred after the melting of the glaciers at the end of the last Ice Age. Most of the Bay has been filled with Pleistocene and recent deposits. The uppermost layer of the latter deposit, which covers much of the area of the Bay outside of the main channels, is a blue-grey silty clay locally termed "Bay mud." This is a soft and very compressible deposit. The void ratios commonly range from about 2 to 4, depending on the depth and location of the layer. The unconfined compressive strength normally ranges from 300 to 600 psf. The average value of the liquid limit would run from 50 to 80, with the corresponding plasticity index ranging from about 25 to 40. It is not uncommon for 80 to 90 percent of the particle sizes to lie between 1 and 50 microns.

Near the shore the mud surface is often exposed either in the form of marshes or as a bare mud deposit at low tide. Many such shore-line areas have been filled during recent years to provide sites for housing projects and light industrial developments together with the accompanying streets and other improvements. The amount of fill placed varies, from as little as 2 ft to more than 10 ft. The settlement due to the weight of the fill will vary with the compressibility and thickness of the deposit and with the thickness of the fill and the magnitude of the building loads. (These latter loads are normally of much less consequence than the weight of the fill.) Anticipated ultimate settlement may vary from several inches to several feet.

Practically all the building comprising the housing projects built on these marginal lands, as well as most of the light industrial buildings, are supported on spread footings resting in the fill. This is done not only because it is more economical, but also because it is generally more desirable to have the structure settle at the same time the streets and the general area subside, as long as the settlement is fairly uniform.

It is often helpful to accomplish a portion of the expected consolidation prior to actual construction of structures. This is particularly true when the weight of the structure might produce appreciably more settlement under the building area than would occur under the streets and the surrounding areas. Surcharge loads have been successfully used to accomplish this purpose. Although the rate of settlement of the Bay mud is considerably less than that of the deposits described in the subject paper, preconsolidation by surcharge loads still accomplishes a great deal of good. A surcharge load left on for several months can and generally does cause several inches of settlement. Most of this settlement occurs in the uppermost portions of the consolidating deposit; thus producing a thicker mat of stiffer soil materials which tends to spread or "bridge" areas of variable compressibility in the deeper deposits.

Preconsolidation by surcharge, when applicable, has been well worth the effort expended.