SITE PRELOADING ELIMINATES PILES FOR TWO OIL STORAGE TANKS

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(Presented at a meeting of the Structural Section, B.S.C.E., held on November 14, 1956.)

Synopsis

The Metropolitan Coal Company proposed to construct two fuel oil storage tanks 120 ft. in diameter and 60 ft. high at its branch in Chelsea, Massachusetts. Soil conditions at the site consist of approximately 13 ft. of miscellaneous granular fill underlain by from 6 to 13 ft. of soft organic silt and inorganic silty fine sand. Stiff yellow clay, sand, gravel, and boulders occur below this compressible stratum.

The problem was one of selecting a suitable foundation treatment for the oil storage tanks. These site conditions would normally require a pile foundation but after considerable study it was concluded that a site preloading operation could be successfully carried out at less than half the cost of a pile foundation. Therefore, approximately 28,000 tons of sand and gravel were placed first at one site, then moved to the second site for the purpose of precompressing the underlying soil to minimize foundation settlement.

This paper describes the soil engineering investigation for the design and control of site preloading for these oil storage tanks.

Introduction

Site preloading consists of applying a dead load or surcharge over the site for a proposed structure, generally equal to or greater than the total weight of the structure. After compression of the underlying soil has occurred under the preload, which is usually an earth fill, the preload is removed and the structure is built. Thus, the purpose of site preloading is to develop settlements before construction and therefore to minimize the structural settlement.

The theory behind the principle of preloading, which involves the nonelastic nature of soils, is now well known to soil engineers. Briefly,

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when a soil stratum first experiences a given load it compresses over a period of time under that load. If the load is removed, the stratum will usually expand a small percentage only of the compression it underwent. Finally, if the load is reapplied the recompression will be small, perhaps only slightly larger than the expansion. The relative magnitudes of initial compression, expansion, and recompression depend on the soil type, thickness of the stratum and time of loading. The most complex problems occur where organic soils are encountered.

While preloading for structures is relatively uncommon, the procedure was used for bridge approach fills and other engineering projects long before the advent of soil mechanics. Four years ago, Stanley Wilson, then Assistant Professor of Soil Mechanics and Foundation Engineering at Harvard University, gave a paper before the Structural Section on the "Control of Foundation Settlements by Preloading."[1] Mr. Wilson described in detail the fundamental load-settlement characteristics of soil and then described several projects where preloading had been applied successfully. Site preloading for a cathedral in Baltimore was reported in the Engineering New Record, April 28, 1955. Dr. Arthur Casagrande was consultant on this project. In recent years reports on various forms of preloading and overloading in connection with earth embankments for highways have become common.

I would like to describe the soil engineering investigation for the design and control of site preloading for two large oil storage tanks. This has been one of the most interesting and challenging projects in which I've participated largely because of the nature and scope of the field data which were obtained. This was possible only through the complete cooperation of the owner, the engineer, and the contractors, all of whom recognized the importance of the undertaking.

Preliminary Considerations

The Metropolitan Coal Company proposed to construct two oil storage tanks, 120 ft. in diameter and 60 ft. high, at its Chelsea, Mass. branch on Broadway Street near the north end of the Mystic River Bridge. Mr. Everett C. Hunt of Hunt and Slayter, was engineer for the 120,000-barrel tanks. I served as soils consultant to Mr. Hunt.

[1]Journal of the Boston Society of Civil Engineers, January, 1953.

Site Conditions:

Figure 1 is a photograph of the site taken from the Mystic River bridge. The open area where the tanks will be constructed, is studded with concrete piers which supported an elevated track used to convey coal to the area. The track and supporting timber structure were demolished in preparation for the new tanks shortly before this pic-

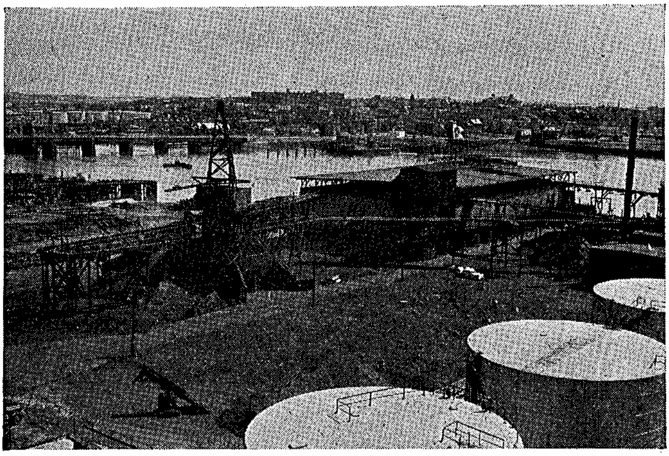


FIG. 1.—PHOTOGRAPH SHOWING PROPOSED SITE.

ture was taken. The remains of the coal pile, at times as high as 33 ft., are shown in the lower central part of the picture. The small building with stack at the far right of the picture is the Company's boiler house. The Chelsea River in the background joins the Mystic River to the right of the picture.

A site plan showing the maximum extent of the coal pile is shown in Figure 2. The proposed locations for the oil storage tanks and steel safety dykes are shown by dotted circles. It can be seen that both sites have already experienced various degrees of preloading from the coal pile over a long period of time. However, since coal weighs about 60 lb. per cu. ft. only, the maximum preloading is approximately 1 ton per sq. ft. which is less than 60 per cent of the future load under the oil storage tanks. The east edge of the site has experienced no preloading.

Soil Conditions:

Eight core borings were made by the Raymond Concrete Pile Company in January and February of 1955. Locations of these borings are shown on Figure 2 by Nos. 1 through 8. Soil conditions disclosed by these borings are summarized in Figure 3 which shows cross-sections, looking north, through the center of the proposed tanks.

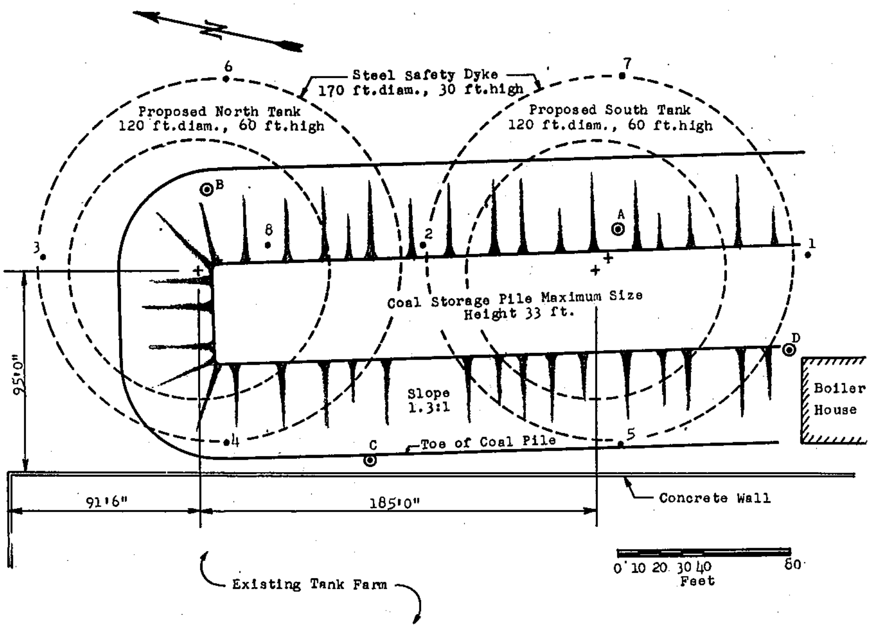


FIG. 2.—SITE PLAN SHOWING LOCATION OF BORINGS.

Approximately 13 ft. of miscellaneous granular fill having a standard penetration resistance (blows per foot on the split spoon sampler) of from 3 to 60. The original surface before filling varied from mean sea level near the boiler house to El. +3 at the North end of the site. Soil conditions in this former tidal flat vary from soft grey organic silt and silty fine sand to grey inorganic silty fine sand. The thickness of this compressible stratum is 13 ft. at the South tank site tapering off to about 6 ft. at the North tank site. Stiff yellow clay, sand, gravel and boulders occur below the compressible stratum. Some evidence of soft blue clay, peat, and medium yellow clay was observed in various borings. It should be noted that the vertical scale in Figure 3 is exaggerated. In reality, the compressible strata are thin relative to the tank size.

These soil conditions combined with the fact that the tank sites had experienced varying degrees of preloading positively eliminate the possibility of constructing the tanks without piles or some special foundation treatment.

The Preloading Decision:

Three possibilities for providing adequate foundations for the proposed tanks were studied at various times by Mr. Hunt, by Mr. Charles C. Ladd[2] and by myself. These foundation treatments were:

1. The use of bearing piles, driven into the hard yellow clay, to support the entire load.
2. The removal of existing fill and all other compressible soil and replacing it with compacted granular fill.
3. Preloading the sites to precompress the underlying soil for the purpose of minimizing differential tank settlements.

Several general requirements were observed during these studies. First, the size and location of the tanks could not be changed. The base of each tank should not be below El. +15 to provide adequate safety against excessively high tides. Finally, one tank would store No. 2 domestic fuel oil weighing 53 lb. per cu. ft. while the second tank would store No. 6 fuel oil weighing 60 lb. per cu. ft. Therefore, the design uniform loads for tanks 60 feet high were 1.6 and 1.8 tons per sq. ft. The weight of the welded steel tank is negligible.

Pile Foundation: Creosoted wood piles driven into the stiff yellow clay would vary from 25 to 35 feet in length and could be designed for 16 tons per pile. The cost to the pile foundation with a reinforced concrete pile cap was estimated to be from $200,000 to $250,000 for the two tanks. This is only slightly less than the cost of the two steel tanks erected on prepared foundations.

Excavation of Compressible Soil: If the depth of compressible soil is not excessive it is frequently economical to remove the soil and replace it with granular fill compacted under carefully controlled conditions. In this case excavation would have to be carried to a maxi-

[2]"Design and Comparative Analysis of an Oil Storage Tank Foundation," S.B. Thesis, M.I.T. May, 1955, unpublished.

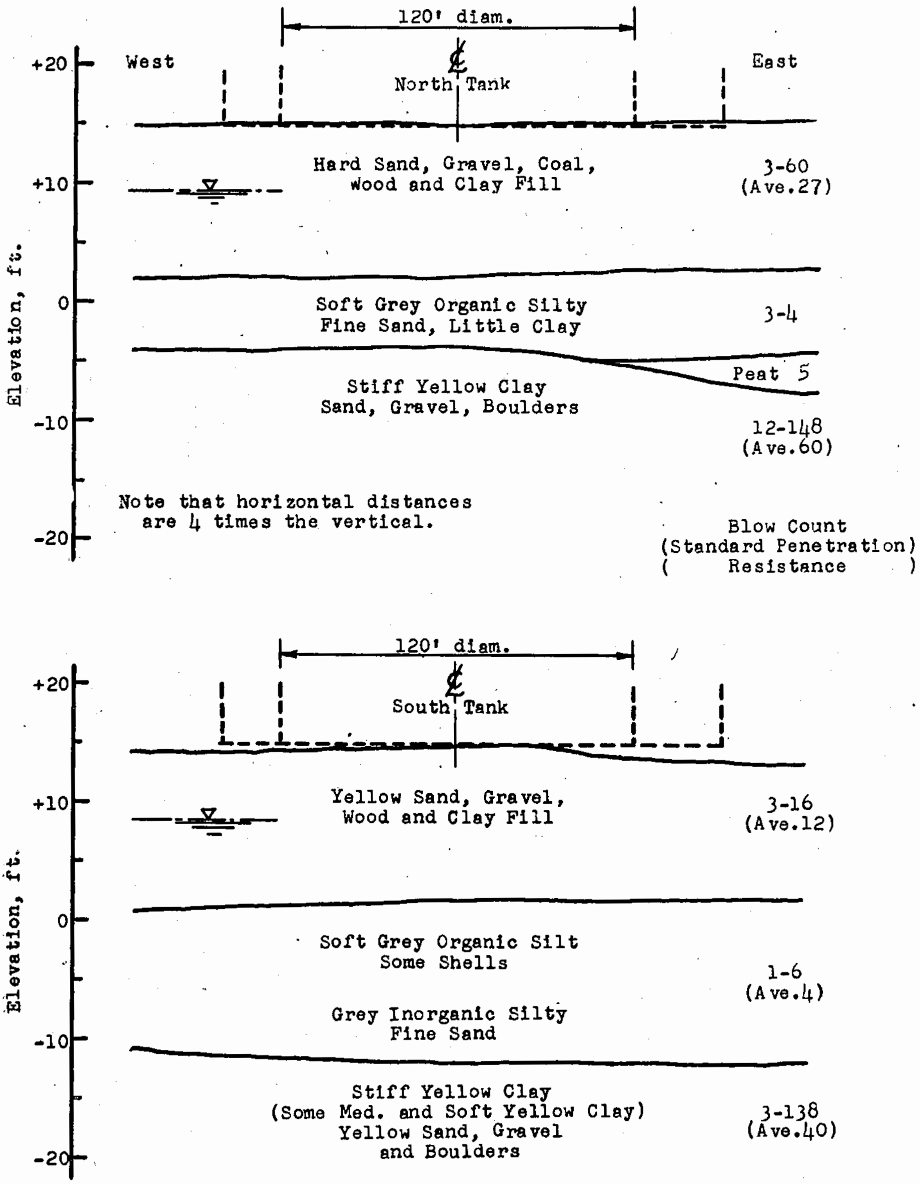


FIG. 3.—SOIL CONDITIONS BELOW PROPOSED OIL STORAGE TANKS.

mum depth of 28 ft. or 20 ft. below ground water level. Therefore, because of the confined area and soil conditions, anchored steel sheet piling would probably be required. Although this possibility was not explored in detail, a rough estimate of the cost of sheeting, excavation of fill and other compressible soil, and the cost of a compacted granular backfill exceeded $200,000 for the two tanks.

Site Preloading: A minimum preload of 1.8 tons per sq. ft. on the foundation soil would require a mound of earth at least 27 ft. high. If the soil were piled in the shape of a truncated cone covering most of the tank area, about 16,000 cu. yds. would be required. One site could be preloaded immediately while a sandy gravel pad was placed at the second site. The preload could then be moved to the second site while the first was prepared for its tank. After all preloading was completed, a portion of the material would be used to dress the area while the remainder would be sold.

The estimated cost of the preloading operation plus reinforced concrete rings below the tank sheets and additional site preparation for the tanks was less than $100,000 for both tanks. Therefore, a saving in excess of $100,000 was indicated if the preloading procedure could be carried out successfully. Several important questions arose, however. How long would each site have to be preloaded? How much settlement would the tanks experience after the sites were preloaded? Would the tanks be adequately safe against a shear or displacement failure in the foundation soil? Answers to these questions appeared favorable from the available data but additional subsoil exploration and laboratory tests on undisturbed samples were indicated.

Laboratory Tests on Undisturbed Samples

In February, 1955, the Raymond Company was directed to make four borings for the purpose of obtaining undisturbed samples by means of the 3-inch fixed-piston sampler. The location of these borings, A through D, is shown in Figure 2. Good undisturbed samples were obtained in Borings A and D only, because of the difficulty in advancing the drill hole through the miscellaneous fill and because of shells and other foreign material in the silt stratum.

The extreme variation in soil types within the compressible stratum is evident when results of Atterberg Limits are plotted on a plasticity chart as shown in Figure 4. From these results and visual examination of numerous samples from all borings, the following general observation is apparent. The top half of the compressible stratum below the South tank is a soft organic silt of medium to high plasticity and indeed compressibility, while below a depth of 20 feet (El. —5) the soil is a non-plastic inorganic silty fine sand. Consolidation of the compressible stratum during preloading will, therefore, occur far more rapidly than if the entire 13-foot depth were organic

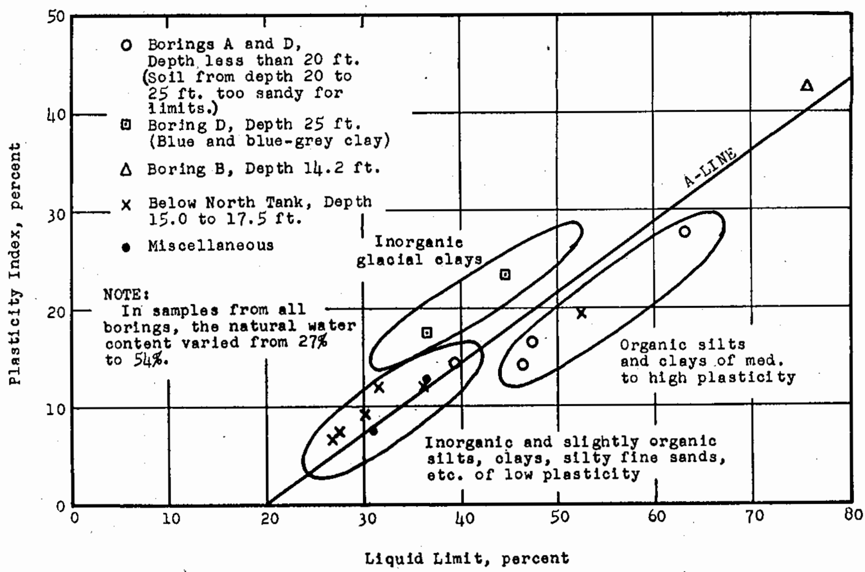


FIG. 4.—PLASTICITY CHART SHOWING EXTREME VARIATION IN ATTERBERG LIMITS FOR SOILS WITHIN THE COMPRESSIBLE STRATUM.

silt. In addition, long term secondary compression will be less. Soil within the compressible stratum below the North tank appears to fall between the extreme classifications found below the South tank.

Consolidation tests were run on seven samples from Borings A and D. Results of these tests, summarized in Table 1 show a wide variation in compressibility as expected. A measure of compressibility is given by the compression index which varied from 0.5 for samples at the top of the compressible stratum for 0.05 at the bottom. For comparison, the compression index for soft Boston blue clay is about 0.3.

A crude estimate of settlement under a 27 ft. preload fill for the

South tank can be made from consolidation test results according to the expression:

$$\rho = \frac{12\ Th}{1 + e_1} \cdot \frac{0.435\ C_c}{p_{ave.}} \Delta p$$

Where:

ρ	Settlement in inches
Th	Thickness of compressible stratum in feet
e_1	Initial void ratio oof compressible soil
C_c	Compression Index
$p_{ave.}$	Average intergranular pressure during consolidation
Δp	Pressure increment due to preload fill.

Extreme results may be obtained using the following data:

$Th = 13$ ft.
$p_{ave.} = 1.6$ tons per sq. ft.
$\Delta p = 1.8$ tons per sq. ft.

Low Compressibility:

$e_1 = 0.8$
$C_c = 0.07$

High Compressibility:

$e_1 = 1.10$
$C_c = 0.30$

which yield a settlement of from 3 to 11 inches. This result assumes no effect of preloading from the coal and does not include settlement within the existing granular fill overlying the compressible stratum or hard yellow clay below it. Indeed, when one considers these factors, settlement from local pockets of peat and soft clay and the varia-

TABLE 1
CONSOLIDATION TEST SUMMARY

Test No.	*Boring No.*	*Depth (Ft)*	*Ave. Natural Water Content (%)*	*Compression Index* C_c*
1	A	15.1	35.0	0.293
2	A	19.5	32.8	0.159
3	A	21.9	28.7	0.065
4	D	21.4	31.9	0.095
5	D	16.8	48.8	0.497
6	D	23.0	28.0	0.049
7	D	19.0	36.8	0.200

*C_c Compression Index, slope of virgin (straight line) portion of void ratio versus pressure (log scale) curve.

tion in thickness of the compressible stratum, a settlement variation of from 2 to 12 inches under the preload fill at the two tank sites is realistic. For comparison, observed settlements varied from 1.7 inches to 6.4 inches.

Settlement of the oil storage tanks after the sites had been preloaded will be a fraction of the settlement under the preload fill. That fraction depends on the magnitude of preload compared to tank load and on the time of preloading. More important however, is the fact that preloading will reduce undesirable diffierential settlements to a minimum. The only prediction I made prior to construction is expressed in the following statement taken from a letter to Mr. Hunt: "Preloading is expected to last from 3 to 4 weeks. The settlement of a full tank founded on the preloaded soil is expected to be fairly uniform and less than 3 inches over a period of years." While I have no conclusive information relative to allowable differential settlements for oil storage tanks, this prediction is apparently well within tolerable limits.

Attempts were made to run triaxial shear tests on samples from Borings A and D but the results scattered considerably. Difficulty was experienced in obtaining good samples for these tests. Samples were either disturbed, too soft, too granular or contained too many shells. Nevertheless, from the available data it was concluded that there was adequate safety against a shear failure or extrusion failure within the compressible stratum. The stratum was thin relative to the loaded diameter which leads to a higher ultimate bearing capacity than would be computed from conventional formulas. Furthermore, primary consolidation would occur rapidly eliminating excessive pore water pressures.

From results of the laboratory investigation and design analysis it was concluded that site preloading could be used successfully for the oil storage tanks. The Metropolitan Coal Co. and Mr. Hunt agreed then to proceed with the preloading plan.

The Preloading Plan

The preloading plan called for a fill of clean bank run gravel since a good granular soil was required for permanent compacted pads below the tanks. Settlement observation platforms and piezometers were specified to obtain field information for control of the preloading sequence. The contract for preloading and preparation of foundations for the tanks was awarded to the C. J. Maney Co. Elements of the preloading plan and foundation treatment are summarized in Figure 5.

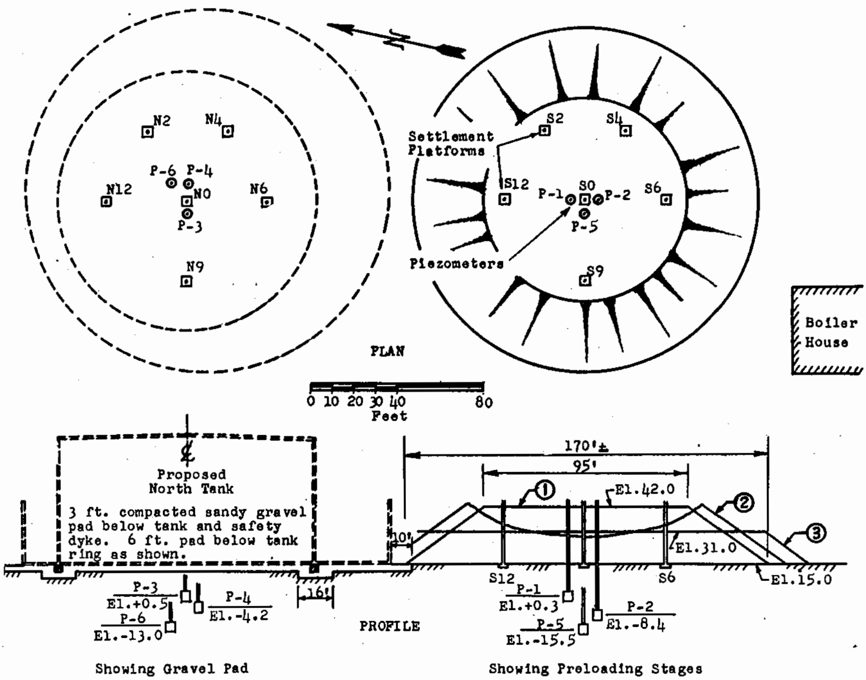


FIG. 5.—LOCATION OF SETTLEMENT PLATFORMS AND PIEZOMETERS DURING SITE PRELOADING FOR OIL STORAGE TANKS.

Settlement Observation Platforms:

Six settlement observation platforms were provided at each site, as shown in Figure 5, for the purpose of recording settlements during preloading. These platforms are numbered clockwise; for example, platform 12 is at the north edge of the fill which is defined as the 12 o'clock position. Platform 0 is at the center of the preload fill.

Each settlement platform consists of 2 ft. 6 in. square base made up of channel sections welded together. A ½ inch diameter steel rod is attached to the base and protected by a 2-inch I.D. steel pipe sleeve also mounted to the base. The base is firmly seated before preloading on a sand cushion at El. +15. The rod and protecting sleeve project upward through the fill. Elevations are taken on the tip of the rod from which settlements may be computed. Sections are added to the rod and sleeve as needed while the fill is placed.

The pipe sleeve was painted a bright yellow to warn the dozer operator. We were fortunate to have an experienced operator who appreciated the importance of the field observations. We did not lose a single settlement observation point or piezometer during the entire preloading sequence.

Piezometers:

In order to observe pore water pressure within the compressible stratum during preloading, three piezometers were installed by the Raymond Company at various depths near the center of each preload fill. These piezometers, the location of which is shown in Figure 5, are a nonmetallic type developed by Dr. Arthur Casagrande for use at the Logan Airport.[3] The piezometer point is a porous stone tube surrounded by a pocket of sand. A plastic tube, ½ inch in diameter, extends to the surface within a steel casing. Measurements of water pressure are computed from observations of water level in the plastic tube. The water level in turn is determined with an electrical sounding device.

Preload Fill:

It is generally desirable to provide a preload fill in excess of the structural load especially if organic soil exists at the site. A preload of from 125 to 150 per cent of the structural load should be used if possible. In this instance, however, the preload was designed to be 100 per cent of the maximum tank load. This would be satisfactory because the full tank load would be effective for a relatively short period of time each year. Furthermore, the tanks could withstand greater settlements than most structures. Finally, from the standpoint of economy, a 10 per cent increase in preload height maintaining the 95 ft. top diameter would require approximately 17 per cent additional earth fill.

Preloading at the South tank site was made in three stages as shown in Figure 5. In the first stage, 28,000 tons of sandy gravel were placed in the shape of a truncated cone 27 ft. high with a top diameter of 95 ft. and bottom diameter of approximately 170 ft.

[3]"Soil Mechanics in the Design and Construction of the Logan Airport," Journal of the Boston Society of Civil Engineers, April, 1949.

Twenty-five calendar days, beginning April 22, 1955, were required to bring the material to the site. Stage 1 preloading remained for 23 days before the pile was scooped out from the middle to form a dish shaped surface to give Stage 2 in Figure 5. The purpose of this preloading stage was to give additional precompression to the soil immediately below the tank ring. This stage remained 4 days before the material was leveled to El. +31 over the full area of the safety dyke. This is referred to as Stage 3.

During preloading for the South tank the foundation pad for the North tank was placed, Figure 5. An area extending 10 ft. beyond the safety dyke was excavated to El. +12 except below the tank ring where a strip 16 ft. wide was excavated to El. +9. About 8,000 tons of clean sandy gravel were imported and compacted in layers by a tractor dozer and loaded truck to bring the surface back to El. +15. On June 14, 1955, the dozer began to move the preload fill from the South tank site to the North.

The North tank preload, Stage 1, was completed in 39 calendar days and allowed to remain 16 days after which 8,000 tons from the west side, where settlements were small, were taken for the South tank pad. At the same time the remaining fill was moved upward toward the east to provide additional preloading below the future tank shell where preload settlements were very high. Sandy gravel was taken from this modified Stage 2 preloading to fill and otherwise grade various areas on the property while the excess was sold.

Field Observations During Preloading:

Settlement observations and piezometer readings during preloading were made daily by Dick Hume of Hunt and Slayer and Al Kapchus of the Metropolitan Coal Co. A summary of these observations is shown in Figures 6a and 6b.

Observation platforms 2 and 4 showed maximum settlements at both sites which was expected. It was noted earlier, Figure 2, that the east edge had not been preloaded with coal. Furthermore, some peat was encountered toward the east of the North tank site, Figure 3.

Maximum observed preload settlements at the South tank varied from 0.14 to 0.34 ft. Rebounds were from 0.028 to 0.038 ft. except observation point S12 which was 0.070 ft. In percentage of the maximum settlement, rebounds varied from 11 to 20 per cent except for S12 which was 39 per cent. At the North tank site maximum observed settlements varied from 0.17 to 0.53 ft. while the rebound was remarkably constant varying from 0.054 to 0.060 ft. only, or from 10 to 34 per cent of the maximum settlement.

The piezometer readings were useful to show that primary consolidation within the compressible stratum occurred nearly as rapidly as the preload was added since very little excess pressure was observed. Only P-2 within the silty fine sand below the South tank showed a rational response to the preload filling. During each working day the water level rose as much as 2 ft. in response to the filling. As consolidation continued overnight, the piezometer level fell. Three Sundays when no fill was placed, are shown at about 9, 16 and 23 days. Piezometer P-2 rose a maximum of only 6.5 ft. which is slightly more than 10 per cent of the theoretical maximum rise of about 60 ft. The latter would occur in a saturated soil if no consolidation took place while the 27-foot preload fill was applied.

Piezometers below the South tank were preserved following preloading by extending the tubes below the tank to the boiler house where they were connected to mercury manometers. This was done so that pore water pressures could be observed while the tank was filled. Since the tanks can be filled in less than 24 hours, pore pressures could be considerably higher than during preloading.

Tank Settlement

Following preloading, final site preparation for the oil storage tanks was carried out. A reinforced concrete ring, 3 by 3 ft. in cross section, was provided below the tank shell to give additional stiffness. The welded steel tanks and safety rings were provided and erected by the Hammond Iron Works.

Twelve settlement observation points were established around the exterior of each tank. Numbers were again assigned clockwise with the twelve o'clock position at the north edge of each tank. One-inch steel cubes were welded to the tanks and elevations were made on steel pins inserted horizontally into the cubes. The maximum and minimum settlements for each tank are shown in Figure 7.

Each tank was test filled with sea water before it was used to store oil. Since foundation conditions were more favorable at the North tank site, the heavier No. 6 fuel oil was stored in this tank. Test filling with sea water therefore exceeded the maximum fuel oil load by approximately 7 per cent in the North tank and 20 per cent in the South tank.

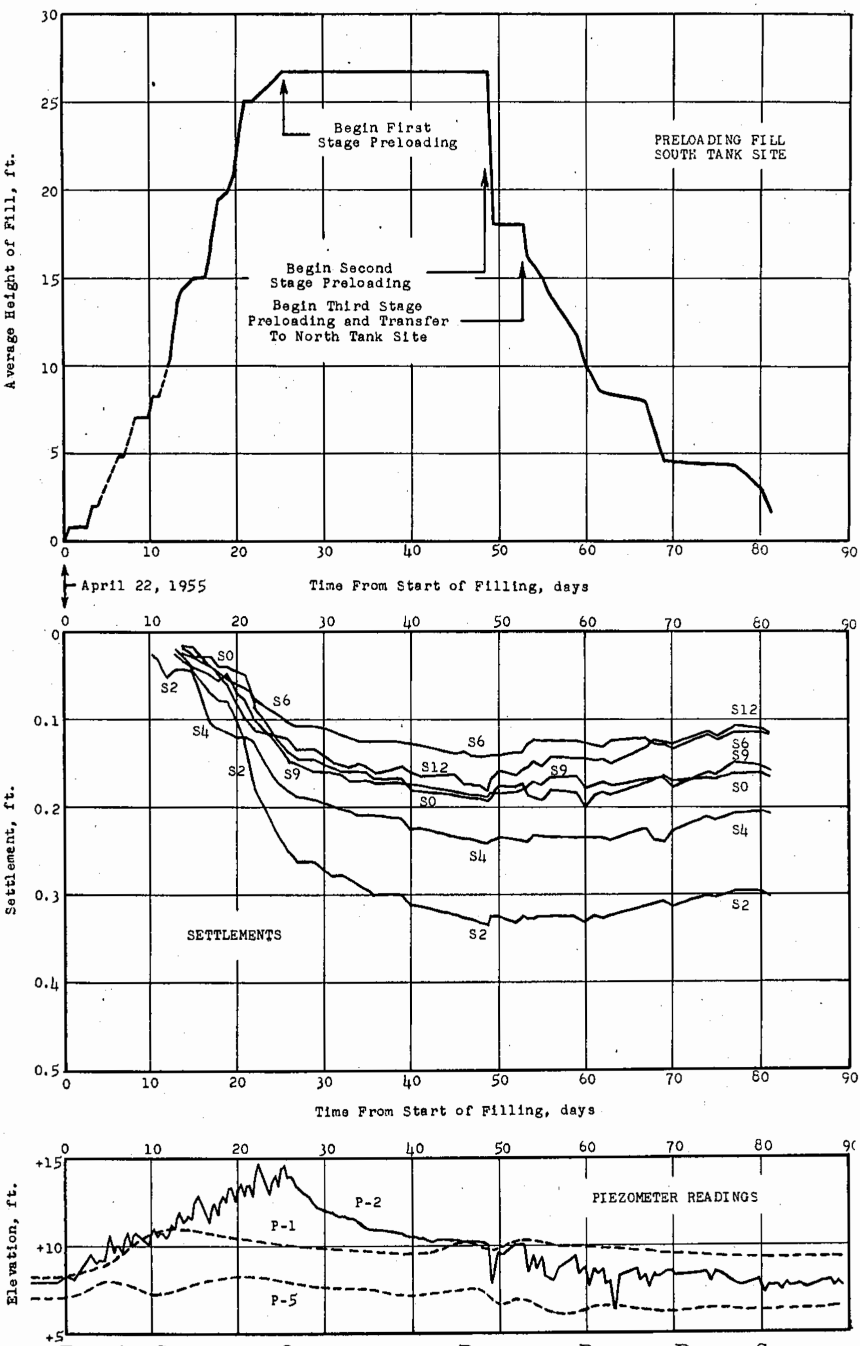


FIG. 6A.—SETTLEMENT OBSERVATIONS AND PIEZOMETER READINGS DURING SITE PRELOADING FOR SOUTH TANK.

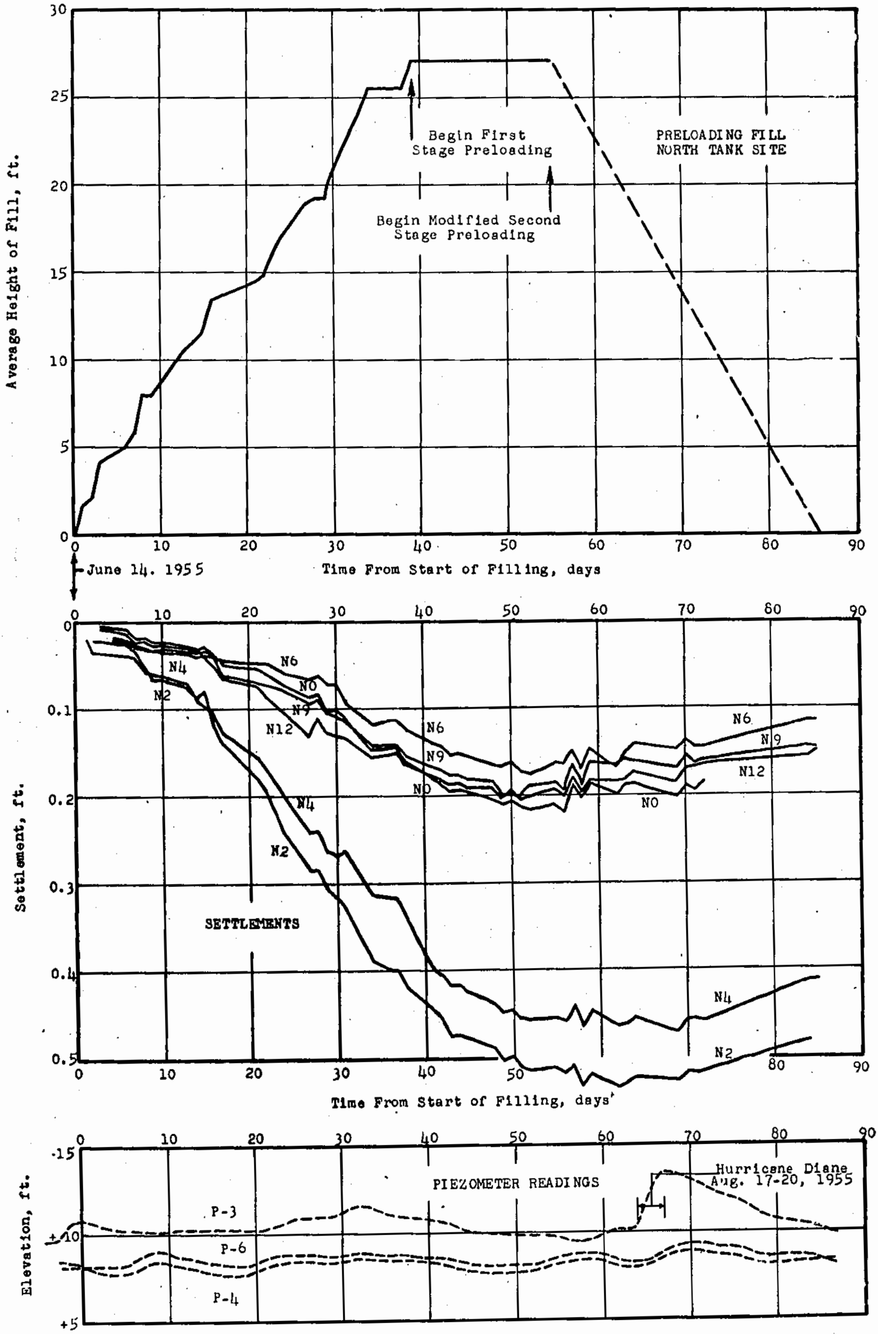


FIG. 6B.—SETTLEMENT OBSERVATIONS AND PIEZOMETER READING DURING SITE PRELOADING FOR NORTH TANK.

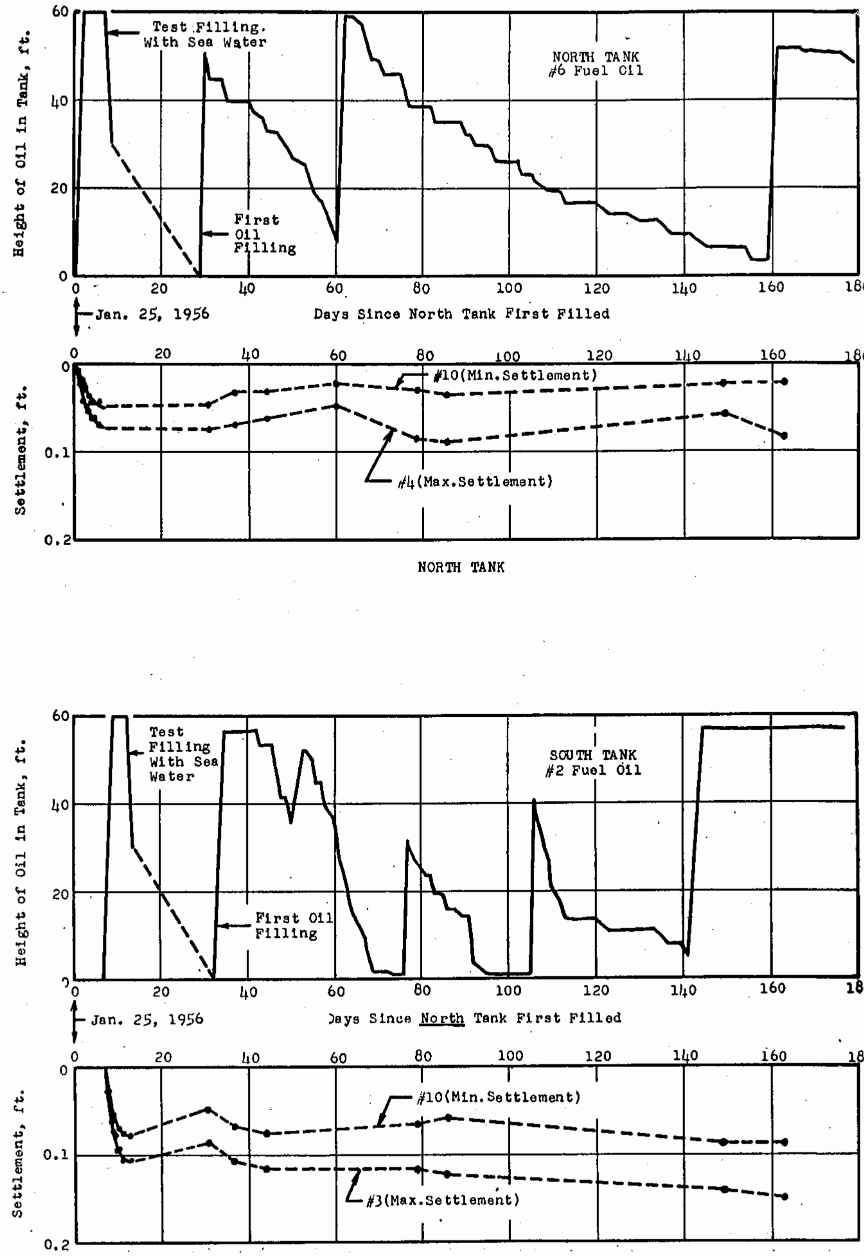


FIG. 7.—MAXIMUM AND MINIMUM OBSERVATIONS OF SETTLEMENT AT NORTH AND SOUTH TANKS.

Again as expected, the maximum observed settlement is occurring along the east edge, at the 4 o'clock and 3 o'clock positions at the North and South tanks, respectively. The minimum observed settlement occurs on the opposite side indicating that the tanks are tilting slightly to the east. As of July 7, 1956, the maximum observed settlement of the North tank was 16 per cent of the maximum preload settlement. At the South tank it was 45 per cent which reflects the effect of the organic silt found at this site.

The July 7th settlement observations for all points have been plotted in Figure 8. It can be shown that if the settlement of each observation point is plotted as shown, a uniform tilt without warping would be represented by points falling on a sine curve. I have

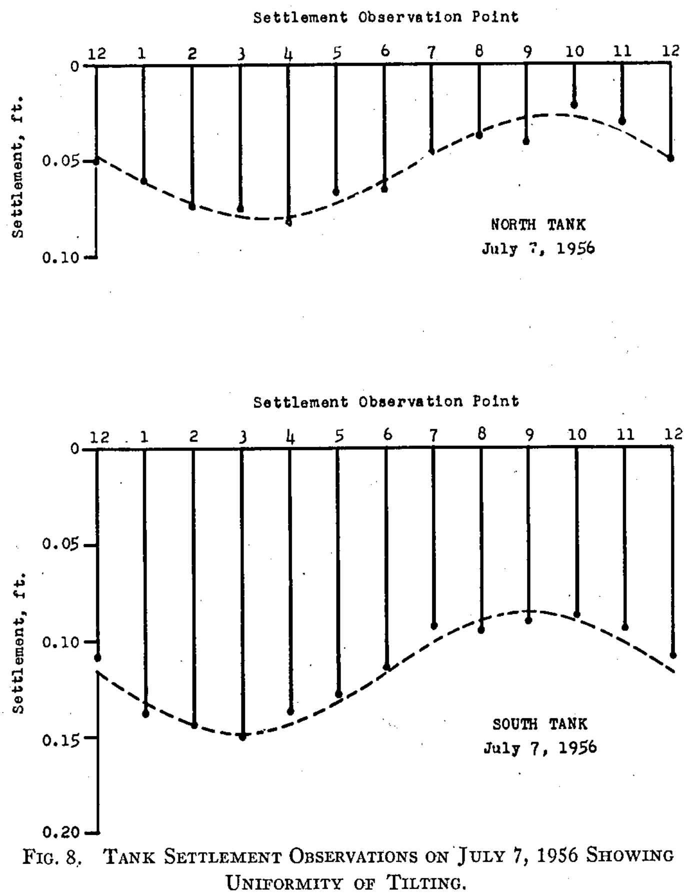
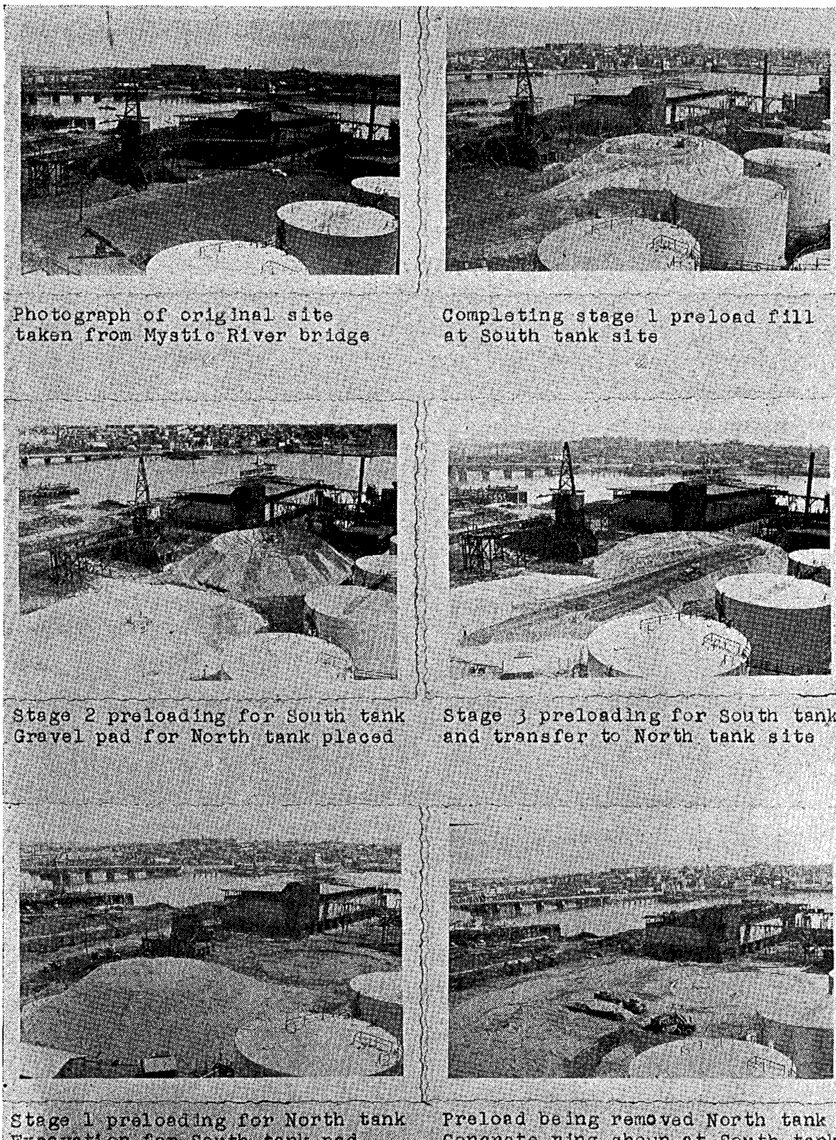


FIG. 8. TANK SETTLEMENT OBSERVATIONS ON JULY 7, 1956 SHOWING UNIFORMITY OF TILTING.



Photograph of original site taken from Mystic River bridge

Completing stage 1 preload fill at South tank site

Stage 2 preloading for South tank Gravel pad for North tank placed

Stage 3 preloading for South tank and transfer to North tank site

Stage 1 preloading for North tank Excavation for South tank pad

Preload being removed North tank Concrete ring shown at South tank

FIG. 9.

drawn the best sine curve through the plotted points. Only one point, No. 9 at the North tank, deviates more than 0.01 ft. or about one-eighth inch from the sine curve. This performance is indeed very satisfactory.

Settlement below the center of the tanks is undoubtedly greater than that observed below the ring. However, a tank can withstand considerable differential settlement in this direction. Future settlement is expected to be well within tolerable limits.

A final check on the performance of the South tank was made from mercury manometer readings for Piezometer P-2 when the tank was filled with sea water. These readings converted to piezometric levels indicated a rise of 14 ft. compared to a maximum of 6.5 ft. during preloading. Nevertheless, this still represents only 23 per cent of the theoretical maximum even though the tank was filled in 48 hours.

The total cost of site preloading and foundation preparation for the two tanks including piezometers, settlement platforms and concrete rings was about $93,000.

Summary

Site preloading is occasionally a practical and economical method of controlling foundation settlements. This is especially true under the following conditions:

1. When earth fill is needed at the site for other reasons such as grading for parking areas, etc., or when earth is readily available at or near the site.
2. When the compressible strata are inorganic and where the soil type and strata thickness are such that compression will occur rapidly.
3. When sufficient time is available for a thorough soil engineering study and for the preloading operation itself.

In the project which has been described, site preloading appears to have saved the owner more than $100,000 over the cost of a pile foundation.

Acknowledgments

I wish to acknowledge the splendid cooperation from Mr. Everett Hunt and the Metropolitan Coal Co. during the entire investigation. Many people assisted the author and their important contributions have been noted throughout the paper.