ers (Picture 6 and 7).

As a further measure of safety, a concrete approach span 23 ft. long (g on Drawing No. 4) was constructed on both sides.

The gravel fill for the roadway was then completed on both sides and instead of a concrete roadway, a waterbound macadam roadway was constructed. The structure is now in service for a decade and no further movement could be detected during that time.

Conclusion. The fact that the bridge stood up under the heavy truck traffic for more than ten years without further movement seems to indicate that the writer successfully equalized the pressure exerted by the roadway fill upon the marsh. However, he does not maintain that he, with his method, arrested the soil movement. Since the soil is still moving, although sidewise only - this assertion can be proved by the settlement of the roadway fill to the east and west of the bridge. From time to time the macadam roadway leading up to the bridge from both sides must be brought up to grade with the concrete roadway slab of the approaches.

No. F-6

A CASE OF SETTLEMENT OF A BRIDGE PIER

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1. Location of the Structure. The bridge in question is located in a city on the Atlantic shore in southern New England. It spans a river close to the point where the latter flows into the sea and is within the zone of tidal influence.

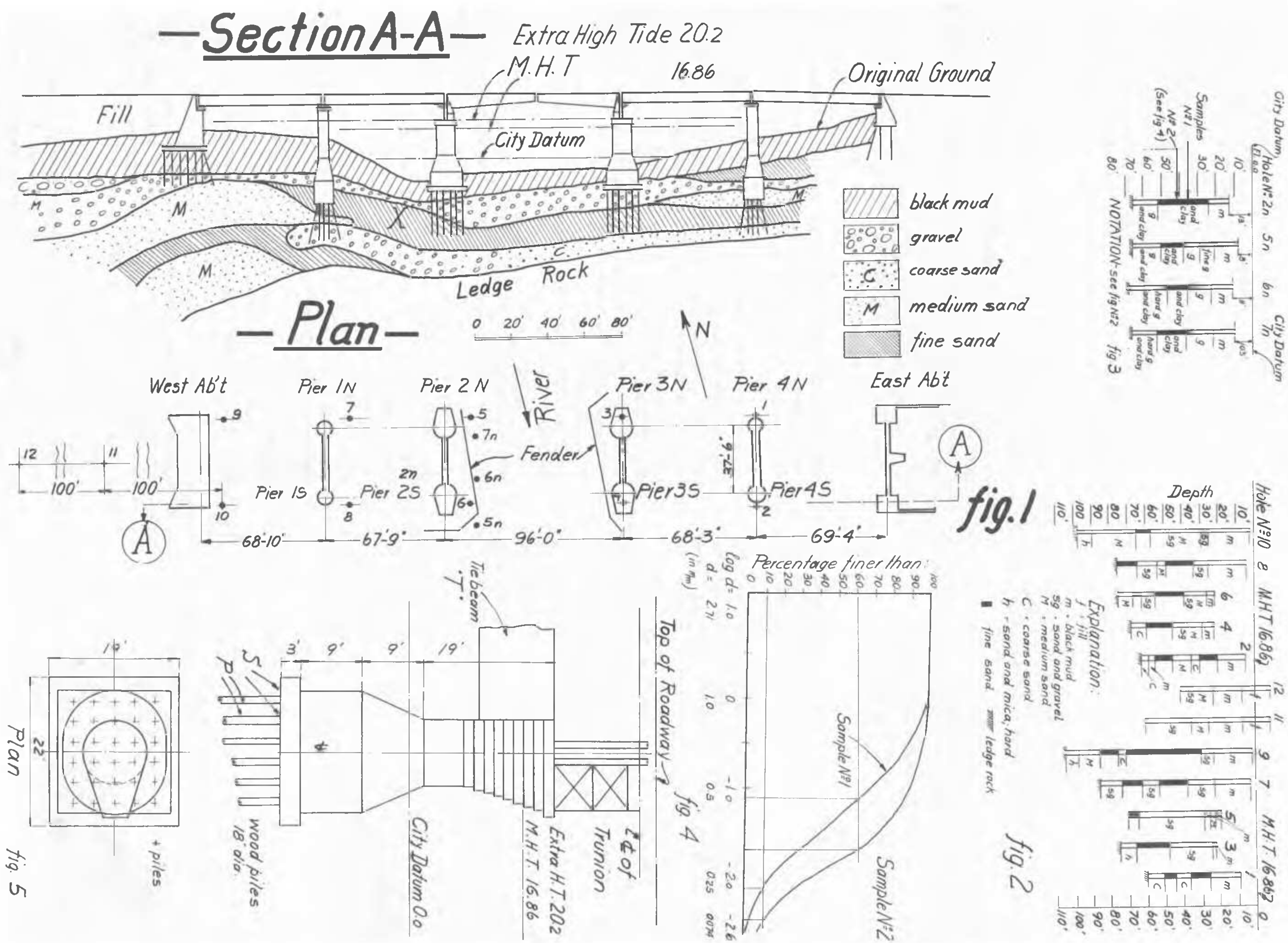
2. Geological Conditions and Subsoil Exploration. Prior to the construction of the bridge, about 1918, wash borings were made, (twelve holes, marked 1-12 in the plan, Fig. 1). Bed rock was discovered at depths fifty to eighty feet below the city datum and the deposit, according to boring data, consists of a layer of black mud, intermixed with sand and rip rap and followed by fine and medium sand interlayered with gravel, (Fig. 2). A tentative geological profile is shown in Fig. 1, Section A-A. There are no samples of soil extracted at these preliminary borings. Additional subsoil exploration was made in April 1935, (four holes marked 2n, 5n, 6n, and 7n in Fig. 1, also Fig. 3). These also were wash borings and the finer material probably was washed out. Results of the mechanical analysis of two samples of sand extracted, are given in Fig. 4. This material contains about six per cent of mica by dry weight.

During the preparation of this paper (March 1936), dry samples were being extracted but their analysis was not yet available.

3. Description of the Structure. The bridge is a double ~~lead~~ leaf bascule span 96 ft. center to center of trunnions; there are approach spans of about 68 ft. on each side. The original plans indicate that each of the four bascule piers, 2N, 2S, 3N, 3S rest on thirty wood piles. It is believed that the actual diameter of these piles is 12 to 14 in., and not 18 in., as shown on the plans. A participant in the construction states that these piles were delivered to the work in thirty-five foot lengths and were cut off ten feet before driving, so that their actual length is about twenty-five feet. According to computations made, the loads carried by the thirty piles supporting each pier amount to 1,630,000 lbs. or about twenty-seven tons per pile. A schematic sketch of a pier is given in Fig. 5. Each pair of piers (2N and 2S, 3N and 3S, Fig. 1) supporting the leaf span, are tied together by a reinforced concrete tie beam, (T, Fig. 5) and are partly faced with granite masonry.

4. Actual State of the Structure. The following was discovered in an inspection of the structure in October 1933. The southwest pier of the bridge (2S, Fig. 1) had settled at the south side about two feet and had moved in a southerly direction perhaps three inches. The heavy reinforced concrete tie beam (T, Fig. 5) was found split from top to bottom with every horizontal reinforcing bar torn, so that a wide crack appeared. Cracks in the granite masonry facing this pier were also found. The southwest trunnion column was badly twisted. Tie and cover plates of that column were torn in two and a main floor beam so badly twisted that it was impossible to lower the west leaf, which was found leaning about fifteen inches toward the south. It was recommended to the city to disassemble the west leaf and to remove a part of the floor and the counterweight. Consequently some other parts were also removed.

5. Trend of the Settlement. In Figs. 7 and 8, the time settlement curves are represented. Shortly after the bridge was open to traffic, a slight settlement was noted about the bascule; and the City Engineer established bench marks connected with adjacent factory buildings, (screws and spikes). The west abutment settled $4\frac{3}{4}$ in. in the first five years of the existence of the bridge but afterwards became stable. To study the inter-relationship of the settlements of the piers supporting the lifting span, the movement of three points on top of each of them was followed; points (a) and (c) are in the northern and southern part of each pier, respectively, and point (b) at the center line of the girders (Fig. 6).



—Section A-A—
Extra High Tide 20.2
M.H.T
16.86
Original Ground
Fill
City Datum
M
X
C
Ledge
Rock
black mud
gravel
coarse sand
medium sand
fine sand
—Plan—
0 20' 40' 60' 80'
N
River
West Ab't
Pier 1N
Pier 2N
Pier 3N
Pier 4N
East Ab't
Pier 1S
Pier 2S
Pier 3S
Pier 4S
Fender
A
12
11
100'
100'
68'-10"
67'-9"
96'-0"
68'-3"
69'-4"
fig. 1
Tie beam "T"
3' 9' 9' 19'
S
P
wood piles 18" dia
City Datum 0.0
Extra H.T. 20.2
M.H.T. 16.86
℄ of Trunion
Top of Roadway
fig. 4
Plan
fig. 5
+ piles
Percentage finer than
Sample Nº1
Sample Nº2
fig. 2
Explanation:
f : fill
m : black mud
sg : sand and gravel
M : medium sand
C : coarse sand
h : sand and mica, hard
fine sand
ledge rock
Depth
Hole Nº10
M.H.T. 16.86
NOTATION - see fig Nº2
fig. 3
City Datum
Samples

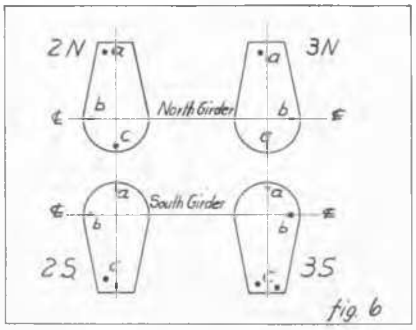


fig 6

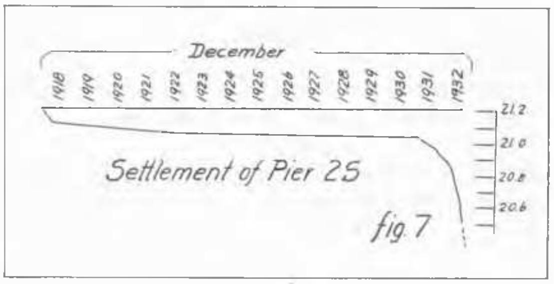


fig 7

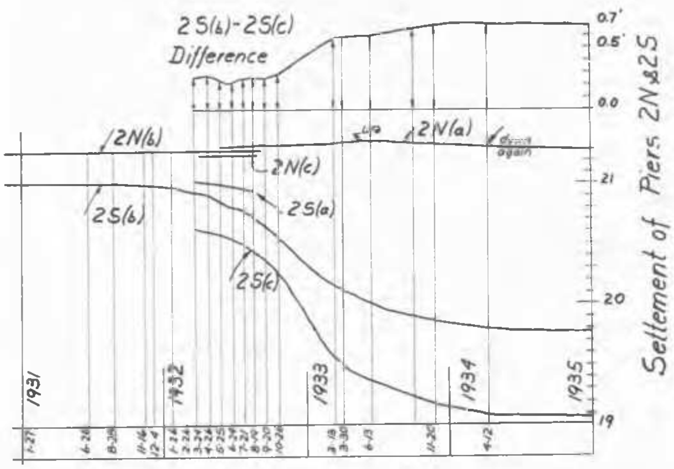


fig 8

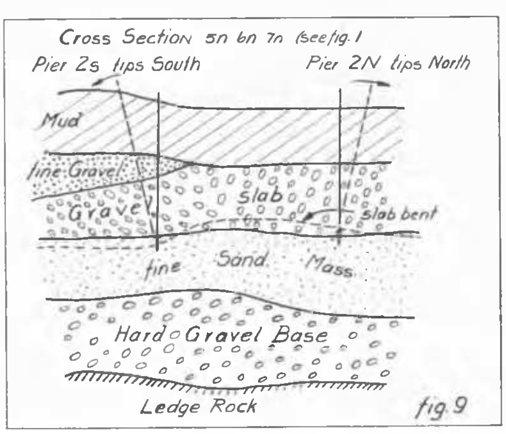


fig 9

Pier 2S revealed a smooth rate of settlement up to June 1931 (Fig. 7). A scow going north, hit the raised lift in February 1931 but apparently did not damage the structure. Only five months later Pier 2S had commenced settling at a sharply accelerated rate. By the end of 1932 the time settlement curve passed through an inflexion point and subsequently the settlement proceeded at a decreasing, though considerable rate. In round figures, Pier 2S, settled twenty-two inches and moved three inches south.

Pier 2N was moving up and down. Its total rise is about $1\frac{3}{4}$ in. accompanied with 1 3/8 in. north movement.

Pier 3N has raised 3/8 in. only and moved north about 5/8 in.

Pier 3S has not moved laterally but moved up about 7/8 in.

6. Interpretation of the Facts Observed.

Since piers 2N and 2S tended to tip in opposite directions, the connecting tie beam (T, Fig. 5) was pulled, with a progressively increasing force. The beam in question failed sometime between June and November 1931 and this was the reason why the settlement of Pier 2S acquired considerable acceleration about that time. Up to October 1932, pier 2S was sinking evenly as may be proved by the constant difference of the ordinates of curves 2S(b) and 2S(c) (Fig. 8). But since that time, which corresponds to the inflexion point of curve 2S(b), the pier in question tilts south (increase in the difference of the curves 2S(b) and 2S(c)). As to the basic cause of the settlement, there are two opinions, called here "Theory A" and "Theory B". Both theories trace the cause of settlement to the instability of the saturated fine sand mass but differ in details.

Theory A. The piles on which Pier 2S is based, rest with their tips in the saturated fine sand mass (Fig. 1), which extends also under other piers supporting the lifting span. If this mass were uncovered from above, the overloading of Pier 2S would cause flow of the mass from underneath that pier and bulging at the surface. But since the mass is covered from above with a slab of sand and gravel, the flow of the mass causes the bending of that slab especially close to its weak point X. (Fig. 1); the uprising of Pier 2N; and its tipping north as shown in Fig. 9. This action extends to piers 3N and 3S but on a slight scale.

Theory B is advanced by an internationally known foundation engineer who examined the case and was also aware of its antecedents. Piles supporting the piers in question had to be driven within a sheet piled cofferdam. The contractor had difficulty in unwatering this cofferdam and the latter was replaced by a tight bottom timber box caisson landed on the tops of the timber piles. Furthermore, piles supporting pier 2S are short; overloaded; and their ends rest on a mass of fine sand. It is admitted that movement of this sand mass away from Pier 2S took place. A combination of all these unfavorable factors caused a heavy settlement of Pier 2S. As to other piers, it is doubtful that they went up. A hypothesis accompanied with a very skillful argumentation has been introduced. Piers 2N, 3N, and 3S did not move upwards but the corresponding bench marks settled about two inches which should increase the total settlement of Pier 2S from twenty-two inches to twenty-four inches. Theory B is unable, however, to explain why piers 2N and 3N tipped north.

7. Conclusions. The behavior of Pier 2S during the first thirteen years of its existence suggests that the saturated fine sand mass either was in process of consolidation like clay or more probably moved plastically (Fig. 7), for saturated fine sand masses apparently may behave as plastic bodies. Actually, plastic flow may be visualized as a slow movement under the action of an unbalanced shear stress. The slow movement element is undoubtedly present in similar cases and the existence of a shearing stress may be proven by the fact that a movement of this kind may be stopped by increasing the shearing resistance of the mass. Dr. Ing. Otto Mast reports a similar case in Germany where the movement of fine sand running from underneath a bridge pier was discontinued by applying the Joosten consolidation process. ("Der Bauingenieur," vol. 15, No.33/34 (August 17, 1934). Obviously the latter process increases the shearing resistance and not the resistance to compression of the material since a saturated fine sand mass is itself incompressible.

In studying this case, a number of questions arose which have remained unanswered. Some of them are: (a) How is the friction resistance distributed along the length of a so-called "friction pile"; (b) is the stress at the tip of a loaded pile a pressure or a shearing stress, or combination of both; (c) how is the load distributed between the soil surface, S, and the piles, P, driven in it (Fig. 5)?

No. F-7

RESULTS OF LONG DURATION SETTLEMENT TESTS

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Introductory remarks. The existing settlement theory supposes that compressible soils demonstrate a definite degree of compressibility so that an increase of loading, after some time has elapsed, simply results in a definite decrease of the porevolume, to an amount, depending upon the properties of the soil and the magnitude of the applied load increment. If, for some soilsample, it lasted some days or even a week and more before final consolidation resulting from the new loading was attained, this circumstance had to be described to a very small degree of permeability and not to secular effects. Recent American publications already mention the observation of a "secondary" time effect and for this reason we supposed, that the results of long-duration tests, performed in the laboratory of soil mechanics of Delft, might be of interest for the purpose of this conference, as it seems that secular effects cannot be neglected in theoretical and practical treatment of the settlement problem.

Observation of settlements. In consequence of the study of time-settlement diagrams of both structures and laboratory samples, if plotted on a semi-logarithmic scale, it appears, that these diagrams may be represented approximately by a vertical line symbolizing the direct compression effect α_p succeeded by a rectilinear slope representing a secular effect α_s, so that

$$z_t = \alpha_p + \alpha_s \log_{10} t \qquad (1)$$

if only the moment of application of the load is taken as time 1.

z_t settlement per unit of thickness of layer or sample dependent from relative thickness thereof.
t time of observation of settlement in minutes.
α_p settlement for 1 kg/cm^2 representing the direct effect and
α_s settlement for 1 kg/cm^2 representing the secular effect of loading for time intervals 1-10, 10-100 etc.

For these intervals z_t increases with equal amounts α_s and of course the value of α_s will depend upon the unit of time we use.
If instead of 1-10, 10-100 minute periods we prefer to use 1 - m, m-10m, 10m - 100m, minute periods, and indicate the ends of these periods as T, 10 T etc, we must put t = m.T, so that

$$z_T = \alpha_p + \alpha_s . \log_{10} mT = \alpha_p + \alpha_s \log_{10} m + \alpha_s \log_{10} T \qquad (2)$$

If we start the diagram at T = 1, situated upon a part of the diagram that is really rectilinear, we can put

$$\alpha_p + \alpha_s \log_{10} m = \alpha_{PI} \qquad (3)$$