GEOTECH-YEAR 2000
Developments in Geotechnical Engineering
27-30 November 2000, Bangkok, Thailand

The geotechnical problems of the Golden Horn - A historical perspective

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SYNOPSIS. With its extraordinary foundation problems, due to the thick alluvium along its shores, the Golden Horn of Istanbul has attracted the interest of engineers for ages. The challenge of the problems of its soils has also inspired Karl Terzaghi to initiate his well-known experimental approach in the field of soil mechanics. The excessive settlements and foundation failures of many structures from different times in this area provide us with a rich data base to understand their causes. This paper examines the foundation problems encountered in the construction of a seventeenth century mosque, an eighteenth century dry dock, two nineteenth century quay walls, a twentieth century building and a recently constructed bridge spanning the deep waters of the Golden Horn.

INTRODUCTION

During his period in Istanbul, Terzaghi had the opportunity to evaluate all the experiences of the past in perspective once more in the corners of his mind (Soydemir, 1973). It was a concentration of enormous intensity. Then came the day: one nice morning in March, 1919 - as he many years later told the incident to Dr. Bjerrum, "... I was sitting in a mood of depression at an old, rustic coffee house overlooking the Golden Horn (Pierre Lotti Coffee House). I suddenly visualised what was needed to obtain a rational approach to the problem involved in earthwork and foundation engineering. I realised that the progress depended entirely on the development of testing equipment and methods which could give a quantitative measure of the mechanical properties of the soils involved. On two sheets of paper, I listed a number of possible ways of testing soils and made sketches of the equipment needed." About the same incident, a year later Terzaghi wrote to Wittenbauer, "... at the beginning of March, 1919, I listed on a single sheet of paper everything we needed to know about the physical properties of clay in order to be in a position to treat the fundamentals of earthwork engineering on a scientific basis. My demand seemed excessive even to myself, and I doubted that I would live to see all the questions are answered."

Along his basic research, in 1921 Terzaghi had a golden opportunity to be involved with an ideal project as a consulting foundation engineer. He wrote about this incident to Professor Peynircioğlu in 1950, "...In 1921, it was in Istanbul, at the site of the steam power plant in Silahtar (situated at the estiuary of the Golden Horn), where I had first opportunity for a practical application of the fundamental principles set forth later in my writings. For this reason, I always considered Istanbul as the birthplace of what I was able to contribute to the scientific development of earthwork engineering."(Peynircioğlu, 1973).

SOIL CONDITIONS

The golden Horn is a 7.5 km. long natural inlet of the Bosphorous Strait. It has a maximum width of about 700 m. near its entrance. Its maximum depth is over 40 m.

The shores of the Golden Horn are completely covered by thick layers of man made fill (Peynircioğlu, 1975). The thickness of the fill is over 40 m. along the south shore and over 30 m along the north shore. The thickness of the fill decreases with increasing distance from the shore. The man made fill layer is generally classified as very loose to loose density. It is underlain by sedimentary layers (it may be called alluvium for simplicity) which are described as gray-dark gray fat organic silty clay of marine origin. It should also be noted that the silt content of the alluvium increases over the depths from 60 to 80 meters below the sea level (Togrol et al, 1991). Therefore a more accurate definition of the alluvium between those depths should be slightly sandy clayey silt. The lower parts of the alluvium is very probably underconsolidated.

The alluvium is generally underlain by few meters of weathered shale and partly by cobbly gravel.The sandstone bedrock is a Palaeozoic formation.

The borings which were carried out along the axis of the new Galata Bridge provided an accurate information concerning the deeper layers.

On Eminönü (south) side of the bridge the alluvium is underlain by a few meters of weathered shale. Over the deeper part of the waterway and towards the Karaköy (north) side the alluvium is underlain by cobbly gravel.

Some properties of the alluvium are given in Table 1.

Table 1. Natural water content and consistency limits of the alluvium.

Property	Mean (%)	Standard error (%)	Number of samples
w_n	49	6	30
w_L	64	8	30
w_p	28	3	30
I_p	44	14	30

The results of Pressuremeter tests in the silty clay layer are,

$$E_m = 680 - 2170 \text{ kPa},$$

$$p_L = 200 - 590 \text{ kPa}.$$

The in-situ vane test results varied between 69 - 124 kPa. The strength characteristics of the layer increased with depth.

The variation of shear strength with depth is given by Akgüner et al (1996) as

$$s_u / \sigma_{v0}' = 0.25.$$

Consolidation characteristics of the alluvium is determined on a number of samples obtained from borings. The results are given at Table 2.

Table 2. Consolidation characteristics of the alluvium.

Depth (m.)	C_c	C_α (Δp = 100 - 200 kN/m^2)	γ_n (kN/m^3)	w_n(%)	C_α / C_c
56	0.45	0.015	16.7	52	0.03
68	0.38	0.008	17.5	42	0.02
74	0.43	0.011	16.9	42	0.03
61	0.56	0.019	17.0	51	0.03
54	0.55	0.015	16.1	62	0.03
58	0.50	0.016	16.7	55	0.03
66	0.56	0.011	16.9	45	0.02

At Table 2, secondary compression index and related settlements are calculated as suggested by Mesri (1973) and Mesri and Godlewski (1977). The ratio between the secondary and primary consolidation indices are calculated by Mesri and Castor (1987) for clays and silty clays over a large interval as

$$C_\alpha / C_c = 0.04 + 0.01.$$

From Table 2 the value of C_α / C_c for the alluvium could be given as 0.03 which is consistent with the values given by Mesri and Castro (1987).

Akgüner et al (1996) has calculated the compression index of the alluvium on samples obtained from the western part of the Golden Horn as,

$$C_c = 0.141 \gamma_s^{1.2} \{ (1 + e_0) / \gamma_s \}^{2.38}$$

Through oedometer tests on alluvium samples at a pressure of 100 kPa (void ratio c.1.00) the coefficient of consolidation is found to be

$$c_v = 3.8 \text{ x } 10^{-8} \text{ m}^2/\text{s}.$$

The sandstone bedrock underlies the whole area of the Galata Bridge crossing. From Pressuremeter tests carried out in bedrock gave the following results;

$$E_m = 1341 - 101\,618 \text{ kPa},$$

$$p_L = 1120 - 2430 \text{ kPa}.$$

The total recovery of 75 mm. diameter cores was low and not exceeding 41 %, with an average of about 20 %. The maximum RQD value was 13% and it was frequently zero. These results suggest the crust of the sandstone i.e. friable and heavily fractured.

YENI CAMI

Yeni Cami (the New Mosque) built between 1597 and 1664 along the Golden Horn (Ülgen, 1942). It is known that the construction of the mosque was interrupted for 58 years due to the foundation problems.The construction continued in 1661. A recent study (Peynircioğlu et al., 1978) has shown that it was seated on a 10 m. thick man made fill which is underlain by 20 m. thick sand and gravel series (alluvium). The depth of the bedrock varies between 40 and 42 meters. Between the sand and gravel series and the bedrock a sandy silt layer was also encountered.The mosque has developed a tilt towards the Golden Horn with an average slope of 0.005. However, no damage is observed in the structure. The average stress induced from the foundation is calculated as 220kPa. Most probably the delay of the construction allowed the soil complete the major part of its consolidation (Peynircioğlu et al, 1981).

EIGHTEENTH CENTURY DRYDOCK

A sharp contest took place between French and Swedish engineers when the construction of drydock at the Golden Horn shipyard was put to tender in late eighteen century (Togrol et al, 1981).

The construction of the drydock (now called Drydock No.3) started in 1796 and took three years to complete. An extension to it built in 1876. In the same shipyard Drydock No.2 built in 1825 and Drydock No.1 built in 1870 employing the same principles of construction adopted in the earlier design.

A number of exploratory borings recently made in the vicinity of drydocks. The soil profile consists of an artificial fill of 4 meters thick that is underlain by a 10 meters thick grey sandy silty clay layer. The sandy silty clay layer rested on the shale bedrock. Upper surface of the bedrock has a slope of 0.10 towards the Golden Horn. Borings also made just off the shore. It is found that the properties of the soft layers overlying on the bedrock on and off the shore are different.

The grey sandy silty clay layer has an average liquid limit of $w_L = 0.50$, plastic limit of w_p=0.27 and the natural moisture content of $w_n = 0.33$. The green silty clay that is encountered just off the shore has average liquid limit of $w_L = 0.90$, plastic limit of w_n=0.45 and natural moisture content of $w_n = 0.60$.

Construction of the Drydock No.3

French and Swedish dock engineers presented their proposals to the authorities in 1796 (Togrol et al, 1981; Müller-Wiener, 1994). French engineers' plan involved sinking a construction caisson after preparing the required channel by dredging and underwater blasting. Then the water inside the caisson would have been pumped out to build the quay walls. This type of construction required a very large caisson. It seems that the French design was based on the experience gained in the construction of Drydock No.1 built between 1774 and 1777 at Toulon Harbour (Noel, 1850).

Swedish engineers proposed to drive sheet piles in order to seal the working area, and to make both the excavation and the construction in a dry pit.

French proposal was found to be 2.2 times more expensive than the Swedish one and the Swedish engineers won the contract (Arne, 1952) In order to determine the exact location of the drydock test pits dug at the shipyard area. The pits were 18 m. by 18 m. with a depth of 10.50m. Through those pits not only the soil conditions but also the choice of the drainage equipment were determined.

Once the location of the drydock was decided wooden sheet piles driven along the shoreline and the excavation started. The size of the excavation was 37.50 m. by 75.00 m. with a depth of 10.50. m. The sides of the excavation were supported by timbering.

Ground water level controlled throughout the construction. Continuous drainage of water from the excavation did not present serious problems except causing damage to two nearby storehouses. Cracks and settlements were observed in these buildings and one was to be demolished later.

The drydock built in the Golden Horn shipyard about two centuries ago is one of the remarkable constructions of the eighteenth century. It is still in use and it was not subjected to any serious deformation or damage. It is an excellent example emphasising the importance of correct assessment of geotechnical problems. If it was built only few meters closer to the shoreline than where it is now, the drydock might have been sharing the fate of many other buildings along the Golden Horn which were heavily damaged by subsidence and slides.

QUAYWALLS

Towards the end of the nineteenth century great lengths of quaywalls built along the entrance of the Golden Horn. Although the necessity for quay walls had been badly felt during the Crimean Campaign (1856) it was not possible to find a willing contractor ready to deal with the site conditions. The 758 m. long north (Galata) quay walls built between the years 1892-1895 and 370 m. long south (Eminönü-Sirkeci) quay walls built between the years 1894-1900 (Togrol et al, 1991).

The foundations of the quay wall consisted of rubble stone. Heavy stone blocks used to construct the wall.The rubble base laid on soft cohesive layers and the average pressure exerted on soil was between 200 and 250 kPa.

From the beginning of the construction of the north quay wall in 1892 till the end of 1893 about 232 000 cubic meters of material for foundations and 157 000 cubic meters of material as backfill were used. The construction continued during the winter of 1893- 1894 despite the unfavourable weather conditions and settlements.On September 1895 the first ship berthed at the quay. The settlement record of the 758 meter long north quay wall is given in Figure 1.
Settlements of the south quay wall started towards the end of 1895, immediately after the construction started . In July 1896 the whole construction was collapsed. In October 1896 the second and in 1898

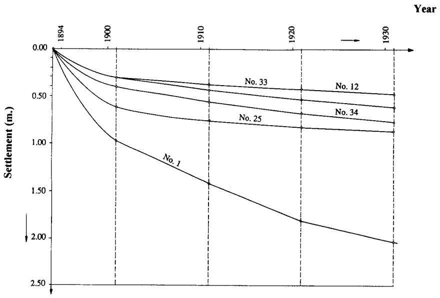

Figure 1. Time-settlement curves of north quay wall (Kann, Suman, 1941)

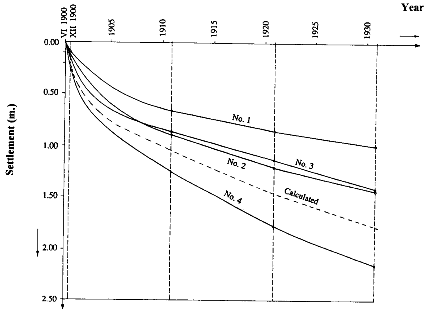

Figure 2. Observed and calculated settlements of the south quay wall (Kann, Suman, 1941).

the third slide took place.The quay wall sunk about 20 meters and the lateral movements reached to 12 meters. The construction of the 300 meter quay wall was completed by 1900. The settlement of the quay wall was 15 cm/year in 1900 and 1.7 cm/year in 1930 (Peynircioğlu, 1978). Difficult soil conditions had made the construction nine times more expensive then envisaged.

The soil profile at the site of the south quay wall is better known. Therefore a settlement calculation is attempted for that location. A 4.00 m. thick rabble foundation is considered and the thickness of the compressed soil is assumed to be 25.00 m. The total settlement is calculated under the load exerted by the quay wall as,

$$s_c = 360 \text{ cm.},$$

And the 95 p.c. consolidation is calculated to be reached in

$$t_{95} = 190 \text{ years.}$$

Calculated settlements are consistent with the observed ones (Fig.2). Secondary time effects could also be evaluated as a creep process. Janbu et al (1989) define the settlement potential that is the product of the observed rate of settlement and time. It was further stated that the ratio of layer thickness to creep potential is of a given magnitude for a given soil. The ratio, r_s, is called creep resistance and its value is given as 10-100 for peat and organic clays, and 100-300 for clays and silts.

Settlements of the Golden Horn quay walls may have a very long time for completion of consolidation. However, settlement potential vs. time curves are levelled off at north quay after eleven years and at south quay after twenty one years. Final values of settlement potential are comparable, which are 0.02 and 0.03, respectively.

If we assume a drainage path of 25 m., creep resistance for south quay wall is calculated as, $r_s = 833$. That value of creep resistance falls within the range of silty sands. That may suggest shorter drainage paths within the alluvium that may be due to permeable seams in the layer.

EXCESSIVE DEFORMATIONS - OLD FRUIT MARKET

With its extraordinary foundation problems, the shores of the Golden Horn for many years served as an open air laboratory for geotechnical engineers. Karl Terzaghi was probably the first geotechnical engineer to use it early this century (Peynircioğlu, 1975). However, the "laboratory" area of the Golden Horn in eighties incorporated into a vast green belt and practically all the tilted or partly damaged buildings were demolished.

The old Fruit-market was a reinforced concrete structure that was constructed in 1935 over a floor area of 6300 m^2. Hinged frames of the main structure were founded on individual footings. The building was demolished in 1986 as a result of the green-belt development project of the municipality.

The footings were rested on the man-made fill layer that has an increasing thickness towards the sea, from 26 m. to 35 m. The fill layer is underlain by a silty clay layer (alluvium) of varying thickness. The depth of the bedrock changes from 33 m. to 69 m. with a general slope of 20^0 towards the sea.

Differential settlements between the nearest and most distant axes to the sea were measured in 1962, 1975 and in 1985 along the column axes (Togrol et al, 1986; 1991). Reliable measurements of total settlements could not be obtained due to the difficulty of securing reference points.

Measurements indicated that the settlements continued over the years (Fig.3). The maximum differential settlements were 255 mm. in 1962, 340 mm. in 1975, and 380 mm. in 1985. The rotations were 2/500 in 1962, 2.7/500 in 1975, and 2.9/500 in 1985. The differential settlements caused considerable distortions and damage in the building. It was interesting however to observe that settlements continued after fifty years of its construction and the building was still in use. If we take the differential settlement as the three-quarters of the maximum settlement then the probable total settlements of the Fruit-market building was at least 340 mm. in 1962, 453 mm. in 1975, and 507 mm. in 1985.

The designer of the fruit-market was compelled to estimate settlement on the basis of semi-emprical rules since an extensive soil investigation had not been available. He was obviously aware of the magnitude of the total settlements. Accordingly the upper structure was designed to withstand distortions. Settlement calculations were made by assuming a loading of 20 kPa on the soil surface. The total maximum settlement was calculated as 100 mm. and the differential settlement as 50 mm. It was considered that the 95 p.c. of the consolidation would be completed within 11 years at the farthest axis and within 140 years the nearest axis to the sea. An additional settlement was expected due to time effects.

Observed and calculated settlements were widely different. When settlements are exceedingly or uncontrollably large it is usually said that the foundation has broken into the ground or has experienced a bearing capacity failure. However, the distinction between excessive settlements

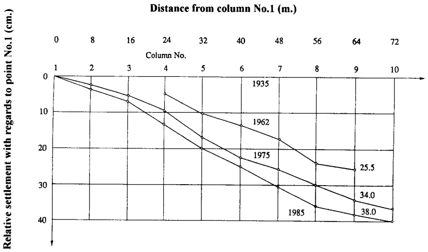

Figure 3. Differential settlements at various points of the Fruit-market.

and failure by breaking into the ground, in many instances, is quite arbitrary as in the case of the old Fruit-market.

CONSTRUCTION OF THE NEW GALATA BRIDGE

The new Galata Bridge spans the deep water harbour of the Golden Horn. A replacement for the decaying pontoon bridge built by the Sezai Türkeş, Feyzi Akkaya Construction Co. The 500 meters long doubledeck bascule bridge has a reinforced concrete superstructure and is supported by 114 large (2 m.) tubular steel piles with lengths extending down to 80 m. below water level (Togrol, 1989).

The design of the new bridge was subject to severe constraints (Togrol et al, 1989). It has a massive superstructure with two decks of 42 m. wide. The lower deck is exclusively for the use of pedestrians and housing some 6000 sq.m. of shops and restaurants. Although a fixed high-level bridge would have posed fewer problems a low deck profile is adopted to permit an uninterrupted view of the historical waterfront of the city.

The tender design was for a reinforced concrete pontoon bridge. The alternative design proposed by the contractor and approved by the authority envisaged the employment of large diameter closed-end tubular steel piles with a relatively thin wall as for the foundations. The piles are designed to carry high working loads (up to 12 000 kN) in end bearing and to comply with the severe restrictions on allowable settlement. The criterion for the maximum allowable settlement of piles is given as residual settlement of 10 mm. after the second application of the proof load of 1.5 times the working load. The severe limitation imposed on the allowable settlement is mainly due to the sensitivity of superstructure to the differential settlements both in longitudinal and transverse directions. The design of the superstructure has led to the arrangement of four-pile bents with a spacing of 22.30 m. The abutments, and the bascule piers located on both sides of the 80 m. wide shipping channel are also massive structures to resist high intensity of seismic loads and impact of shipping.

In the area of the bridge crossing the shores are completely covered by thick layers of man-made fill. The thickness of the fill is more than 40 m. along the Eminönü (south) shore, and more than 30 m. along the Karaköy (north) shore.

The borings that were carried out along the bridge axis indicated that the fill is underlain by sedimentary layers (alluvium) consisting of sand, silt and clay. The layer thins out towards the Karaköy shore and to a lesser extend towards the Eminönü shore. Its maximum thickness along the bridge axis is about 35 m.(Fig.4).

Excluding the 22 driven-and-bored piles on Karaköy (north) side all piles are closed ended with steel conical shoes that are strengthened with concrete. The bedrock is overlain by up to 12 m. cobbly gravel.

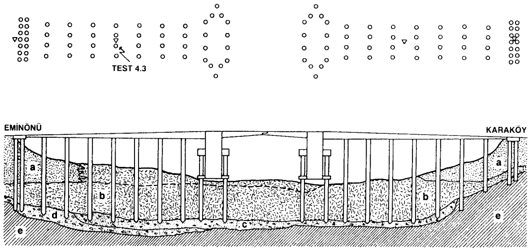

Figure 4. The New Galata Bridge (a. Man-made fill, b. Alluvium, c. Cobbly gravel, d. Weathered shale, e. Sandstone bedrock.)

The adoption of a closed-end pile having a diameter of 2 m. is well suited to the soil conditions and the design requirements - the damage of the pile tip is prevented, settlement criteria are satisfied.

Pile bridge has also the advantage of not being an obstacle to the free flow of water. The sea floor surveys indicated that a 8 m. high threshold was created just below the old pontoon bridge, possibly because of the role of the bridge as a barrier.

A driveability analysis was performed for a number of piles in order to select a pile hammer with sufficient energy to ensure penetration of the pile to the dense gravel and also to ensure that driving stresses were within acceptable limits. The analysis was performed by Frederic Harris, Inc.(1986). The hammer considered for the analysis was Delmag D100 diesel hammer, with a maximum energy of 340 kN/m. Driveability analysis showed that the Delmag D100 Diesel Hammer, operating at maximum efficiency, is capable of driving the 2000 mm.'s diameter pile to an ultimate bearing capacity in the order of 18000 kN. In order to achieve the ultimate bearing capacity of 20000 kN it was necessary to use a drop hammer of 250 kN. dropping from a height of up to 3 m. Pile stresses remained within acceptable limits, i.e. below 90 % of yield for hard driving conditions.

To calculate the soil resistance, laboratory and field values are employed. The end bearing of cobbly gravel is assumed as 12 MPa (for a pile with 2 m. diameter 38450 kN) and in sandstone/shale as 5.9 Mpa (for a pile with 2 m. diameter 18910 kN).

The effects of pile driving in soft cohesive soils are not always easily understood. In spite of the soil disturbance that takes place immediately after a pile is driven into soft clay or soft plastic silt, a recovery in strength is usually expected. There are a few unusual cases, however, in which significantly smaller values have been experienced (Peck, 1961). Such a relaxation effect is also experienced during the pile driving at the Galata Bridge crossing (Tomlinson, 1994).

For the lowest 5 to 10 m. of the penetration the relaxation effect is much more notable. The driving resistance increases steeply to virtual refusal of 400 to 600 blows per 250 mm. and falls back to 100 and 200 blows per 250 mm. when driving re-starts after a waiting period. It has been necessary to make an average of 10 re-drives to achieve the final set of 300 blows for 40 mm. of penetration.

The reduction observed in the penetration resistance is obviously a complex phenomenon. Pile capacity analysis based on penetration records might be misleading by indicating high bearing capacities. The effect of re-driving was observed in alluvium and to a greater extend in its lower part where the silt is the dominant fraction of the soil. One explanation, could be found in Peck et al's (1974) statement, "If the fine sand or silt is dense, it may prove highly resistant to penetration of piles because of the tendency of dilatancy and the development of negative pore pressure during the shearing displacements associated with insertion of piles."

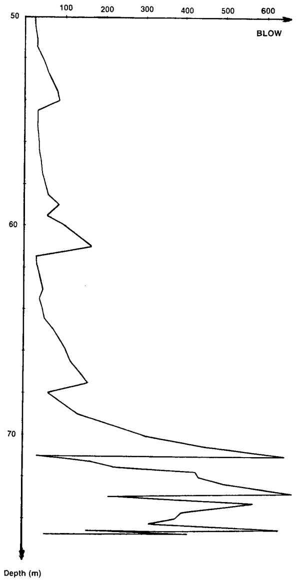

Figure 5. Driving record of the test pile (No.4.3).

Under the ideal conditions it would have been normal practice to drive the piles to sound bedrock to overcome severe settlement requirements (Togrol, 1989). Yet at the design stage it was considered as satisfactory and economical to drive piles close-ended to cobbly gravel layer. However, this assumption was tested by carrying out sufficient number of full scale load tests to secure maximum allowable settlement. One of the test piles had a diameter of 2000 mm., the same as the other piles and a wall thickness of 20 mm. The length of the pile under the water level was 74.58 m. An estimate of the end bearing capacity was made as 121 MN. The test pile surrounded by soft to firm alluvium 47 m. thick. Therefore when the pile is loaded to 1.5 times the working load (to 17.6 MN) about 10 MN of the load is possibly carried by the skin friction. Yet the amount of settlement at the second application of the proof load was the determining factor. When the test carried out the settlement of the pile head after the second loading was as small as 1 mm., i.e. one tenth of the settlement criteria (Figs. 5,6) Similar results were obtained with other test piles.

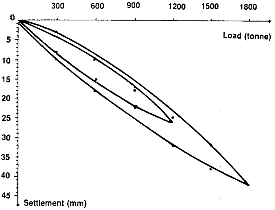

Figure 6. Load test.

CONCLUSIONS

The thick alluvium layers of the Golden Horn area have created a great number of geotechnical problems causing excessive settlements and foundation failures. It has also helped the geotechnical engineers to gain more experience on the behaviour of soft soils.

The foundations along the Golden area suffered large and detrimental settlements. Magnitudes of settlements are often far beyond the estimates made by conventional methods.

It was also interesting to observe that inspite of the large total settlements there were a number of surviving buildings in the area.

Secondary time effects are also important at an area such as Golden Horn. Existing data stretched over many years indicates the importance of that effect. In the consolidation theory the most important discrepancy between the theory and the observations is the secondary time effect or creep. The quay walls and the Fruit-market building had an almost constant rate of creep rate after roughly 20 p.c. of primary consolidation. The rate indicated shorter drainage paths than expected.

Pile driving thorough the alluvium presented a number of interesting problems. Significantly smaller skin friction values observed when piles are re-driven after a temporary interruption. Pile capacity analysis based on driving records might be misleading by indicating high bearing capacities. On the other hand, the relaxation effects might have been exaggerated by using large diameter piles and a pile hammer operating close to its maximum energy output. If a heavier hammer was used for a smaller diameter pile the phenomenon would have gone unobserved. The effect of re-driving was observed in alluvium and to a greater extend in its lower part where the silt is the dominant fraction of the soil.

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