Settlement Observations in Finland

Observations de tassements en Finlande

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Summary

The article deals with settlement observations carried out by the Geotechnical Department of the Finnish State Railways. These observations consider the settlements of buildings, railway embankments, railway culverts and bridges and go partly back as far as the twenties. They cover an exceptionally wide range of soil and load conditions and the observed settlements vary from 0 to about 200 cm. The observations were started by Prof. *Th. Brenner* and Eng. *R. Forsten* and the measurements were made by the later.

The observations show that buildings based directly or indirectly by piles, on "friction soil" of a density which is normal in Finland are as a rule exposed to settlements of a magnitude of about 0 to 5 cm and the differential ratio between the settlement and the greatest total settlement is about $\frac{1}{4}$ to $\frac{3}{4}$. In Finland buildings based on cohesive soil are often exposed to settlements of a magnitude of some 20 or 50 cm or even more and the differential ratio of the settlement to the greatest total settlement seems, for cohesion soils also, to be about $\frac{1}{4}$ to $\frac{3}{4}$. Smaller settlement differences than $\frac{1}{4}$ of the total settlement are rare, greater differences than $\frac{3}{4}$ can occur where the load or ground conditions vary a great deal.

During the first stage of consolidation the course of the settlement can be characterized by a powers expansion. The velocity of settlement under this "power period" is often inversely proportional to the magnitude of the settlement at the same moment. During the later stage of the hydrodynamic settlement period the course of the settlement can be characterised by an exponential expression. The exponent factor (β) gives the ratio of the velocity of settlement to the primary settlement that will occur in the future and its value depends above all on the consolidation coefficient and the thickness of the compressible layer. The velocity of settlement during the "exponent period" is consequently directly proportional to the primary settlement, remaining at the same moment (the future settlement). The hydrodynamic settlement is followed by a secondary (secular) settlement. During this "logarithmic period" the velocity of settlement is inversely proportional to time.

Sommaire

Cet article traite des observations sur les tassements, observations faites par le Département de la Géotechnique des Chemins de Fer finlandais. Ces observations étudient les tassements des immeubles et talus des chemins de fer, tunnel de chemins de fer et ponts, et ont été continuées depuis 1920. Elles couvrent une exceptionnelle variété de sols, de charges, et les tassements observés varient de 0 à 200 cm.

Les observations montrent que les bâtiments fondés (directement ou indirectement sur pieux) sur un sol à frottement dont la densité est normale, en Finlande, sont, d'une manière générale exposés à des tassements d'une grandeur de l'ordre de 0 à 5 cm et le rapport entre la différence de tassement et le plus grand tassement final est d'environ $\frac{1}{4}$ à $\frac{3}{4}$. En Finlande, les bâtiments fondés sur un sol cohérent sont souvent exposés à des tassements pouvant atteindre 20 cm, 50 cm, ou même plus.

Des différences de tassement inférieures à $\frac{1}{4}$ du tassement total sont rares; des différences supérieures aux $\frac{3}{4}$ du tassement total peuvent être observées lorsque les conditions de chargement et de sol varient beaucoup. Pendant le premier stade de consolidation, la variation de tassement peut être caractérisée par une équation de puissance. La vitesse de tassement pendant cette «période de tassement» est souvent inversement proportionnelle à la grandeur du tassement au même moment.

Pendant le premier stade de la période du tassement hydrodynamique, la variation de tassement peut être caractérisée par une équation exponentielle. Le facteur exponentiel (β) donne le rapport de la vitesse de tassement au tassement primaire permanent et sa valeur dépend avant tout du coefficient de consolidation et de l'épaisseur de la couche compressible. La vitesse de tassement pendant la «période exponentielle» est, par conséquent, directement proportionnelle au tassement primaire permanent au même moment.

Le tassement hydrodynamique est suivi par un tassement secondaire (séculaire). Pendant cette période logarithmique, la vitesse de tassement est inversement proportionnelle au temps.

Dwelling-House in Turku

The one-storey wooden house represented by Fig. 1, a dwelling-house belonging to the Finnish State Railways and situated in the city of Turku, was built in May–July 1935. The building is based on continuous footings on loose deposits of muddy clay. The surface-layer of the ground is somewhat firmer in consequence of evaporation, and, in order to make use of the stress-dispersion capacity of this layer, only the course of humus at the top was removed and the footings were placed

Fig. 1 Dwelling-house in Turku
Maison d'habitation à Turku

on a gravel filling layer, about 30 cm thick, at approximately the same level as that of the natural ground surface (Fig. 2). The building was surrounded by a sand-gravel filling, about 1 m thick, in order to protect the footings from ground frost. The soil pressure under the footings is 0.55 kg/cm².

The compressible soil layers consist of a layer of muddy Litorina-clay, rather sandy in its upper parts, with increasing water content downwards and of stratified glacial clay with increasing silt and fine-sand content lower down. The natural water content of the clay, expressed in percentage of dry weight, has about the same value as the liquid limit of the clay, h.e. for the post-glacial clay a maximum of about 150–170, and for the glacial clay about 70–80. The shearing strength of the clay under the firmer surface-layer varies between about 0.15 and 0.25 kg/cm². The thickness of the compressible layers (= the depth of the dense moraine-gravel layer under the clay) varies between 16 and 22 m.

According to settlement observations (Fig. 3) the building has been exposed to settlement of a magnitude of about 30–40 cm during a period of 15 years. The greatest settlement which has been observed so far is at point 9, where the thickness of the compressible layers is the smallest, and the smallest settlement at the opposite corner, where the thickness of the compressible layer is the greatest.

According to theoretical settlement analysis the influence of the filling around the building is very important: the settlement is almost 3 times greater than it would have been, if there had been no filling at all. The settlements have been calculated with a stress distribution according to *Boussinesq* as well as to *Westergaard* with *Poissons* ratio = 0 and according to the empirical relation, found from numerous compression tests, between *Terzaghi* compression index C_c and the natural water content of the clay ($C_c = k \cdot \sqrt{w^3}$, where k—a constant $\cdot\, W/L_w$—varies between 0.5 and 1.5, for Finnish clays, in most cases, $k = 0.85$, $w = W/100$, where W = water content in per cent of dry weight and L_w = liquid limit). The probable primary settlement S_P was calculated according to the formula:

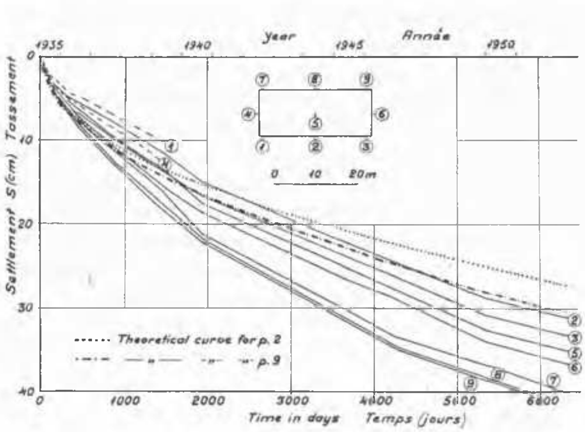

Fig. 3 Observed settlements for the building in Fig. 1. Point 1 and 4 have not been observed since 1939, because this part of the building was damaged through bombing during the war.
Courbes de tassement de la maison d'habitation de la Fig. 1.

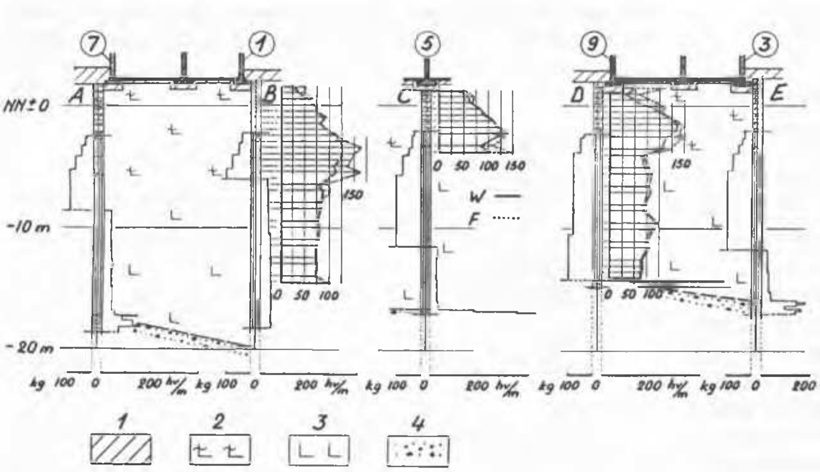

Fig. 2 Soil profiles under the building in Fig. 1. The diagrams to the left of the soundings show the load on the sounding borer (Swedish sounding-method), the diagrams on the right show the number of half-turns per meter penetration (hv/m), when the borer has a load of 100 kg. The number in the rings 7, 1, 5, 9, 3 correspond to the observation points in Fig. 3. Soil signs: 1 = filling, 2 = muddy clay, 3 = glacial clay, 4 = moraine. W = water content in per cent of dry weight, F = relative fineness, corresponds approximately to the liquid-limit.
Couches du sol sous la maison d'habitation de la Fig. 1. Les diagrammes à gauche de chaque trou de sondage donnent la charge employée pour enfoncer la sonde selon la méthode suédoise; les diagrammes à droite donnent le nombre de demi-tours nécessaires par mètre de pénétration à la charge de 100 kg. Les chiffres dans les cercles 7, 1, 5, 9, 3 correspondent aux points d'observation de la Fig. 3. Symboles du sol: 1 = remblai, 2 = argile vaseuse, 3 = argile glaciale, 4 = moraine. W = eau en pourcentage du poids sec, F = pureté relative.

$$S_P = \sum_{0}^{H} \frac{k\sqrt{w}}{\gamma_s + \dfrac{1}{w}} \cdot \log \frac{p}{p_0} \cdot \Delta H$$

where $H = \sum_{0}^{H} \Delta H$ = the thickness of the compressible layer

p_0 = average preloading pressure in the layer ΔH
p = average soil pressure in the layer $\Delta H = p_0 + \Delta p$
γ_s = specific gravity of the soil substance.

The calculated probable settlement for point 2, according to *Westergaard*, is $S_W = 56$ cm and, according to *Boussinesq*, $S_B = 61$ cm, and that for point 9 is $S_W = 53$ cm and $S_B = 58$ cm. The theoretical time-settlement curves for point 2 and 9 (Fig. 3) are calculated with a stress distribution according to *Westergaard* and according to the theoretical consolidation curve for consolidation of an open soil layer with constant excess hydrostatic pressure (u_0) at the moment of loading ($t = 0$). The compressible soil layer is assumed to be homogeneous with an average value of $c_v = 2.7 \cdot 10^{-4}$ cm²/sec of the coefficient of consolidation.

As to shape the time-settlement curves—as seen in Fig. 3—diverge from the theoretical progress of consolidation; they are straighter and have a smaller curvature than the theoretic curves. If the time-settlement curves are expressed on a double logarithmic scale (Fig. 4), it will be found, that they are nearly rectilinear, and that the settlement S_t can be expressed as a function of the time t by means of a power equation of the type $S_t = S_1 \cdot (t/t_1)^a$, where the power factor a has values between 0.63 and 0.70 (S_1 = the settlement at the time $t = t_1$

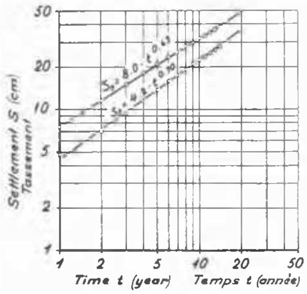

Fig. 4 The progress of settlement for points 2 and 9 in Fig. 3 on a double logarithmic scale. The progress can approximatively be characterized by powers expansion.
Progrès de tassement en échelle logarithmique double de la Fig. 3. Le progrès peut être approximativement caractérisé par des équations de puissance.

= 1 year). The theoretical settlement curve by consolidation of an open layer with rectiliner distribution of the excess hydrostatic pressure u_0, can, during the first stage of consolidation (the power period), be characterized by a power equation, in which the power factor a has the value 0.5 (for settlement curves observed in practice the power factor is almost 0.4–0.6). Since the power factor in this case has a considerably higher value, it can be supposed, that secondary time-effects influence the progress of consolidation already at an early stage. This assumption was also confirmed by the fact that compression tests with muddy clay from the same place showed, after one day, a secondary compression of not less than 87% of the primary compression, the last-mentioned having been determined according to the "square root of time"-method.

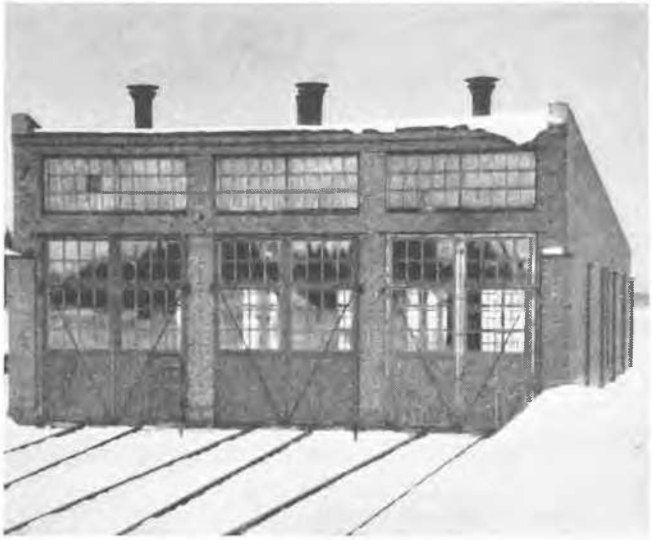

Fig. 5 Locomotive shed in Kerava
Hangar à locomotives à Kerava

Locomotive Shed and Turntable at Kerava

The locomotive shed at Kerava railway station (Fig. 5), 29 km north of Helsinki, was built in the spring of 1933. The building is based on a reinforced concrete mat, 50 cm thick. During the foundation-work the humus layer was removed from the ground surface and the reinforced concrete mat was founded on a layer of gravel filling, about 170 cm thick and about 150 cm above the natural ground surface. The concrete mat and the gravel filling rest upon clay deposits, 9–12 m thick, with varying water contents increasing downwards. The variations of the natural water content, of the relative fineness and of the shearing strength are seen in Fig. 6. Under the clay there is a thin layer of dense moraine, resting upon rock.

The load on the natural ground surface under the filling is 0.65 kg/cm², the permanent load of the building and the gravel filling taken into consideration. At fully traffic load (3 locomotives) the pressure on the ground increases to 0.71 kg/cm². If the probable settlement was calculated for the middle of the

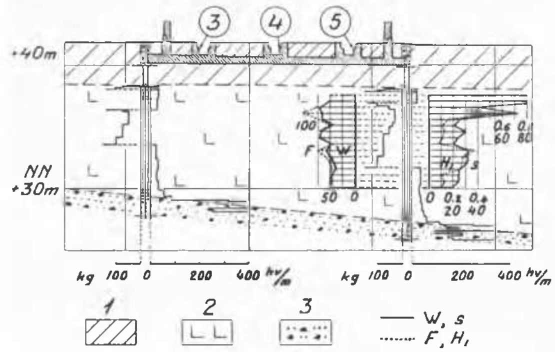

Fig. 6 Soil profile under the building in Fig. 5. The profile is taken near the observation points 3, 4 and 5 in Fig. 7. Signs: 1 = filling, 2 = clay, 3 = moraine. s = shearing strength in kg/cm², H_1 = relative strength value in fully remoulded state.
Couches du sol sous le hangar à locomotives de la Fig. 5. Le profil a été pris aux points d'observation 3, 4 et 5 de la Fig. 7. Symboles sous le hangar à locomotives de la Fig. 5: 1 = remblai, 2 = argile, 3 = moraine. s = résistance au cisaillement kg/cm², H_1 = valeur relative pour sol. remanié.

locomotive shed (point 4, Figs. 6 and 7) according to the natural content of the clay, the probable settlement would be 54 cm. The corresponding calculation according to executed compression tests give the value of 48 cm. In reality the middle of the locomotive shed up until now has settled about 35–40 cm, which means that the calculated values are too high. The progress of the settlement at the different observation points (steel knobs in the concrete mat) is seen in Fig. 7.

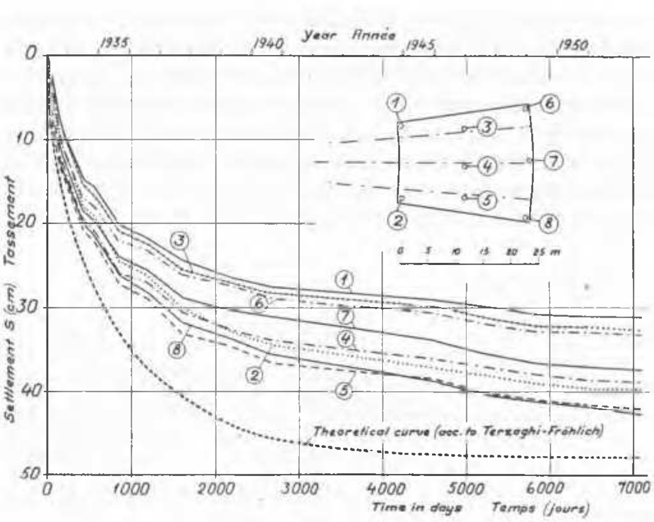

Fig. 7 Observed settlements for the building in Fig. 5 and theoretical settlement curve for the middle of the locomotive shed (point 4). Courbes de tassement observée au hangar à locomotives de la Fig. 5 et courbe théorique pour le point 4.

Since the load area is large in relation to the thickness of the compressible layer, the progress of the settlement should agree with the theoretic progress of the consolidation in an open layer with rectilinear distribution of the excess hydrostatic stress u_0. The corresponding theoretical settlement curve for the middle of the locomitive shed, according to *Terzaghi-Fröhlich*, is also reproduced in Fig. 7; the average value of the coefficient of consolidation for the whole clay layer is taken as $c_v = 1.24 \cdot 10^{-3}$ cm²/sec. As seen from this figure, the actual progress differs from the theoretic one—except in regard to the magnitude of the settlement (the calculated settlement is, as mentioned before, too high)—especially in that the theoretic curve approaches a horizontal asymptote, while the settlement in reality still gradually continues, that is, the curves show a typical "secondary time effect". The observation period, however, is still too short to allow an exact determination of the characteristic of the secondary settlement in this case.

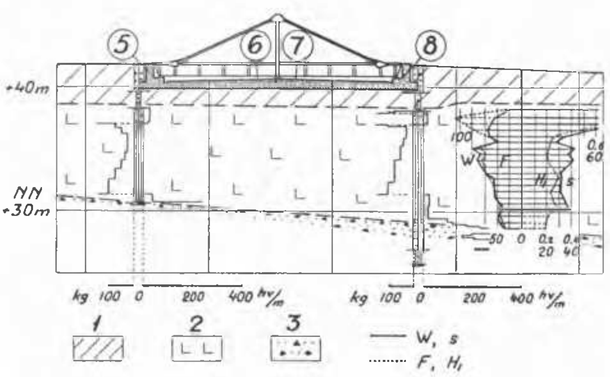

Fig. 8 Soil profile under the turntable for the locomotive shed in Kerava. The profile is taken near the observation points 5, 6, 7 and 8 in Fig. 9. Soil signs: 1 = filling, 2 = clay, 3 = moraine. Courbes du sol sous la plaque tournante du hangar à locomotives de Kerava. Le profil a été relevé aux points 5, 6, 7 et 8 de la Fig. 9. Symboles du sol: 1 = remblai, 2 = argile, 3 = moraine

The ground conditions of the turntable near the locomotive shed agree on the whole with the conditions at the locomotive shed. This turntable also is based on a reinforced concrete mat. The mat is circular with a diameter of 22.4 m and the thickness decreases from 78 cm, at the centre, to 57 cm at the edge of the mat. The mat rests upon gravel filling on a clay layer, 8–10 m thick, which rests upon dense moraine and rock (Fig. 8). The permanent load of the turntable and the gravel filling gives a pressure of 0.47 kg/cm² on the natural ground surface under the filling. At full traffic load (1 locomotive) the pressure is increased with 0.025 kg/cm². If the real loading intensity was taken into consideration there would be almost no influence of the variable load upon the settlement. In consequence of the traffic-vibrations the influence, however, may be of a certain significance.

If a calculation of the probable settlement was made in view of the permanent load alone according to the executed com-

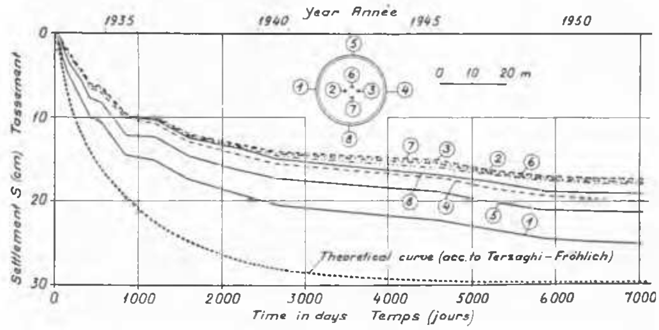

Fig. 9 Observed settlements for the turntable in Kerava and theoretical settlement curve for the centre of the circular concrete mat. Couches de tassement pour la plaque tournante du hangar à locomotives de Kerava et courbe théorique pour le centre de la plaque tournante.

pression tests, a probable settlement of 30 cm would be found for the centre of the circular concrete mat. On account of the variations in the natural water content of the clay at different depths a corresponding calculation would give the value of 37 cm. The centre of the turntable, however, has settled only about 18 cm up till now (Fig. 9) and—in regard to the form of the time-settlement curves)—the progress of consolidation has now reached the final phase of the hydrodynamic settlement period. The calculated settlements thus seem rather high. Since the loading area is large in relation to the thickness of the compressible layer, different assumptions concerning the stress distribution are of very small importance in view of the magnitude of the calculated settlements. The principal error in the calculations thus might have been caused by the values of the volume compressibility being too high.

Railway Culvert at Halikko

The culvert, a stone culvert 38 m long with two 0.80 × 1.80 m orifices, is situated west of the railway station of Halikko in southwestern Finland. It was built in 1926 at a place, where the railway line crosses a narrow valley with clay layers of 15–20 m thickness. The upper, post-glacial parts of the clay are muddy, while the glacial parts are very silty. The liquid limit, for the post-glacial clay, is from 70–80, that of the glacial clay about 50. The natural water content of the post-glacial as well as of the glacial clay are above the liquid limit.

The top of the railway embankment at this place is 8.6 m above the crown of the culvert and in regard to the slight bearing capacity of the ground (the shearing strength of the clay varies in most cases between 0.1 and 0.2 kg/cm²) the culvert was based on wooden piles. (The railway embankment at the culvert is stabilized with the aid of loading berms, and the culvert under the loading berms is lengthened through double concrete pipes; ∅ 1.0 m, see Fig. 10.) In the autumn of 1926 trial loadings were executed with 9 m and 6 m long 8″ cohesion piles and the breaking limit was found to be only 5–6 tons, corresponding to a mantle cohesion of only 0.75–1.0 ton/m². Later on the stone culvert was based on a concrete slab, 38 m long, 8.8 m wide and 1.2 m thick, and resting upon cohesion piles, 10 m long. There are not less than 612 piles under the culvert. At the middle of the culvert the piles are driven down closer to one another (the minimum c/c-space between the piles is only 0.50 m). On both sides of the culvert there are also 209 + 147 = 356 embankment piles, 6 m long, so that the total number of piles at the culvert is 968. If the entire load were carried by piles, the load per pile would be 8–9.5 tons. Since the bearing capacity of the piles, in consequence of the small space between them, is less than that found by trial loadings for a single pile, the pile load—in spite of the great number of piles—is considerably greater than the bearing capacity of the piles. Hence it is to be expected that the culvert, in spite of the expensive pile foundation, will be exposed to great, uneven settlements. This has been fully confirmed by settlement observations. As a matter of fact the foundation

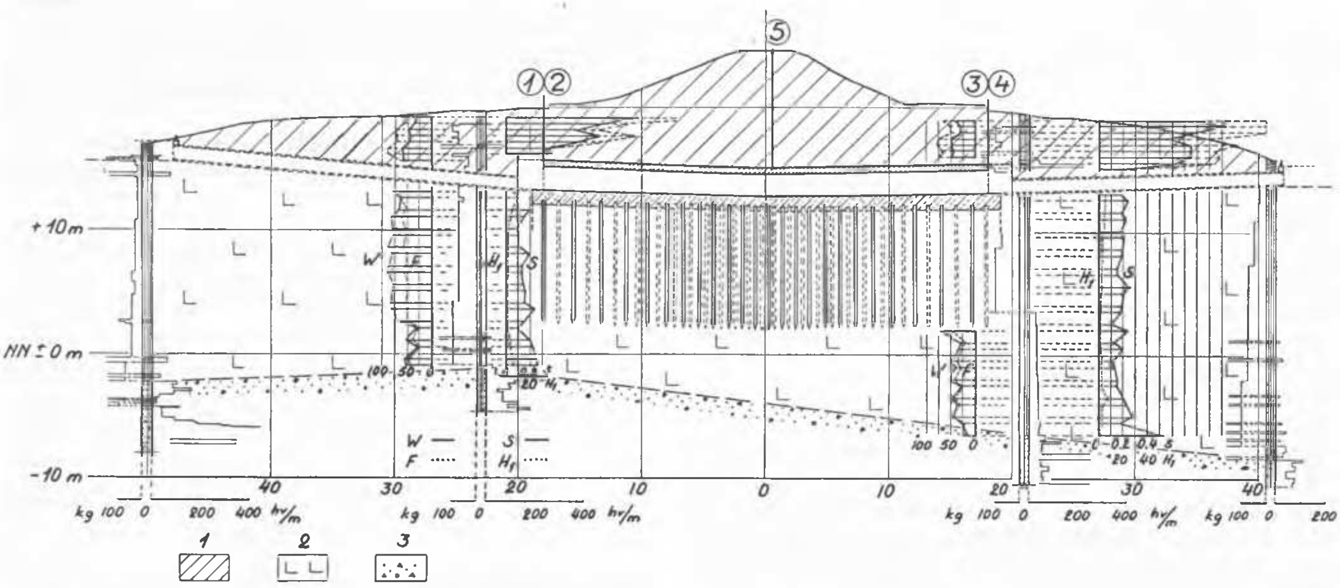

Fig. 10 Soil profile under the railway culvert at Halikko. The number in the rings correspond to the observation points in Fig. 11. Soil symbols: 1 = filling, 2 = clay, 3 = moraine.
Couches du sol sous un conduit souterrain près de Halikko. Les chiffres dans les cercles correspondent aux points d'observation de la Fig. 11. Symboles du sol: 1 = remblai, 2 = argile, 3 = moraine.

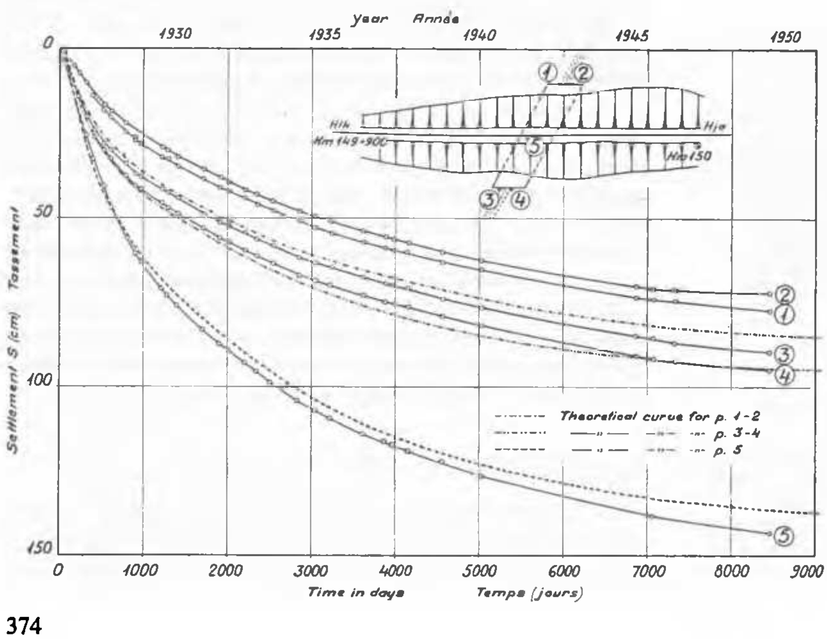

Fig. 11 Observed settlements and theoretical settlement curves for the railway culvert in Fig. 10.
Courbes de tassements observés et théoriques près du conduit souterrain du chemin de fer de la Fig. 10.

in this case must be considered altogether as miscalculated. It affords, however, an instructive example of the great and uneven settlements to which such a construction may be exposed.

Since the pile load is greater than the bearing capacity of the piles, the pile-points are pressed down in the under-lying clay layers and the load thus in fact is transferred direct to the clay layers under the ground plate. In case the settlement was calculated under the assumption that the entire load acted on the clay layers under the ground plate of the culvert, that is to say, in case the problem was dealt with, as if the piles did not exist, the calculated settlement for the left end of the culvert would be 89 cm, for the middle 142 cm, and for the right end 98 cm. These values agree, as seen in Fig. 11, fairly well with the settlements so far observed. When the culvert was constructed, it had been calculated that one part of the load would be carried by the piles, while the rest would be transferred direct to the ground between them. The settlement observations show that this supposition was altogether wrong —a fact which has already been pointed out.

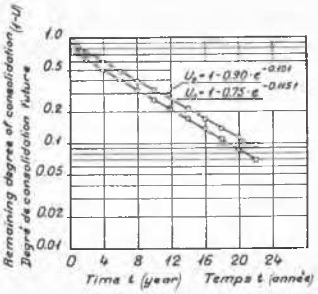

Fig. 12 Relation between the remaining degree of consolidation (1-U) and time (t) on a semilogarithmic scale according to the settlement observations for points 2 and 5 in Fig. 11.
Relation entre le degré de consolidation (1-U) comme une fonction du temps (t) en échelle semi-logarithmique selon les observations aux points 2 et 5 de la Fig. 10.

Investigation of the progress of the settlement shows that the velocity of settlement during almost the whole observation period was proportional to the primary settlement, remaining at the same moment "the future settlement" and that the settlement can be expressed as a function of time by an exponent equation (Fig. 12). The exponent factor varies in this case between 0.10 and 0.115 and the velocity of settlement v_S can thus be expressed by the simple equation $v_S = 0.10$ à $0.115 \cdot S_R$ (S_R = primary settlement remaining at the moment). It can be stated, that the theoretic time-settlement curve during the later period of the hydrodynamic consolidation (the exponent period) can be represented generally by an exponent equation of the type $S_t = S_P\,(1 - b \cdot e^{-\beta \cdot t})$, where S_P = the total primary settlement, b = a constant (almost = 0.6–1.0) depending on the stress distribution, and the exponent factor $\beta = v_S/S_R = \pi^2/4 \cdot c_v/H^2$.

Fig. 13 Station building in Riihimäki
Gare de Riihimäki

Station Building at Riihimäki

The station building at Riihimäki railway station, 71 km north of Helsinki, is a 76 m long brick building (Fig. 13), built in June–November 1933. The ground conditions vary only slightly in horizontal direction (Fig. 14). Under a layer of gravel and sand filling, about 1.5 m thick, there is a clay layer, about 9 m thick, containing organic matter only in its uppermost parts, the remaining clay being of a glacial, markedly stratified type with summer-layers of fine-sand and silt. The natural water content of the clay are near to or somewhat above the liquid limit. The shearing strength in the deaper clay layers is in most cases about 0.10 kg/cm². Close to the

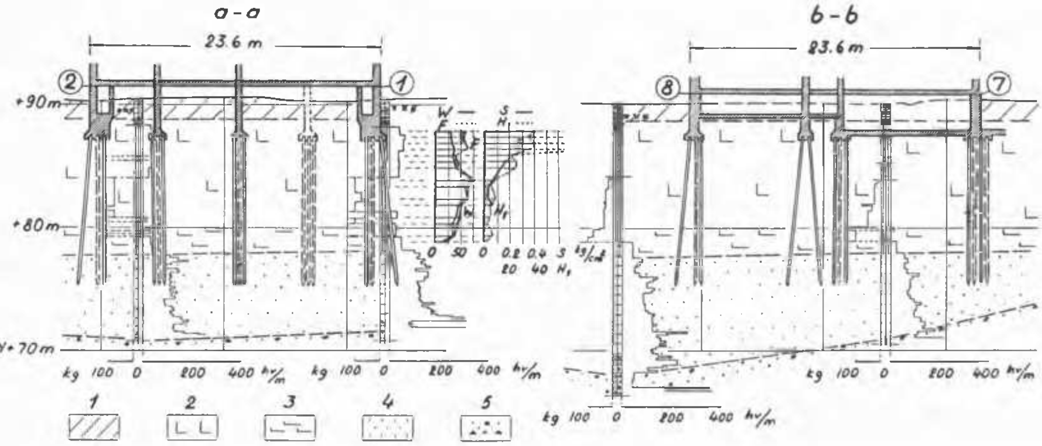

Fig. 14 Soil profiles under the station building in Fig. 13. Profile *a–a* is taken near the observation points 1 and 2, profile *b–b* near points 7 and 8, at either end of the building (Fig. 15). Soil symbols: 1 = filling, 2 = clay, 3 = silt, 4 = sand, 5 = moraine.
Couches du sol sous le bâtiment de la gare de la Fig. 13. Le profil *a–a* est pris aux points d'observation 1 et 2, le profil *b–b* aux points 7 et 8 aux extrémités du bâtiment (Fig. 15). Symboles du sol: 1 = remblai, 2 = argile, 3 = moraine.

bottom-layers the clay is more coarse-grained and changes finally into a silt-fine-sand layer, 1–2 m thick. Between this silty layer and the moraine-gravel, at a depth of 18–20 m, there is a homogeneous layer of fine-sand and sand, 6–8 m thick.

The station building is based on wooden piles. In order to determine the adequate length of the piles and the allowed pile-load a series of trial-loadings was undertaken with 12 m long 8″ piles, which had been hammered down 3.5–4 m into the fine-sand and sand layer under the clay. According to the trial-loadings the elasticity limit of the piles was from 11 to 14 tons and since the maximal load per pile is about 8.5 tons, this pile length (12 m) was considered sufficient.

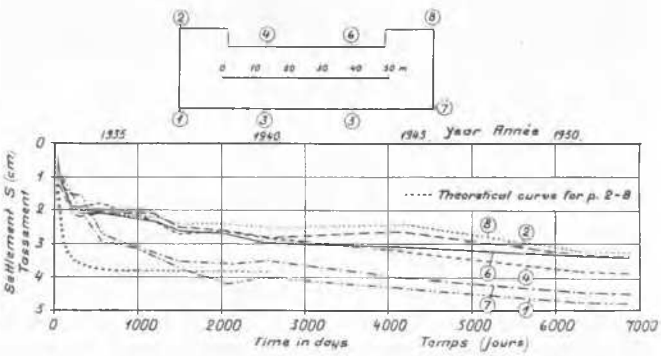

Fig. 15 Observed settlements for the station building in Fig. 13.
Courbes de tassements observés pour le bâtiment de la gare représenté à la Fig. 13.

The size and the progress of the settlement of the different parts of the building are seen in Fig. 15. During the first half year after the completion of the building the settlement exceeded maximal values by about 2 cm, but since then the settlement velocity has been small. The observation points 1 and 7, which lie on the side of the platform, however, have settled a little more, presumably depending on the fact that the floor construction for the platform was not completed until the early part of 1935.

The piles have an average diameter of 22 cm and the pressure on the corresponding cross-section is 22.4 kg/cm². Since the modulus of elasticity in the direction of the wood fibres is about 120.000 kg/cm², the settlement due to the compression of the pile material is only about 2 mm. The settlement thus, practically speaking, depends only on the forcing down of the pile-points and the compression of the heavy loaded sand parts under the points. In spite of the fact that the pile load was chosen after trial-loadings in such a manner, that the maximum load allowed should not be more than about $\frac{3}{4}$ of the elasticity limit, the building has settled 3–5 cm, hereof 1–2 cm after the completion of the building (this after-settlement presumably depends principally on the influence of the railway traffic vibrations). If the probable settlement is calculated under the assumption that the entire load will act on the pile-point level and if the average volume compressibility of the fine-sand and sand layer according to the soundings is estimated at $m_v = 0.02$ cm²/kg, the probable settlement will be about 3.8 cm. The corresponding theoretical time-settlement curve for points 2 and 8 is seen in Fig. 15.

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