No. D-1

COMPARISON BETWEEN CONSOLIDATION, ELASTIC AND OTHER SOIL PROPERTIES ESTABLISHED FROM LABORATORY TESTS AND FROM OBSERVATIONS OF STRUCTURES IN EGYPT

Gregory Tschebotareff, Dipl.Ing.(T.H.Berlin); A.M.I.Struct.E.(London)
Research Engineer in the Foundation Soils Research Laboratory

Settlement studies for purposes of future forecasts primarily involve systematic collection of settlement data and its semi-empirical correlation with physical soil properties established by laboratory tests. Work done here in this direction is described in Paper F-1.

The second stage in settlement studies is the attempt to forecast the behaviour of structures directly from laboratory soil tests and computations.

Locally the most practically important items of such forecasts are :

(1). Rate of settlement of structures.
(2). Final value of settlement.
(3). Distribution in plan of settlement.
(4). Remoulding effects on different clay types, in connection with pile driving.

The distribution in plan of settlements is discussed in Paper No. E-1. I shall now examine the remaining items, comparing theoretical and observed results and giving developments made to usual methods.

(A) Comparison of Theoretical and Observed Time-settlement Curves

Before comparing settlement coefficients derived from laboratory tests with those estimated from observed buildings still settling, the settlement rate must be examined.

The shape of time-settlement curves is governed primarily by soil permeability, as established in mathematical form by Dr. Terzaghi.

Local formations seldom have uniform permeability vertically. Therefore, for first comparison, theoretical time-settlement curves were computed for limit and average permeability coefficients "k_o" established by tests.

Further, time-settlement relations depend on other varying values, such as compressibility coefficient "a", determined from oedometer tests. These may give excessive values and their accuracy has to be estimated by means of the time-settlement curve itself, to obtain the final settlement value.

Therefore, I adopted following procedure. Trial comparisons were made, for different assumed values of "a", until approximate agreement with "a" value estimated from observed and correlated theoretical time-settlement curves was reached.

To accelerate computations, the formula published in "Proceedings, Amer.Soc.C.E.", May 1933, for time "t" until percentage of consolidation "Q" is reached corresponding to coefficient "N", with layer thickness "h_1" (one surface of unwatering), was reduced to metrical measures:

$$t = \frac{h_1^2 \times a \times 10^{-5}}{1.3 \times (1+e')^2 \times k_o} \times N \text{ years} \qquad (1)$$

and represented graphically (Fig. 5).

Basic diagram (A) made for constant value of $e' = 1.00$; $a = 0.02 cm^2/kg$; allows to find directly value of "t" corresponding to Q = 90%; N_o; and any values of "k_o" and "h" (two surfaces of unwatering). Diagrams (B); (C); (D); give corrections of "h" for variations of "a"; "e'"; "h_1"; "X", etc. Diagram (E) in connection with (A), establishes values of "t" for any value of "Q" or "N".

Combined permeability and compressibility tests take a long time. Few samples can be tested. Liquid limit ("w_1") tests, done rapidly, found useful for rough estimation of permeability. Fig. 6 gives relation established between "k_o" and "w_1". For all disturbed and undisturbed plastic samples it appears to follow definite law. Samples with root openings formed exception. Also brittle samples damaged by extraction. Therefore "w_1" gives check on real permeability of latter.

Fig. 7 gives observed and theoretical time-settlement curves for building "I", (see Paper No. F-1). Subsoil water level variation is small and has no noticeable effect here. Theoretical curves for clay layer were laid through two observed end points of period marked "a" when live load was fairly constant. Observed curve agrees with theoretical for $k_o = 2 \times 10^{-7}$ cm/min. Laboratory test value was 5×10^{-7} cm/min. Good agreement of order of dimension.

Building "II" is founded on permeable soil. Settlements should have stopped by 1936, but continued, presumably because of weekly variations of live load.

Building "III" is on similarly permeable soil. Good agreement. Settlements stopped as expected theoretically, except points "26" and "28" with repeated live load variations.

The remaining buildings were observed for too short time to make comparisons.

(B) Comparison of Settlement Coefficients "X" Derived from Laboratory Tests and from Settlement Observations

The first buildings observed showed smaller settlements than expected from laboratory tests.

To facilitate direct numerical and graphical comparison between observed and laboratory values of soil compressibility, the following method was substituted for usual procedure of settlement computation from voids ratio - pressure curves. The compressibility coefficient "a" = $\Delta e / \Delta p$, derived from above curves, does not give a directly comparable relation to settlements.

A new coefficient, which I termed settlement or compression coefficient "X", was used. It represents the compression in centimetres of a soil layer one meter thick under an average stress of 1.0 kg/cm^2 and is expressed in %.

Its average value, based on observed settlements, ("X_o"), for a compressible layer of thickness "h", is computed from: $$X = \frac{s}{\Delta p \times h} \tag{2}$$

where: s = final settlement value.

Δp = average stress on layer due to weight of buildings.

The value of "X", established by laboratory tests, ("X_1"), is computed from:

$$X_1 = \frac{a}{1+e'} = \frac{\Delta e}{\Delta p \times (1+e')} \tag{3}$$

$$= \frac{\Delta h - \Delta h'}{h' \times \Delta p} \tag{4}$$

$$= \frac{1}{E} \tag{5}$$

Δ h and Δ h' represent the compression of sample after and before load stage with stress increase "Δ p" for which "X" is computed, and h' the thickness of sample before it.

The method of computation from equation (4) has advantage of allowing the determination of "X_1", immediately after each load stage, directly from dial readings and estimated initial thickness of sample, with less than 3% error. The values of "X_1" are plotted against mean range pressure of each load stage, (See Fig. 8, 9, 10, 12).

The coefficient "X" may be considered as corresponding to inverse value of Young modulus "E".

Such graphical representation showed that for all samples tested the first run of loading invariably gave much higher values of soil compressibility, expressed by "X", than was actually observed. For silty materials, slightly dislocated by extraction, "X_1" was 3 to 4 times higher than "X_o". But even for stiff brown clays, extracted entirely undisturbed, "X_1" of the first run of loading was higher by 80% to 100% than actual observed value of "X_o".

This was attributed to swelling of clay during extraction below water level. Nevertheless examination of voids ratio - pressure curves in undisturbed, also in entirely disturbed state, (Fig. 11, stiff brown clay), showed that it had been precompressed, presumably by repeated drying, to over 9.0 kg/cm^2. (See also Paper No. C-1, by Dr. W. S. Hanna, concerning formation of local deposits).

A repetition of natural precompression was attempted in oedometers, by repeated loading to 6.0 kg/cm^2 and entire unloading, (sample tested under water). The second run gave smaller values of "X_o" than the first; all following (3rd; 5th; 7th.) compression and expansion runs after complete unloading gave approximately the same values as the second, but still their "X_1" was higher by 60% than the observed "X_o" (= 0.7% to 0.8%) of the building (No. "I").

Unloading only till pressure corresponding to weight of upper layers for intermediate runs (4th; 6th; 8th;), and loading again gave approximate agreement with observed value. (See Fig. 8.)

Therefore, in such precompressed clay materials further consolidation in the ground appears negligible and influence of elastic reversible swelling deformations predominates.

Up to about 1.0 kg/cm^2 their "X_1" from compressive strength tests has same order of dimension (See Fig. 13) as from oedometer tests.

The rate of loading was not found of importance. (See Fig. 8, 2nd specimen, loaded at rate of 0.08 kg/cm^2 per 24 hours.)

Other samples (Fig. 9, 10, 12), disturbed and undisturbed, showed similar behaviour, although 2nd specimen, Fig. 12 was precompressed to 18.0 kg/cm^2.

Siltier material, (Fig. 10), gave by this method slight permanent sets, not apparent from "X" diagram, and approximate values of "X_1" = 0.3% to 0.4% corresponding to estimated observed value of "X_o" 0.3% to 0.5% for building "II" with similar soil.

Building "III", although partly on clay material, but of a brittle type, gave observed value of "X_o"= 0.3%. Dark clay layer beneath was very thin (about 1 meter).

Building "IV" has short period of observation nevertheless showing that it will have higher "X_o" values. Not yet clear whether due to considerable water level variation or to presence of 3 meter thick dark clay layer.

This clay type has higher water content and does not seem precompressed to same extent as brown clays. First loading in oedometer still gives improbably high values. Investigation of dark clay properties not completed. Upper limit of "X_o", roughly estimated from other observations, should not exceed 1.5% to 2.0%.

Remaining buildings observed for too short period to allow comparisons.

I express my acknowledgements to Kamal Khalifa Eff., Dipl.R.S.E., for assistance in carrying out tests and computations.

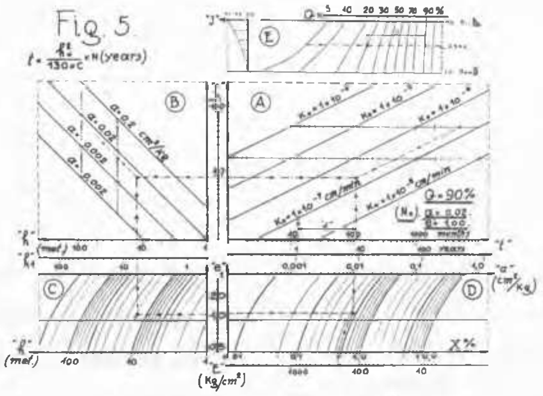
(C) Different Effects of Remoulding on Two Local Types of Clay Possible Relation to Pile Bearing Capacity

Compressive strength tests, (unconfined lateral expansion), were made on about 40 samples from 6 different localities, both undisturbed and entirely remoulded. Remoulding was done under slight compression, involving about 10% decrease of volume.

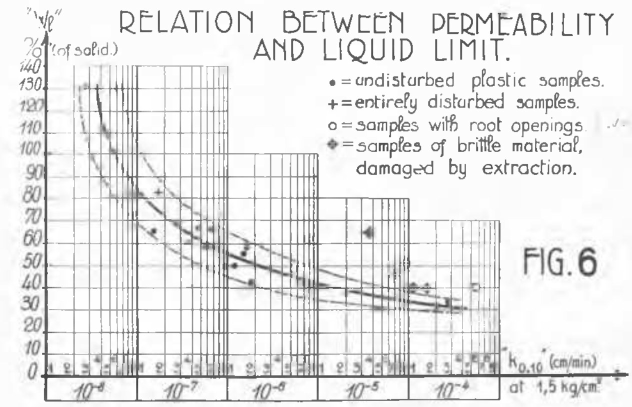
I found entirely different behaviour of two clay types. The compressive strength "q_d" of brown clays was either increased (brittle material) or unchanged (plastic material), the strains of remoulded samples being approximately equal or smaller than those of undisturbed samples.

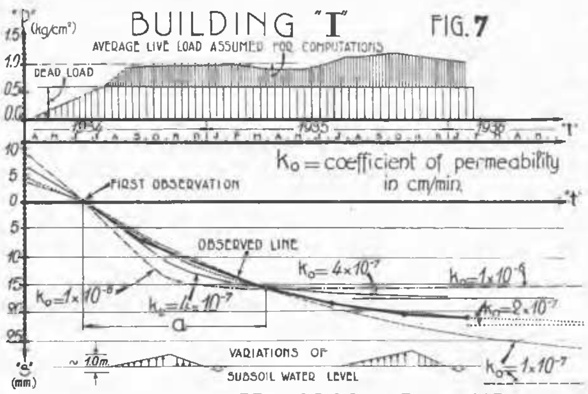
The dark remoulded clays had smaller "q_d" and greater corresponding strains than the undisturbed.

Fig. 14 and 15 show the different effect of remoulding on samples of dark (stiffest specimen so far met) and of brown clay, with approximately the same "q_d" when undisturbed, and the same liquid ("w_1")

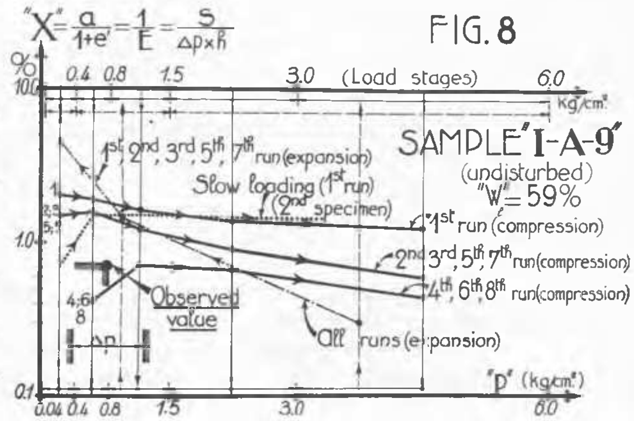


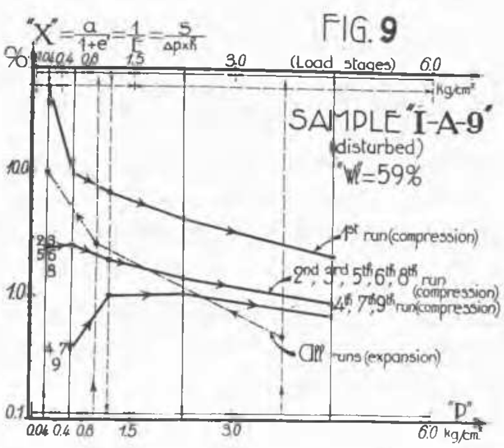
GRAPH FOR COMPUTATION OF TIME-SETTLEMENT CURVES

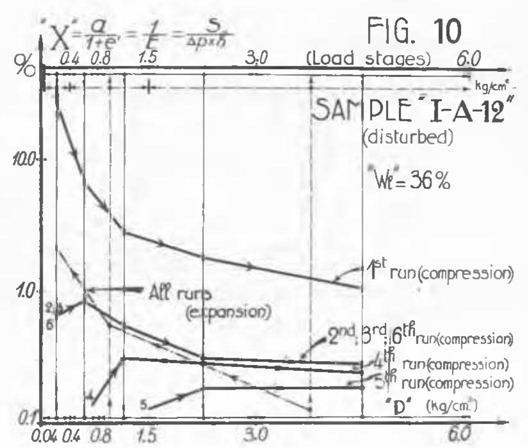


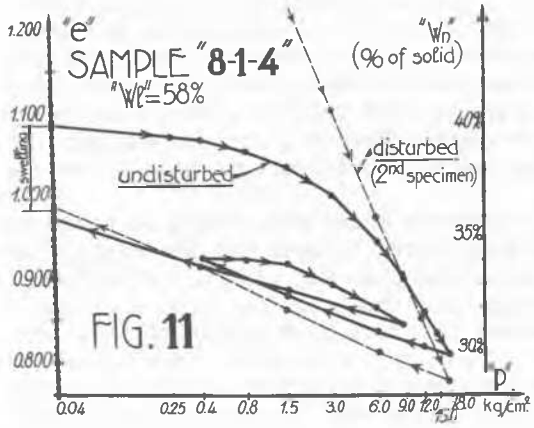


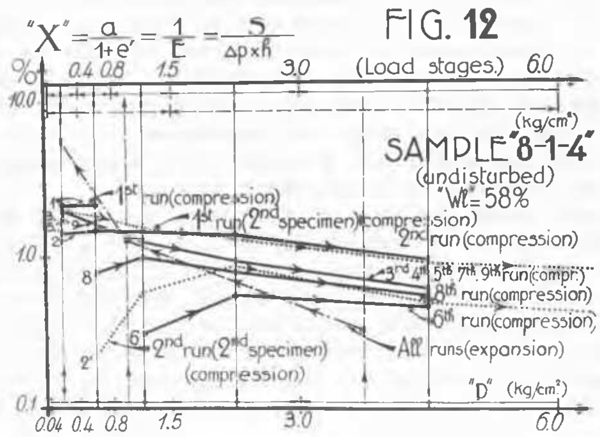
RELATION BETWEEN OBSERVED AND THEORETICAL TIME-SETTLEMENT CURVES.

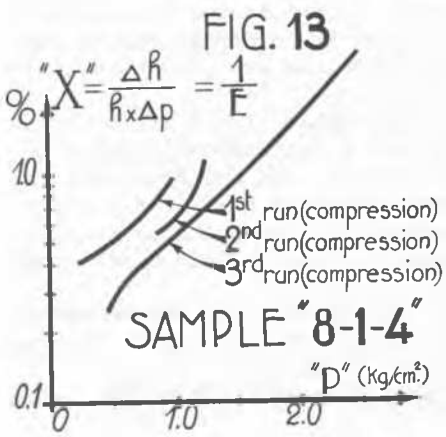


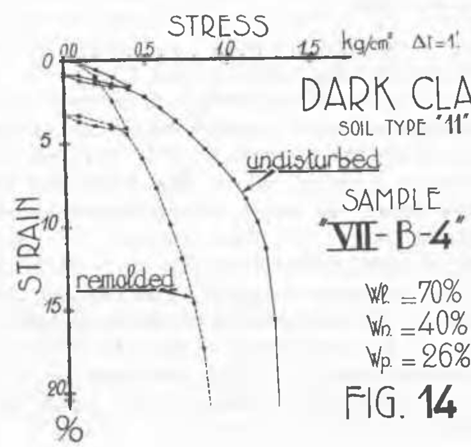


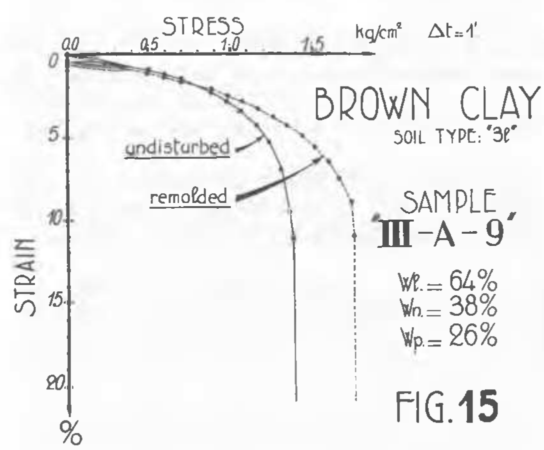












and plastic ("w_p") limits and natural water content ("w_n").

Reasons for this difference under investigation. (See Dr. W.S. Hanna's Paper, No. C-1.)

A past case is known to me where two different types of situ-cast piles, driven under normal resistance during ramming of tube into layer of dark coloured clay material, both failed, when tested, at fraction of load they normally carry in brown clays. There may be relation to remoulding phenomena. Investigation undertaken.

Conclusions.

The rate of settlement of buildings so far observed agrees with order of dimension of theoretical rate computed from permeability tests. Method for facilitating comparisons developed. Repeated live load and water level variations can affect rate of settlement.