Consolidation with Sand Piles of Soft Clays in Levee Foundation

Consolidation d'Argile Tendre dans la Fondation d'une Digue au Moyen de Puits de Sable

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Summary

This paper illustrates the principles adopted in the planning and construction of an important soil consolidation project involving the construction of drains, and stresses the technical-economical importance of continuous soil sampling and appropriate use of the penetrometer during the design and execution of the work.

Sommaire

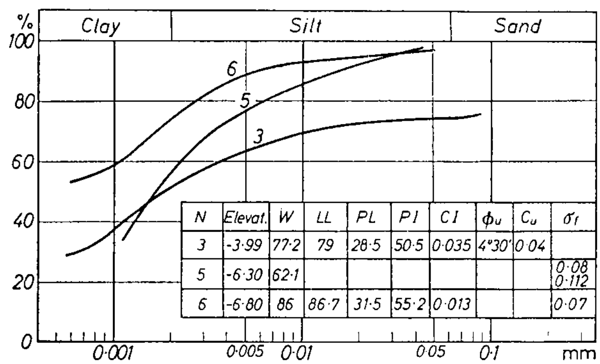
La présente communication expose les principes adoptés pour l'étude et la construction d'un important ouvrage de consolidation du sol par la construction de drains. Les auteurs font ressortir l'importance technique et économique du prélèvement continuel d'échantillons et de l'emploi approprié du pénétromètre pendant la période d'étude et au cours des travaux.

The floods of the river Arno cause considerable damage to the city of Pisa and the surrounding countryside; therefore studies were begun some time ago on the construction of a flood discharge canal which, from Pontedera (21 km above Pisa) would discharge the flood water directly into the sea. It was recently decided to carry out this work, and it has now been almost entirely assigned to contractors.

During the execution of the work many serious difficulties were encountered owing to the unfavourable nature of the soil. The central part of the lower portion of the discharge canal cuts through a very soft shore deposit covered by a dry surface crust. This deposit is a silty clay with a water content very near the liquid limit, and the unconfined compression strengths are as low as 0·08 kg/cm² (see Fig. 1). The deposit has a thickness varying from 6 to 18 m and rests upon a bank of fine sand (Fig. 2). After the right-hand levee (23 m wide at the base, 4 m wide at the summit and about 4 m high) had been built and the upper layers of the soil had been excavated to form the canal two slides occurred, each 200 m long: the first, in September 1953, towards the discharge canal and the second, in June 1954, towards a nearer and older canal called Fossanova, at a section where the levee was only partially completed. The profile of the soil after the slides is shown in Figs. 2 and 3.

From the results of soundings carried out after the slide it was decided to begin deep consolidation in two test sections using the well known method of sand piles. For these tests 40 cm diameter sand piles spaced 4 m apart were installed, using equipment for making bored piles.

During this work the soil was studied more thoroughly by continuously taking samples during the boring by means of a very thin-walled sampler which, once the sample was obtained, was used as a container. On characteristic samples more complete tests were made in the Geotechnique Laboratory of the Construction Science Department of the Polytechnic Institute in Milan (see Fig. 4).



N	Elevat.	W	LL	PL	PI	CI	ϕ_u	C_u	σ_f
3	-3·99	77·2	79	28·5	50·5	0·035	4°30′	0·04	
5	-6·30	62·1							0·08 0·112
6	-6·80	86	86·7	31·5	55·2	0·013			0·07

Fig. 1 Grain size curves of some samples and their index properties
Courbes granulométriques de quelques échantillons et leurs propriétés caractéristiques

The continuity of the sampling proved to be of fundamental importance in this case; without it the final plan would have been out of all proportion to the actual needs. On examining all the samples, after removing the superficial remoulded layer, many pervious layers of clean shells having a thickness varying between a few mm and several cm were found. In the most

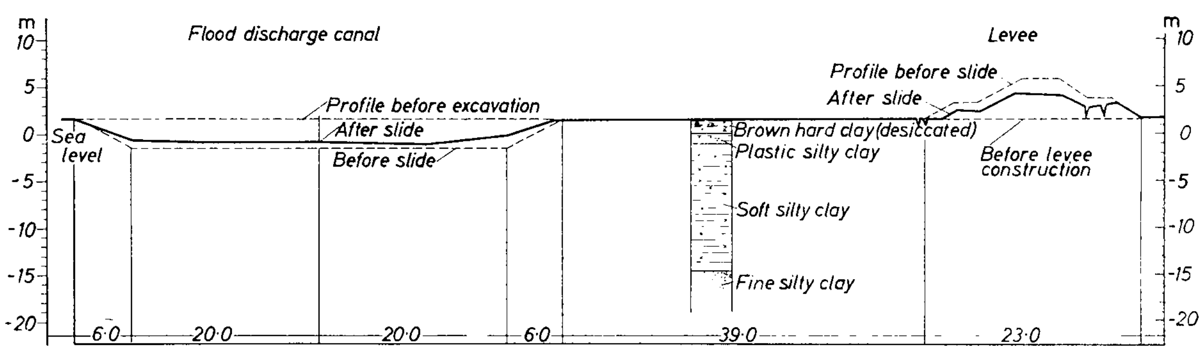


Fig. 2 Cross-section of the first slide and summary of soil profile
Section du 1er glissement et profil d'un sondage

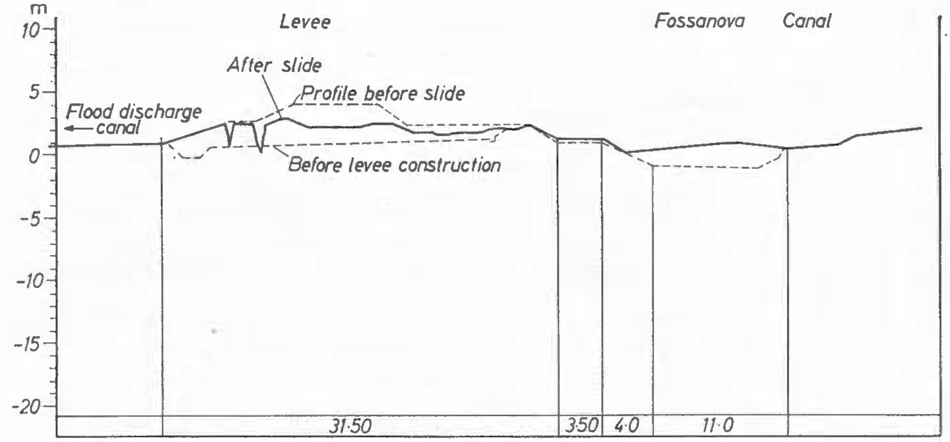


Fig. 3 Cross-section of the second slide
Section du 2me glissement

	γ_s	W	LL	PL	PI	CI	m_v cm²/gm	k_w (Horizontal flow) cm/sec	k_w (Vertical flow) cm/sec	σ_f kg/cm²	Quick direct shear test: τ_f (σ=0·61)	Quick direct shear test: σ_f Remoulded after shear test	20% consolid^{ed} quick shear test: τ_f (σ=0·61)	20% consolid^{ed} quick shear test: σ_f Remoulded
Grey silty clay														
2·00 m														
Peat in clay														
Fractured shells														
Thin shell. Strata in soft silty clay														
-3·00 m														
Thin shell strata														
Soft silty clay	2·605	67·1	81·6	30·2	51·4	0·281	$0{\cdot}87.10^{-4}$ $0{\cdot}71.10^{-4}$	$11{\cdot}22.10^{-9}$	$6{\cdot}58.10^{-9}$	0·112	0·135	0·0845	0·150	0·099
Shells													-	
Shells in soft clay		69					$0{\cdot}872.10^{-4}$ $0{\cdot}722.10^{-4}$	$8{\cdot}80.10^{-9}$	$6{\cdot}70.10^{-9}$	0·150	0·175	0·071		0·101
4·00 m														

Fig. 4 Cross-section showing the position of shell lenses in the soft silty clay and the laboratory tests carried out on the most important samples
Section indiquant la disposition des lentilles de coquilles dans l'argile limoneuse tendre et les essais de laboratoire exécutés sur les plus importants échantillons

important part of the soft clay, which extends from about 2 to 8 m below the surface level (i.e. 1 to 7 m below sea level), these shell layers were on the average about 50 cm apart. These layers could not have been continuous because, being so closely spaced, they would have quickly drained the soil, discharging the water in the lateral unloaded zones and the slides would not have occurred.

The shells probably occur in lenses but it can be presumed that the arrangement of the sand piles is such that all the lenses will be intercepted.

The problem was studied on the basis of this hypothesis considering both the vertical permeability between the lenses of shells, considered as continuous, and the horizontal permeability towards the sand piles, since both are important. The behaviour of the two test sections carried out was interpreted on the basis of the studies of Barron (1948).

The settlements measured at the centres of the levees were found to agree reasonably with those calculated, assuming the distance between the continuous pervious layers to be 160 cm, as against the 50 cm found in the borings in the first 8 m, and a

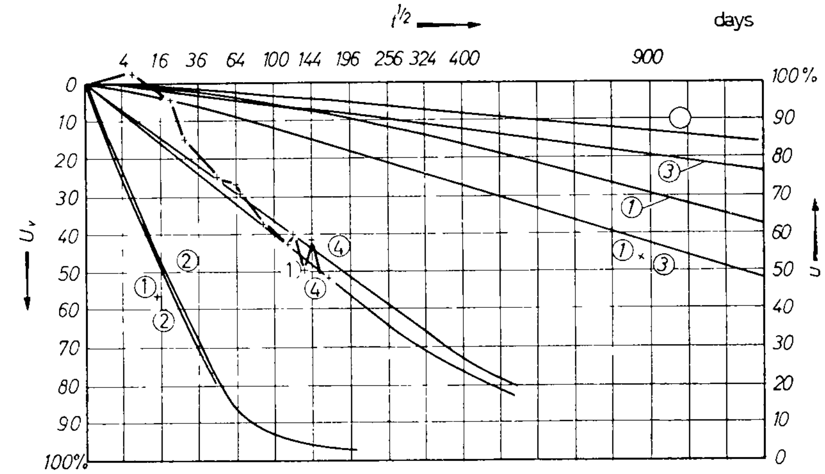


Fig. 5 Relation between per cent consolidation and $t^{1/2}$ with various thicknesses of impervious soil and comparison with actual consolidation

○ Vertical flow only with open layer $2H = 13$ m thick; 1 radial flow only with drain diameter 40 cm, spacing 4·00 m; 2 vertical flow only $2H = 0{\cdot}5$ m; 3 vertical flow only $2H = 9{\cdot}0$ m; 4 vertical flow only $2H = 1{\cdot}6$ m; 1 + 2 both radial 1 and vertical 2 flow; 1 + 3 both radial 1 and vertical 3 flow; 1 + 4 both radial 1 and vertical 4 flow; + — + — + from settlement record under the central part of the levee

Relation entre la consolidation et $t^{1/2}$ avec différentes épaisseurs de sol imperméable et comparaison avec la consolidation réelle

○ Drainage vertical seulement avec épaisseur $2H = 13$ m; 1 drainage radial seulement avec pieux 40 cm distants de 4·00 m; 2 drainage vertical seulement $2H = 0{\cdot}5$ m; 3 drainage vertical seulement $2H = 9{\cdot}0$ m; 4 drainage vertical seulement $2H = 1{\cdot}6$ m; 1 + 2 drainage radial 1 et vertical 2; 1 + 3 drainage radial 1 et vertical 3; 1 + 4 drainage radial 1 et vertical 4; + — + — + Consolidation correspondant aux tassements mesurés au centre de la digue

greater distance (not well defined owing to the difficulties presented by the soil and the system of sampling) at depths of 8 to 16 m (see Fig. 5).

Probably the difference between the actual and calculated distance was affected not only by the greater distance between the deeper lenses but also by the extreme thinness of some lenses, the permeability of which were therefore no longer comparable with the infinitely high theoretical assumption; another reason is probably the fact that not all the smaller lenses may have been tapped by the sand piles.

The final plan, based on the 160 cm spacing between shell layers, was to construct the drains (with a diameter of 40 cm) at a distance of 4·60 m from one another, and then the work would be almost complete.

The length of levee to be consolidated in this manner is 10 km and the extent of the consolidation work both in length and in depth was determined according to the soil-consistency data acquired by the use of a dynamic penetration test.

For this purpose it was decided to employ a penetrometer described by Terzaghi and Peck (1948) of 51 mm diameter, with a 73 kg hammer, a 75 cm height of fall, and surrounded by a casing.

The consolidation work was limited to the section of soil where the penetrometer sank more than 30 cm per blow (the maximum penetration was 105 cm per blow, but penetrations of 70 to 80 cm were very frequent). The maximum depth of the drains determined in this way was 12 m. The penetration tests were generally made at a distance of 150 m from one another, decreasing to 75 m in the softer places. These tests indicated changes in the soil characteristics in spite of the irregular nature of the shore deposit.

The systematic check on the soil properties on the other hand was simply and economically made by taking samples with an 8 cm auger-type bit. These samples, taken down to depths of 20 m, gave sufficient information to confirm the results of the penetration tests.

The dynamic penetration test was chosen both for its great simplicity and because it needs only a few men, with a foreman, to execute the work. The use of a casing proved to be of fundamental importance because the soil often stuck to the rod, considerably increasing the resistance against penetration.

Fig. 6 shows some diagrams of penetration indicating by abscissae the penetration in cm per blow; these diagrams show that the first hammer blow, after the driving-in of the casing, thrust the point into the soil a depth twice as great as that obtained by the following blows. The penetration diagrams have been drawn in these cases using the penetration of the first blow only of each rod section.

Control of the Settlement

The soil settlement under the levees was measured with aligned settlement pegs and piezometers placed in groups of 3 and 5 from each control section. The pegs proved that very limited horizontal movement occurred. The piezometers gave no help because they were mostly affected by the presence of underground gas, so that the pressures rose and fell without any clear reference to the progress of consolidation. The work was therefore followed almost exclusively by means of the data given by the pegs and with the help of a few checks on the variation in soil consistency made once more with penetration tests (see Fig. 6).

The peg method, however, also presented some surprises, because several times inexplicable local heaves occurred (see Fig. 5). This phenomenon was present in almost every section, and it was noticed that it always happened after rainfall and disappeared at the next levelling, a week later.

We impute these risings to the presence of gas under pressure, as already shown by the piezometers and borings. This gas discharges into the atmosphere through tiny channels in the soil, and when these channels are closed by rain water the gas pressure increases and thus causes the heave.

The reference points used in the levelling were probably not affected by this phenomenon or, if so, to a much smaller extent, since they were all placed along the slope of an old levee at least 100 m from the new levees. The surrounding soil not being disturbed by excavations and embankments, the rain water probably ran off more rapidly, penetrating less deeply and therefore causing less disturbance to the gas discharge.

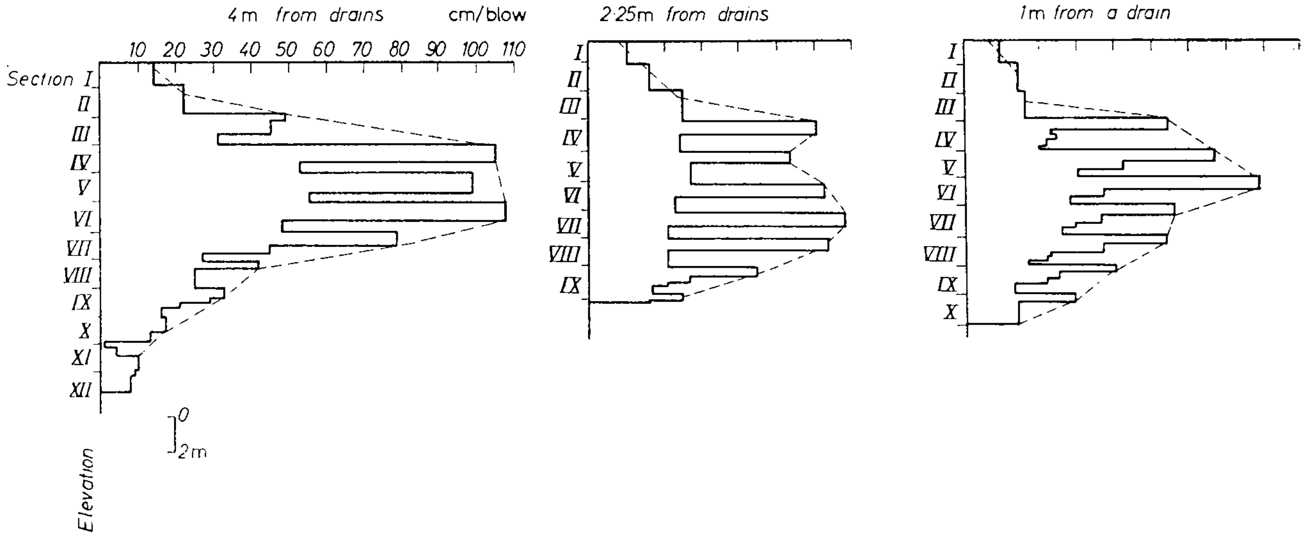


Fig. 6 Penetrometer diagrams after 45 days of drainage at various distances from the drains
Courbes des mesures au pénétromètre après 45 jours de drainage à differentes distances des pieux de sable

The drain and levee construction has been going on now for two years and so far no sliding has occurred in the zones consolidated with sand piles. On the contrary, in undrained zones where other works were going on, some important slides occurred.

The effectiveness of the drains was at once evident by the flow of water from the drainage discharge. In this connection the type of drainage adopted where the levees were already partially built is worthy of note.

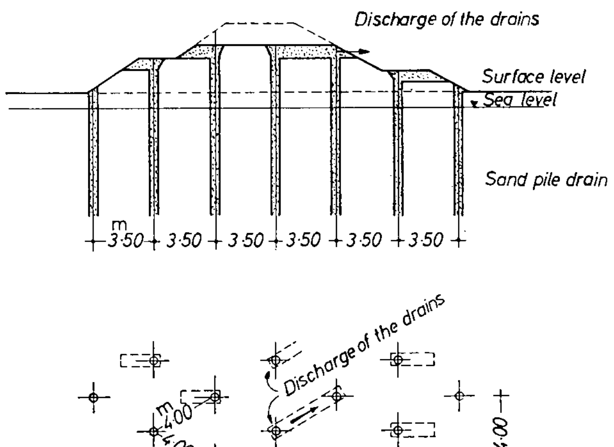


Fig. 7 Arrangement for the discharge of water when the drains have been built in partially constructed levee
Décharge superieure du drainage lorsque les pieux ont été executées sur la digue partiellement construite

The arrangement of the drains at these points was similar to that adopted where the levees were not yet begun with the exception that connections to the surface were changed. In the latter case they were effected by the usual sand bed, but in the former the sand pile heads were joined in twos or threes by means of small sand channels discharging at the levee sides which could be covered with the upper layers of the levee itself without disturbing its effectiveness.

In each case there is a central nucleus of clay which is crossed neither by the sand layers, in the case of unbuilt levees, nor by sand channels, in the case of levees already partially built (see Fig. 7).

This system, suitably modified in size, can be adopted for the foundations of major highways already constructed but needing accelerated consolidation.

Conclusions

The work described above proves once more the importance of continuous sampling and of the careful laboratory examination of these samples. Specimens bigger than the test samples to be used in the testing machines were taken in order to be able to remove the coating of remoulded clay covering their surface, after extracting them from their cylindrical containers. The removal of this coating could be effected with a simple thin, taut wire so that the stratifications could be seen. In this way it is possible to choose carefully the soil samples to be tested and to have a precise visual picture of the soil profile.

Another important finding was the excellent performance of sand piles in the presence of lenses of shells of limited size, because these lenses when full of water cannot discharge it. With sand piles all the permeable material could be exploited.

The working of the penetrometer in very soft soils is also worthy of note.

With these soils, after each blow, the casing must follow the point in order to prevent the soft clay, by closing around the point, altering the results of the test.

References

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Terzaghi, K. and Peck, R. B. (1948). *Soil Mechanics in Engineering Practice*, p. 280. New York; Wiley