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Buckling Tests of Slender Steel Piles in Soft, Quick Clay

Essai de Flambement de Pieux Minces en Acier dans de l'Argile Molle et Fluide

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Summary

Ninety-five piles of a small steel rail section were driven to rock through various strata, including about 25 m of very soft, highly sensitive and quick clay for the purpose of arresting the settlement of Lademoen Church in Trondheim, Norway, by the most economical means. Twenty-three of these piles were subjected to test loads, mostly of 110 or 120 ton, corresponding to unit stresses of 2555 kg/cm² or 2790 kg/cm². One of the piles failed in buckling at the latter load and was pulled for inspection. In addition, four 30 mm diameter piles and one pile made of tramway rail were tested to failure; the four round piles were pulled and found to have buckled.

A Failing Foundation

After nearly 50 years of service, Lademoen Church in Trondheim had settled so much that serious remedial measures became necessary. The church had been built on spread footings in a layer of coarse sand. Soil investigation undertaken during 1950-51 showed that there was only about 2 m of sand between the footings and a deposit of very soft and quick clay 23 to 29 m deep. Under the clay was a 7 to 10 m deposit of a coarse, very dense material, probably containing some big boulders. Beneath this layer was found what is believed to be solid rock.

Piles Tested and Method of Tests

It was planned to underpin the old foundations with slender steel piles, if possible driven to rock. Before a decision was made, four test piles of plain round steel bars 30 mm in diameter, and one pile made of tramway rail, were driven and loaded to failure. Finally, a rather slender ordinary rail section was adopted (the cross-section of this rail is shown in Fig. 1). Load tests were made on many of these piles after driving. One of them, called 'pile 21', failed. This pile and the four round steel bars, designated piles R1 to R4, were pulled after failure and carefully inspected. They had all evidently failed by buckling in the soft clay.

All piles were built up of bars of various lengths welded together. The load in all tests was applied by means of a hydraulic jack in increments of 1 ton for the round steel bars and 10 to 20 ton for the steel rail piles. The downward settlement of the top of the piles was observed by means of dial gauges and controlled by levelling. Settlement readings were taken after the load had been held approximately constant for 10 min. at each increment. In all cases the horizontal deflection of the top of the pile was prevented.

Properties of the Soil

The first soil investigations were made by the Geotechnical Laboratory of the Norwegian State Railways before the test piling was started. After the results of all tests were known

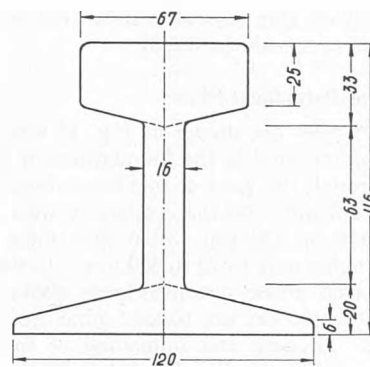
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En vue de mettre fin par des moyens économiques au tassement des fondations de l'Eglise de Lademoen à Trondheim, Norvège, 95 pieux constitués de petits rails d'acier ont été battus jusqu'au rocher, à travers des couches variées de terrain, comprenant environ 25 m d'argile très tendre, extrêmement sensible et fluide. 23 de ces pieux ont été soumis à des charges d'essai, la plupart atteignant 110 ou 120 tonnes, correspondant à des pressions unitaires de 2555 kg/cm² ou 2790 kg/cm².

L'un de ces pieux s'est déformé par flambement, sous l'action de cette dernière charge. De plus, 4 pieux de 30 mm, et un cinquième constitué par un rail de tramway, ont été essayés jusqu'à déformation; les 4 barres rondes d'acier ont été retirées et on constata qu'elles avaient flambé.

the Norwegian Geotechnical Institute made another investigation, particularly of the soil within 1 to 4 m of the locations where the buckling failures had occurred. Although the methods and equipment employed in the two series of tests differ considerably, the results agree closely.

The following may be worth noting from the second series



All dimensions in millimetres

Fig. 1 Rail section used for piles
Profil du rail utilisé comme pieu

of tests (Fig. 2). Starting from the bottom of the sand deposit (3 to 5 m below the surface) it was found:

(a) The upper 7 to 8 m of the clay deposit have a very high water-content, varying from 50 per cent at the top to 30 to 35 per cent at the bottom of this layer. The liquid limit varies correspondingly from about 35 per cent to about 22 per cent. The shear strength of undisturbed specimens is generally between 1 and 2 ton/m², but that of the remoulded material is exceedingly low, giving a sensitivity in most cases higher than 100, and figures up to 240 were obtained.

(b) In the next 15 m of depth the water content is at, or slightly above, 30 per cent and the liquid limit varies between 22 per cent and 25 per cent. The undisturbed strength remains

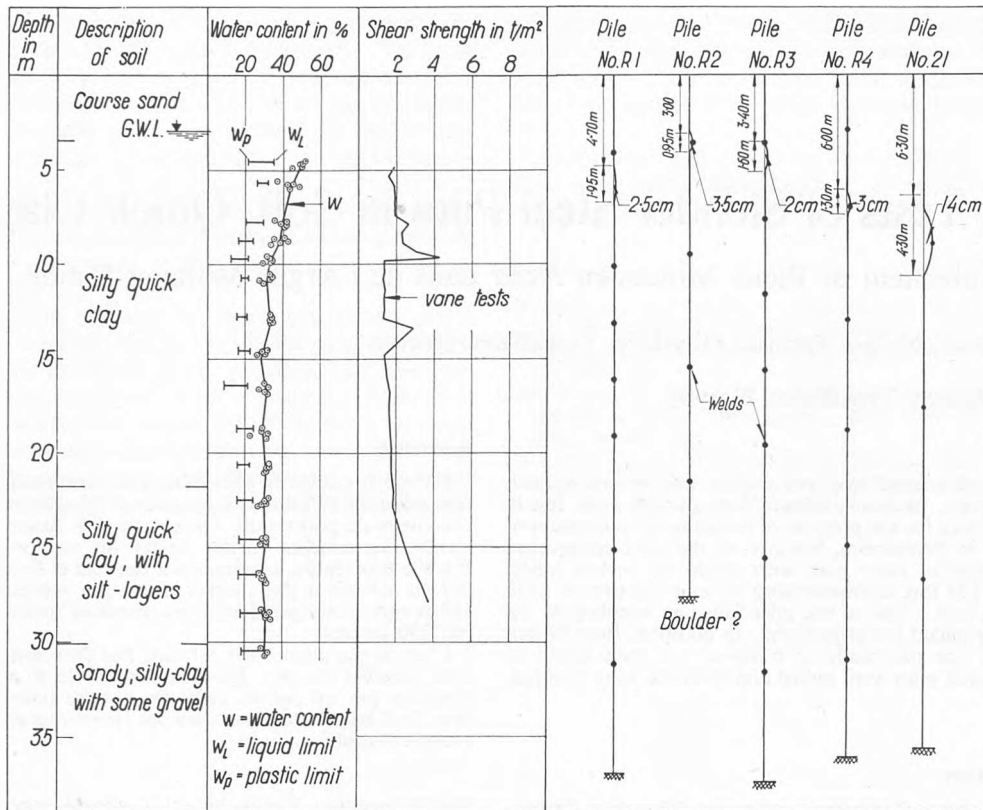


Fig. 2 Soil profiles and details of buckled piles
Coupe du sol et détail des pieux après flambement

as in the preceding layer, or a little lower, but the sensitivity is considerably lower, although still very high (mostly above 40).

(c) The rest of the clay deposit is somewhat less quick, and the strength increases with the depth.

Loading Tests on Permanent Piles

Altogether 95 piles (as shown in Fig. 1) were driven and permanently incorporated in the foundations of the structure. The driving through the hard coarse layer directly above the rock was very difficult. Of these piles, 19 were subjected to test loads of 110 or 120 ton, while four piles, for various reasons, were loaded only to 70 to 100 ton. In five cases some settlement occurred under sustained loads above 80 to 90 ton indicating that the pile had not bedded quite firmly in the rock during driving. Loading and unloading of these piles was continued until practically no further permanent settlement occurred under the maximum load. Under the design load of 70 ton no pile showed permanent settlement.

A typical load-settlement curve observed for one of the piles tested is shown in Fig. 3. The permanent settlement in this case was small but, due to defects in the loading arrangement, the full test load was not reached the first time and a second loading was made. It is seen that at medium loads, the settlements are larger the second time, and the difference is not caused by permanent set of the pile point. Three straight lines (S_1 , S_2 and S_3) have been drawn indicating what should have been the purely elastic settlement of the pile top if the full load were carried by the pile to a point, (1) on top of the hard bottom layer, (2) on top of the rock, or (3) half way between (1) and (2), and were entirely absorbed by the ground at the point mentioned. None of these assumptions is fully plausible but the third one seems the more probable approximation as regards the main part of the load. However, it is clearly seen

(a) that some part of the load was transmitted to the soft soil above, and (b) that this part was considerably smaller at the second loading than at the first, showing that the adhesion between soil and pile had been loosened by the loading of the pile. At 100 ton load, the difference between the actual settlement and that corresponding to the line S_3 is about 7 mm at the first and 3 mm at the second loading, which roughly would

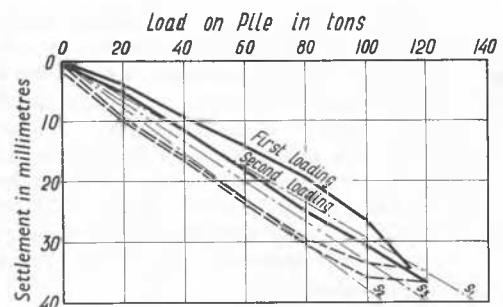


Fig. 3 Load-settlement curves for pile 27
Diagramme des tassements en fonction des charges du pieu 27

correspond to an adhesion of 2.6 and 1.1 ton/m² respectively if the adhesion is assumed to be constant from the top of the hard layer upwards. Similar differences were observed in all tests, although the figures vary slightly. The fact that some adhesion was active has a bearing on the evaluation of the buckling tests.

Buckling of 30 mm Round Steel Bars

The main data from these tests are given in Fig. 2. The piles were placed in one row 50 cm apart. The depth of penetration

was practically the same for three of the piles, but R2 stopped abruptly about 10 m short of the others. The location and extent of the buckling is indicated, as well as the location of welds. It is seen that all four piles failed within the upper 4 m of the quick clay deposit, immediately underneath the top layer of sand. Except for the buckling of the piles after pulling were, as far as could be seen, straight. In two cases the welds were entirely outside the region of buckling, in one case a weld was inside, but somewhat off the middle of this region. Buckling seems to have started at the weld in the case of the pile carrying the extraordinary high load of 16 ton, against 9 and 11 ton for the others.

Fig. 4 shows the load-settlement curves obtained for pile R1, which failed at 9 ton with a permanent top settlement of 10 mm. On reloading only a load of 2 ton was reached. Lines S_1 , S_2 and S_3 , as explained previously, are shown. For loads between 2 and 6 ton, the curve of the actual settlements runs

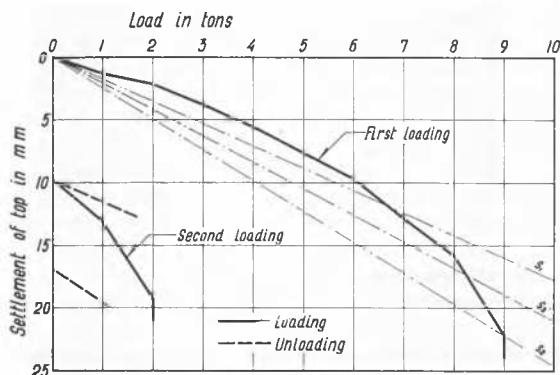


Fig. 4 Load-settlement curves for pile R1
Diagramme des tassements en fonction des charges du pieu R1

nearly parallel to the line S_3 . The difference at 6 ton load is about 2.7 mm, corresponding to a hypothetically uniform adhesion above the hard ground of about 0.9 ton/m².

Buckling of Pile 21

This pile was intended to be part of the permanent structure. The loading test indicated some settlement of the pile point, as explained earlier. Therefore, the pile was loaded four times in the course of 12 days. Fig. 5 shows the load-settlement curves for all loadings, with the time noted for some important observations. On the fourth loading, the load reached 120 ton, intended to be the maximum. In 5 minutes it fell off to 115 ton,* was brought up again to 120 ton and then seemed to stay constant; the test was considered to be complete. However, after 2½ hours, when the loading apparatus was to be dismantled, the load had fallen to 40 ton and could not be increased. The pile was pulled and inspected.

Fig. 2 shows the deformation. The pile had buckled in the direction of least moment of inertia between about 2.5 m and 7 m below the top of the soft clay deposit. Fig. 6 is a photograph of the buckled part of the pile as it appeared above ground. As far as could be ascertained, without special measurements, the pile outside the buckled part was perfectly straight. There was a welded joint in the bend a little below the point of maximum deflection and a crack was found on the tension side. The photograph shows that the rail has been stressed in bending beyond its yield point close to the weld and as such stresses could not have been transmitted through a cracked weld, the crack was clearly the result, and not the cause, of the buckling.

* Probably caused by the loading apparatus.

Fig. 5 again shows the lines S_1 , S_2 and S_3 (described previously). Calculated as before, the average adhesion between pile and clay at 80 ton load would be about 2.0, 1.3, 1.0 and 0.9 ton/m² at the first, second, third and fourth loading respectively.

Test of Tramway Rail Pile

As mentioned before, a tramway rail pile was included among the piles initially tested. The load-settlement curve for this pile, designated 'pile A', is shown in Fig. 7. An attempt to pull this pile after the test was not successful; therefore, the type

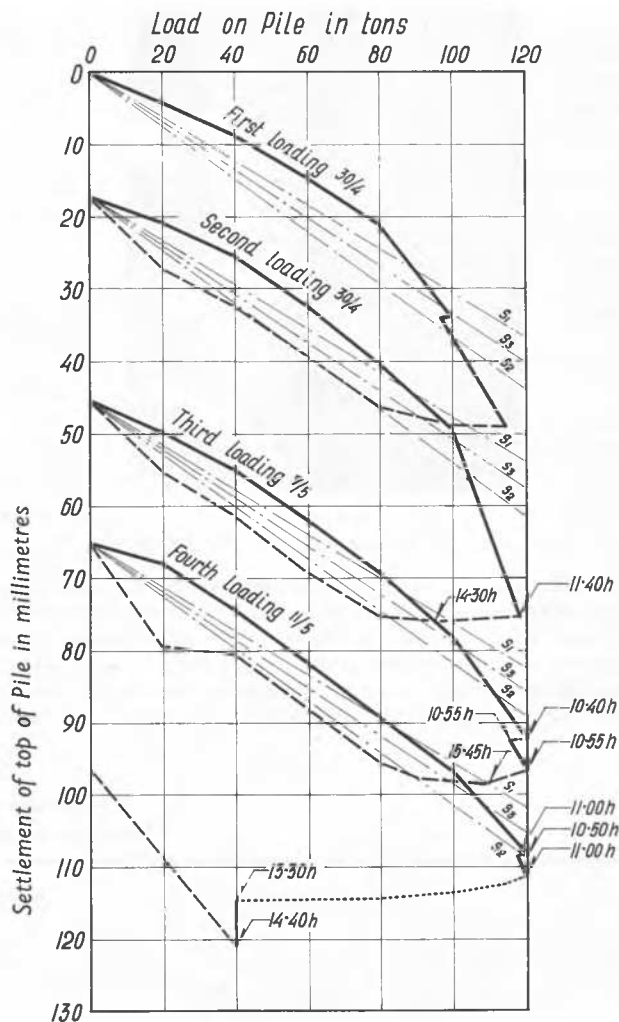


Fig. 5 Load-settlement curves for pile 21
Diagramme des tassements en fonction des charges du pieu 21

of failure is not definitely known. However, Fig. 7 indicates that the pile had buckled and the data for this pile have been included in the summary of test results (see table).

Test Results

The main test data are summarized in the table. In columns 7 and 8 are shown the values of the soil resistance factor, $k = p/y$ (where p is the change in lateral pressure between the pile and the ground due to a horizontal displacement y), and the length of the half-sine waves at buckling failure, $L_{crit.}$, both calculated on the basis of Granholm's solution for initially straight piles:

$$P_{ult.} = 2\sqrt{kEI}, \text{ or } k = \frac{P_{ult.}^2}{4EI} \quad \dots (1)$$

and

$$L_{crit.} = \pi \sqrt{\frac{k}{EI}} = \pi \sqrt{\frac{P_{ult.}}{2k}} \quad \dots (2)$$

Tentative Interpretations of Results

An important question is whether the ultimate loads shown in the table are really applicable to the piles tested and the properties of the soil, or whether accidental faulty alignment

incurred for purely scientific purposes. The piles appeared to be straight and every effort was made to make them so, that is all which can be stated.

A conspicuous feature of the test data is the high load of 16 ton carried by the round bar pile R2, as compared to an average of 9.7 ton for the other three piles of this type. The four piles were placed so near to each other that it is difficult to assume that a variation in soil properties caused this difference in ultimate load. A plausible explanation would be that

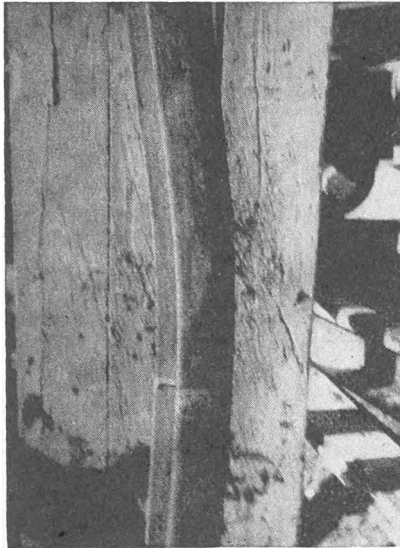


Fig. 6 Buckled part of pile 21
Partie du pieu 21 après flambement

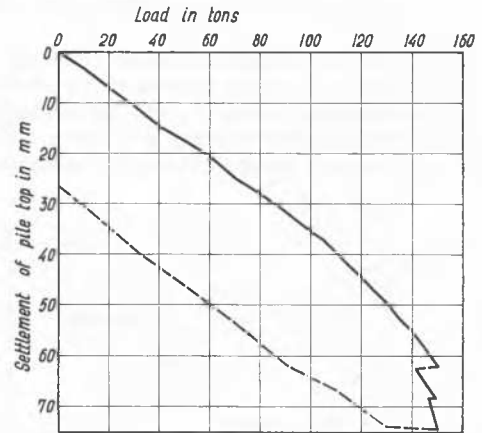


Fig. 7 Load-settlement curve for test pile A
Diagramme des tassements en fonction des charges du pieu A

of the piles may not have been the principal cause of failure in all or several cases. There is no basis of exact observations on which to answer this question. It should be noted that these tests were undertaken solely for the practical purposes of determining the best way of preventing foundation failure within the very limited means available, and no extra costs were

R2 was initially practically straight, while the three others had appreciable accidental initial eccentricities. Assuming this to be so, and assuming further that piles 21 and A were also approximately straight, the significant results would be those obtained for piles R2, 21 and A, for which the k -values of 7.50, 7.98 and 8.52 are in quite close agreement. Moreover, these correspond very well with the value of $k = 8.0 \text{ kg/cm}^2$ which the Norwegian Geotechnical Institute has calculated from the results of compression tests on undisturbed samples of the soil.

Table
Summary of test results
Résumé des résultats des essais

Pile	Pile type A = cross-section I = moment of inertia d = width of pile	Pile length		Results of loading tests			Calculated from Granholt's formulae		Notes
		Total m	In clay m	$P_{ult.}$ ton	$\sigma_{ult.}$ kg/cm ²	Buckled length (Fig. 2) m	k from equation 1	$L_{crit.}$ from equation 2	
	1	2	3	4	5	6	7	8	
R1	30 mm round steel	37	24	9.0	1270	1.95	2.48	1.34	Inspected
R2	$A = 7.07 \text{ cm}^4$	27.4	24	16.0	2260	0.95	7.50	1.05	Inspected
R3	$I = 3.97 \text{ cm}^4$	37	24	11.0	1550	1.60	3.54	1.24	Inspected
R4	$d = 3 \text{ cm}$	36	24	9.0	1270	1.30	2.48	1.34	Inspected
21	Rail $A = 43 \text{ cm}^2$ $I = 210 \text{ cm}^4$ minimum $d = 11.6 \text{ cm}$	32	25	120	2790	4.30	7.98	2.73	Inspected
A	Tramway rail $A = 44 \text{ cm}^2$ $I = 350 \text{ cm}^4$ mini- mum $d = 17.3 \text{ cm}$	36	22	160	3630*	?	8.52	3.04	Not inspected after failure

* $\sigma_{0.2}$ for this steel was 4200 kg/cm²

On the other hand there is some difficulty about accepting this assumption of the straightness of pile R2 and the accidental initial eccentricity of piles R1, R3 and R4. All piles failed in the top zone of the clay, where the soil was least resistant and the residual load in the pile greatest, that is, where they would be expected to fail if the soil and the load were the determining factors.

If accidental eccentricity of the piles were the decisive factor, one would have expected buckling to have occurred in some cases at a lower level in the quick clay. The differences in soil strength and effective pile load between the top zone and points some 5 or 10 m lower down should have little effect compared with accidental eccentricities of a magnitude sufficient to account for the reduction in ultimate load from 16 ton to 9 ton.

Further, an examination of the figures in columns 6 and 8 of the table gives some indication that the type of failure of piles R1, R3 and R4 has not been essentially different from that of pile R2. There is, of course, no reason to expect the length of the buckled part of a pile to be equal to, or even proportional to, the theoretical length of the half-sine wave at buckling, but one would expect some similarity in order of magnitude, and this is found. The high ultimate test load of pile R2 corresponds to a length of buckled pile much shorter than the corresponding length for the other piles.

Consequently it seems that one cannot exclude the possibility of the high ultimate load on pile R2 having been caused by some accidental feature of the test not observed at the time. On this assumption, it would be reasonable to exclude pile R2 from comparison with pile 21, which represents a minimum figure, being the only one of 9 piles which failed at the maximum load of 120 ton. The average value of k for piles R1, R3 and R4 is 2.83, which—compared with 7.98 or 8.52 for piles 21 and A respectively—is so low that no current theoretical solutions can explain the difference.

Formulae for calculating k from the soil properties have, as

far as is known to the authors, all been derived with modifications from Boussinesq's equations for the elastic half-space. The effect of the width of the pile on the value of k in all these formulae is very slight.

Depending on which of the two tentative interpretations given is applied to the results, the tests here reported either confirm or strongly contradict the conclusion that the width of the pile has little effect on the value of k . This important point, therefore, unfortunately remains unanswered by the tests. Should the second conclusion be confirmed, the reason why the effect of the width of the pile is not sufficiently expressed in the theoretical solutions may be found in the assumption of an elastic and homogeneous material. Solutions based on this assumption, while most useful in analysing stress distribution and settlement in large masses of material, may not be suitable for an analysis of local distortion and failure close to the pile.

Conclusions

(a) It has been demonstrated that slender steel piles in a soft clay have actually failed in buckling.

(b) While the piles were very slender, in some cases excessively so, and the soil was exceptionally soft, a large number of piles have been shown to carry actual test loads corresponding to unit stresses considerably higher than what would be applied in ordinary foundation design.

(c) Even in this weak soil, transmission of load to the soil by adhesion along the pile is not negligible.

(d) The test results do not permit definite conclusions whether the width of the pile is a factor of primary importance in determining the ultimate load on such piles.

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