Buckling of Piles with Initial Curvature

Le Flambage des Pieux avec Imperfections Initiales

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SYNOPSIS The paper deals with the problem of the buckling of foundation piles, with initial curvature, under axial load. A solution was obtained for a pile driven through soft soil in which the coefficient of horizontal soil reaction increases with depth, to bearing strata. The critical buckling load was computed for the case of a steel pipe pile for elastic-plastic state of stress by the finite element method. The numerical examples are compared with the results obtained by the test loading of a 40 m long pipe pile with initial curvature and maximal lateral deflection over 50 cm. The investigation was carried out for remedial measures for a piled marine structure, which had been seriously damaged during dredging operations in front of the wharf.

INTRODUCTION

In the comprehensive publication of Professor de Beer in which, among others, all the contributions to the Speciality Session No. 10-IX. ICSMFE, Tokyo 1977, "The Piles Subjected to Static and Dynamic Lateral Loads" are reviewed, i.e. piles subjected to horizontal loading along their shaft, being called "passive piles". Such horizontal loads are caused by horizontal movements of the soil occurring after installation of the piles. Of course such movement of the soil should possibly be prevented but if this cannot be eliminated then damage may occur leading to permanent deformation or even to the failure of the piles. In the cases where soil movement is stopped the problem is encountered of how to calculate the buckling load and the bearing capacity of such deflected piles, which shall be called piles with initial curvature.

In this contribution the numerical analysis of a single deflected pile will first be described and the results of the loading test on a steel pipe pile with a previously measured curvature will be given. Finally a practical case record will be presented of the remedial measures of a piled marine structure where owing to the sliding of an underwater slope the structure and several piles were seriously damaged.

CRITICAL BUCKLING LOAD OF A DEFLECTED PILE

The deflection of the pile should be large enough so that the elastic-plastic analysis and the influence of the axial forces on the bending deformability of the pile have to be taken into account. The numerical analysis of the critical buckling load is established by the "NONFRAN" computer programme, with the following assumptions:

- the cross-sectional area of the pile is compared to its length, so small that the pile can be considered as a beam,
- the bending combined with the axial load are calculated according to the theory of large deformations by the finite element method,
- the elastic-plastic state of stress is considered without elastic unloading of fibres (nonlinear elastic material),
- the soil is characterised by a modulus of sugrade reaction variable with depth,
- the friction in the longitudinal direction between the pile and the soil is neglected.

The calculation is done in two steps: first the stresses within the pile owing to possible or actual deflections of the pile are calculated. Out of the deformation the horizontal load is obtained which would deflect the pile from an undeformed state to a deformed position. The calculation is simplified since an initially underformed state is considered and the time factor of the deflected pile is not taken into account. Thus the influence of retarded (initial) deformations and stresses is neglected. In the second phase the additional axial load ΔP on the pile is calculated which finally causes the buckling of the pile. With the increase of this load the deflected pile becomes further deformed. The bending deformations are resisted by the surrounding soil which is characterised by the variable, linearly increasing modulus of the horizontal reaction. The computer programme for the discussed numerical analysis was made by Banovec (1977). In Fig. 1 an example of a calculation for the bending moments in the pile and for the reactive pressure of the soil in function of the axial load and from the measured initial curvature of the pile. The pile discussed is a steel pipe pile with a diameter of 508 mm and a wall thickness of 8 mm, installed at a construction site (Fig. 5, pile F1). It was driven through weak clayey soil into the bearing gravel layer. The pile head is fixed into a concrete structure and at a length od 29.40 m it is assumed to be completely embedded. At the vertical load P = = 700 kN the pile deflected due to horizontal soil movements as shown in Fig. 1a. The critical axial load at the head of the deflected pile is according to the previously described calculation $P_{cr} = P + \Delta P =$

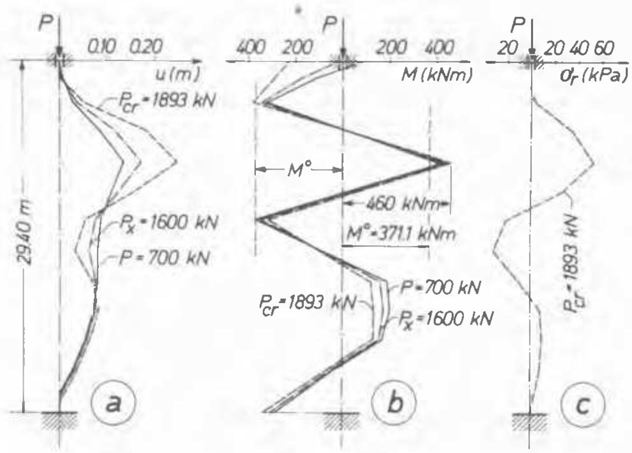

Fig. 1 Numerical analysis of a deflected pile a) deflections, b) bending moments, c) horizontal soil reaction

= 700 kN + 1193 kN = 1893 kN. In Fig. 1a horizontal deflections of the pile at the additional load $P_x = P + \Delta P'$ = 700 kN + 900 kN = 1600 kN and at the critical load P_{cr} = 1893 kN are shown. Fig. 1b shows the bending moments in the pile and Fig. 1c shows the horizontal reactive soil pressure. M_o indicates the bending moments at the plastic limit which in the given example had been partly exceeded at the initial deformation of the pile. This means that owing to plastification of steel the problem of exceeded allowable stresses is not decisive but the critical load is decisive at the moment of the failure of the pile.

RESULTS OF THE TEST LOADING OF A STEEL PIPE PILE WITH INITIAL CURVATURE

The test loading was carried out on a steel pipe pile with a diameter of 508 mm and wall thickness of 8 mm (see Fig. 5, pile C_1). The pile was 40 m long; from the head to a length of 6 m it was in water then it was driven through a 23 m thick layer of normally consolidated clayey sediments in plastic to liquid consistency state into a 11 m thick layer of dense sandy gravel; the pile tip was embedded in firm eozene flysh. In Fig. 2 the geometry of the pile curvature is shown before the test was started; the largest horizontal deflection in the x-direction was 50.5 cm. In the same figure load settlement diagram is given. The horizontal deflection of the pile measured at a depth of 10 m below the pile head increased linearly with the testing load and reached a value od 0.7 mm at the maximal axial force 1650 kN. After the pile was unloaded it nearly completely recovered its initial deflected state. According to the calculation given in the previous chapter the critical buckling load for a pile with the given initial deflection is 1750 kN. That is only a little more than the load reached during the loading test (1650 kN). For a certain reason the loading to the critical point was not carried out. However the conclusion from the obtained results is that the calculated critical buckling load according to the computer programme from chapter 2 is realistic.

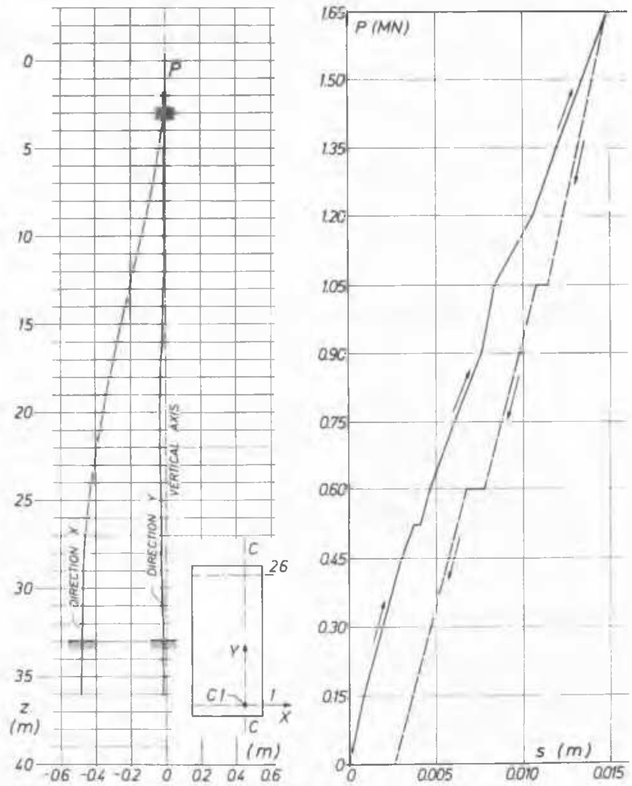

Fig. 2 Load settlement diagram (right) of an initial deflected pile (left)

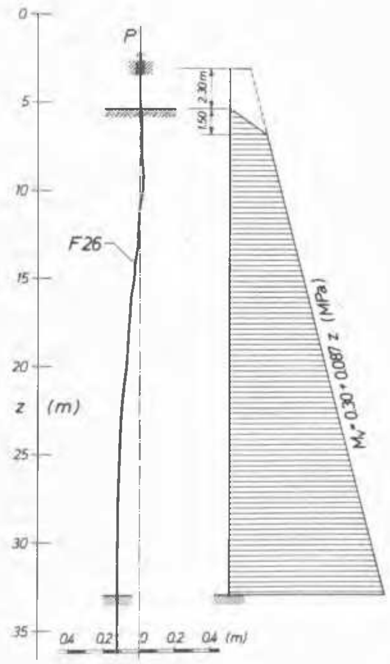

Fig. 3 The influence of the lateral strength of the soil on the critical buckling load for an initial deflected pile

THE INFLUENCE OF THE LATERAL STRENGTH OF THE SOIL ON THE CRITICAL LOAD OF THE DEFLECTED PILE

The influence of the lateral strength of the soil on the critical buckling load of the deflected pile was investigated on a pile with an initial imperfection as shown in Fig. 3 (see Fig. 5, pile F26). If the lateral strength of the soil is not taken into account, then the numerical critical load is 1146 kN. The critical load of 2158 kN relates to the strength of the surrounding soil characterised by the horizontal deformation modulus as shown in the same Fig. But if only 10 % activisation of this strength is considered the critical load is 1960 kN. The conclusion from this is that the calculated bending moments are not particularly sensitive to the values of the modulus of subgrade reaction but they are sensitive to the displacements of the surrounded soil. The same was quoted for vertical piles by Marche (1977).

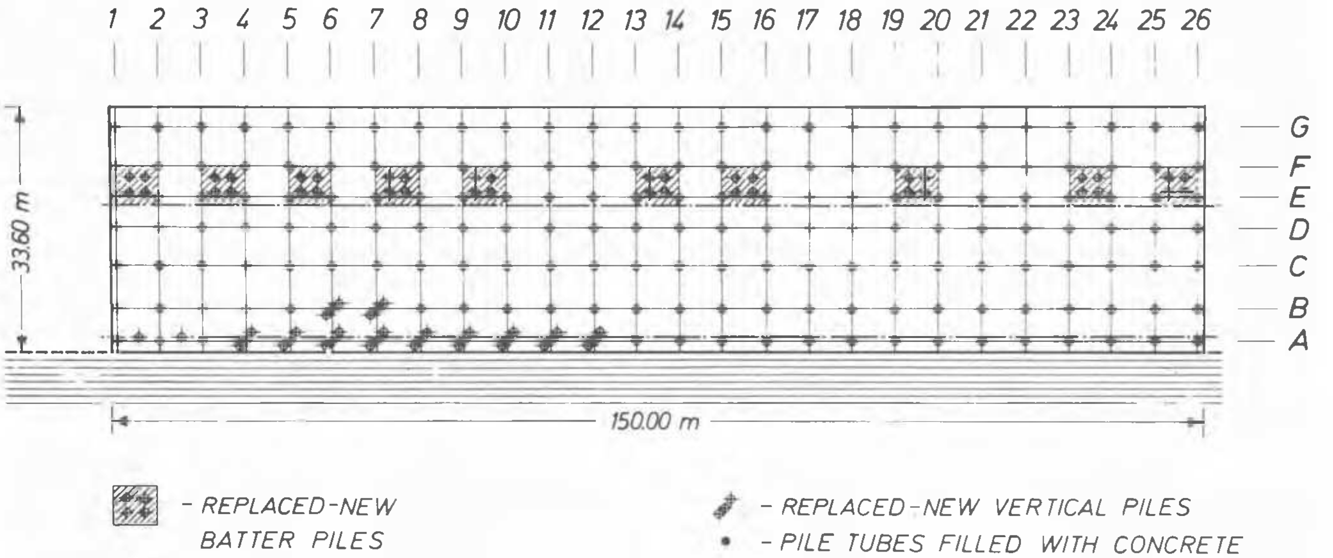

Fig. 4 The plan of the piled marine structure which had been damaged during dredging operation

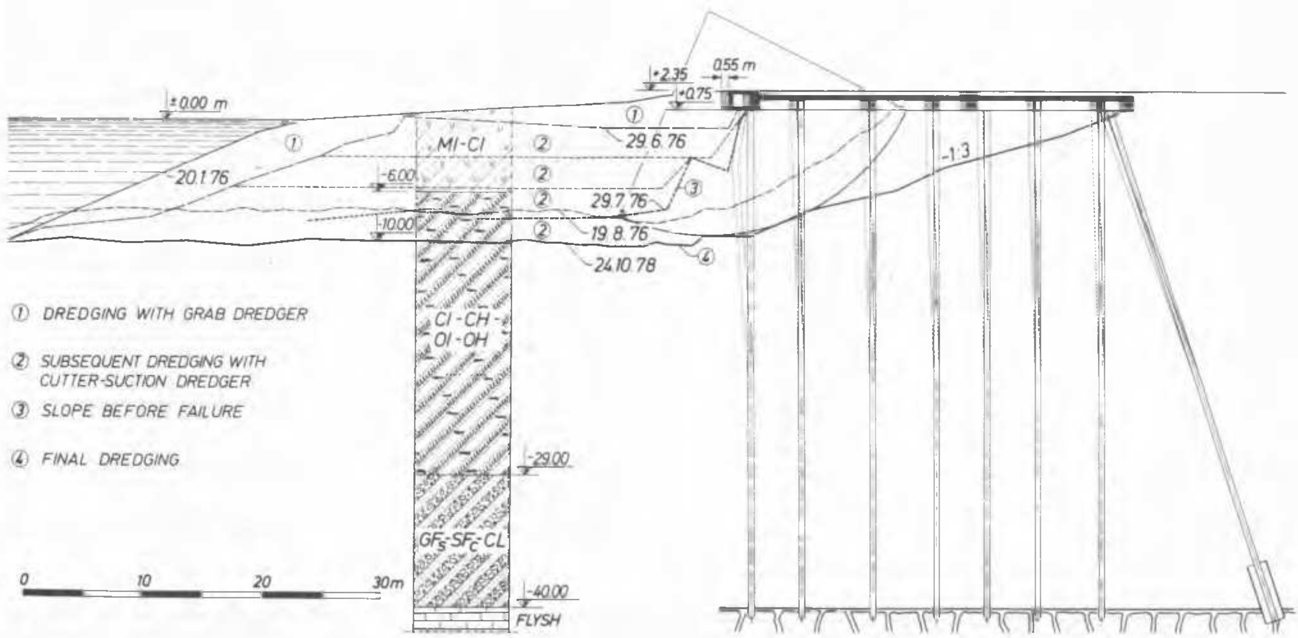

Fig. 5 The cross-section of the structure with the slip surface of the steeply excavated slope, the displaced platform and failed resp. deflected piles

AN EXAMPLE OF DAMAGED PILES OF A MARINE STRUCTURE

Some years ago a container cargo berth was built in an Adriatic harbour. The structure is a reinforced concrete platform founded on vertical and batter steel pipe piles. The plan of this structure and its characteristical cross-section is given in Figs. 4 and 5. The construction was carried out so that the piles were driven from a platform, made of hydraulic fill and covered by a thin gravel layer. After the installation of the piles a concrete structure was laid on to the surface of the platform. This was followed by subsequent harbour dredging of the subsoil which was supposed to be carried out in intervals but in such a way that at no stage the inclination of the underwater slope was steeper than 1:3. Single stages of this dredging operation can be seen in Fig. 5. Unfortunately, however, most stages of the excavation and the last one in particular were not carried out according to the project. The shore was excavated partially at an inclination of 1:1 or even steeper. While this dredging was being carred out sliding of the steeply excavated soil underneath the structure occurred and due to this some piles completely failed while others, however, deflected strongly. Because of the sliding of the ground and damaging of the piles the whole piled structure displaced translatorally and rotationally.

As shown in Fig. 5 the subsoil at the location of the shore is composed of clayey-silty layers to a depth of -6 m. This is followed by normally consolidated soft clayey marine sediments to the elevation of -29 m and after that by sandy and silty gravel. At the elevation of -40 m stiff eozene flysh appears composed of marls and sandstone. Geotechnical properties were investigated "in situ" and in the laboratory. The results for the top silty layer and for the soft marine clay are shown in Fig. 6.

Due to the subsoil movements the piles 4 to 12 in the first row and the piles 6 and 7 in the second row failed. The failure of the piles as shown in Fig. 5 occurred at a depth of 4.60 m below the transversal slabs. The other piles deflected more or less in the direction of the movement of the soil (direction x) as shown for rows 1 and 26 in Fig. 7. Deformation of the piles in the transversal direction (y) was considerably smaller and was to a great extent to be ascribed to the movement of the rigid concrete platform. The deflections were measured by a special inclinometer.

The steeply excavated slope moved on the slip surface as shown in Fig. 5. The cohesion of the soil as shown in Fig. 6 was not sufficient for the overall stability. The sliding soil was held back by the piles themselves to the width of 1.5 diameter of the piles only, so that a more than 2 m high step was formed behind them. The horizontal earth forces which were transfered to the heads of the piles moved the whole concrete platform in the direction of the excavation thus damaging not only the vertical but also the batter

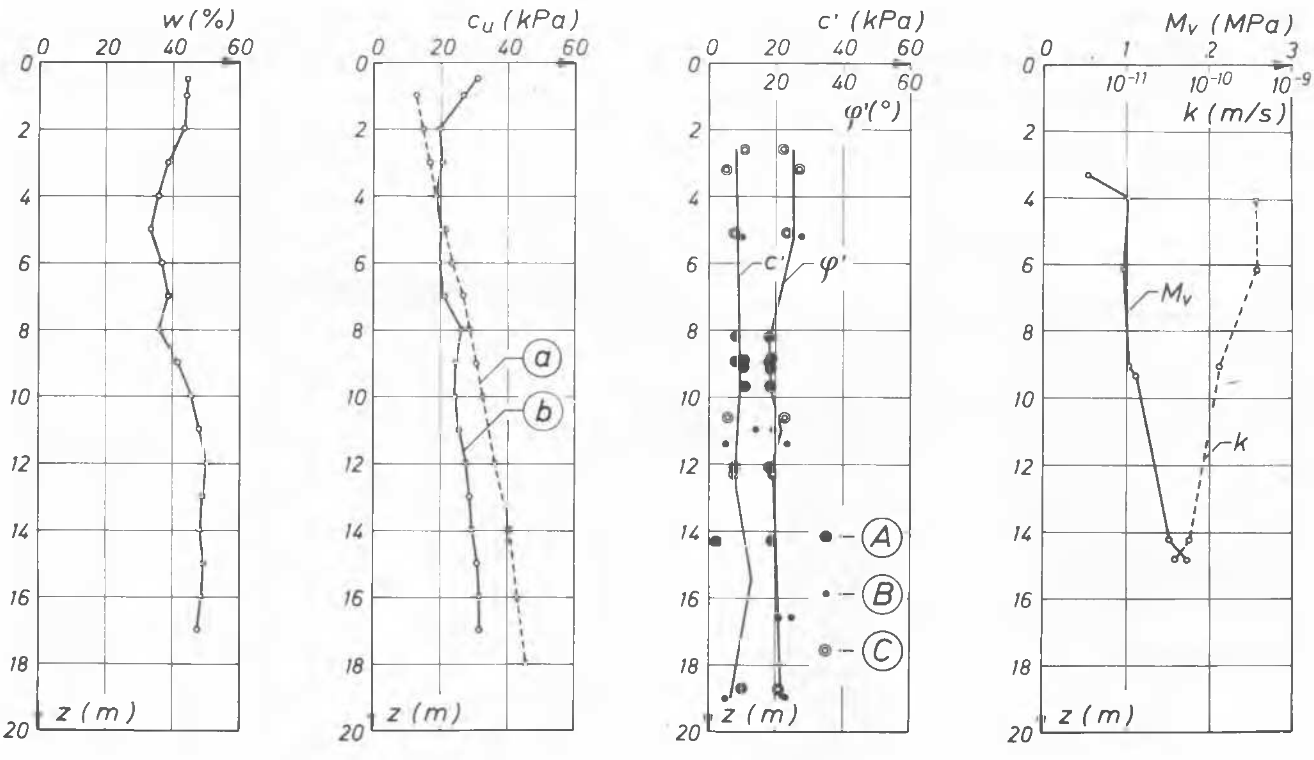

Fig. 6 Geotechnical properties of the subsoil: w = natural water content, c_u = average cohesion (vane test results), a) before the slide, b) after the slide and final dredging, c', φ' = shear strength, A) triaxial tests on 100 mm dia. samples, B) triaxial tests on 37 mm dia. samples, c) direct shear tests, M_v = deformation modulus, k = coefficient of permeability

piles. The steep excavation was carried out only under approximately half of the length of the structure (rows of piles from 1 to 13) whereas the slope under the other half (rows of piles from 14 to 26) was excavated in the inclination of approximately 1:2. At this part of the structure there was no sliding of the slope and the deformation of the piles was minimal at least at the first stage of the platform movement. Therefore the structure displaced first rotationally and only later did it also displace in a translateral direction. After the first deflection of the piles was registered the dredging of the steep slopes underneath the structure was immediately started as well as the excavation of the back filling on the shore side and all this had a stabilizing effect on the movement of the shore and the displacement of the platform. Remedial measures were carried out in the following way: the structure was provisionally anchored to the shore and underneath the platform on the water side between the piles steel flats (floating caissons) were placed to carry the vertical load of the structure instead of the failed piles. This was followed by cutting holes in the concrete structure and driving new vertical and batter piles (Fig. 4) and finally dredging the slope underneath all the structure to an inclination of 1:3 and backfilling the shore side (embankment of light-weight material). The deflected piles and the changed statical system were calculated by the method described in the second chapter . With the piles later filled with concrete a higher load bearing capacity was considered. Further for control of the calculation, axial and circumferential stresses in steel pipes were measured 25 cm below the heads of the deflected piles C_1 and F_1. On the pile C_1 after a hole was made in the concrete platform, a loading test was carried out the result of which has been given in the third chapter. During remedial measures the displacements of the structure were observed. Fig. 8 shows the horizontal displacements of the measuring marks 1 to 3 and Fig. 9 shows the settlements of the piled structure.

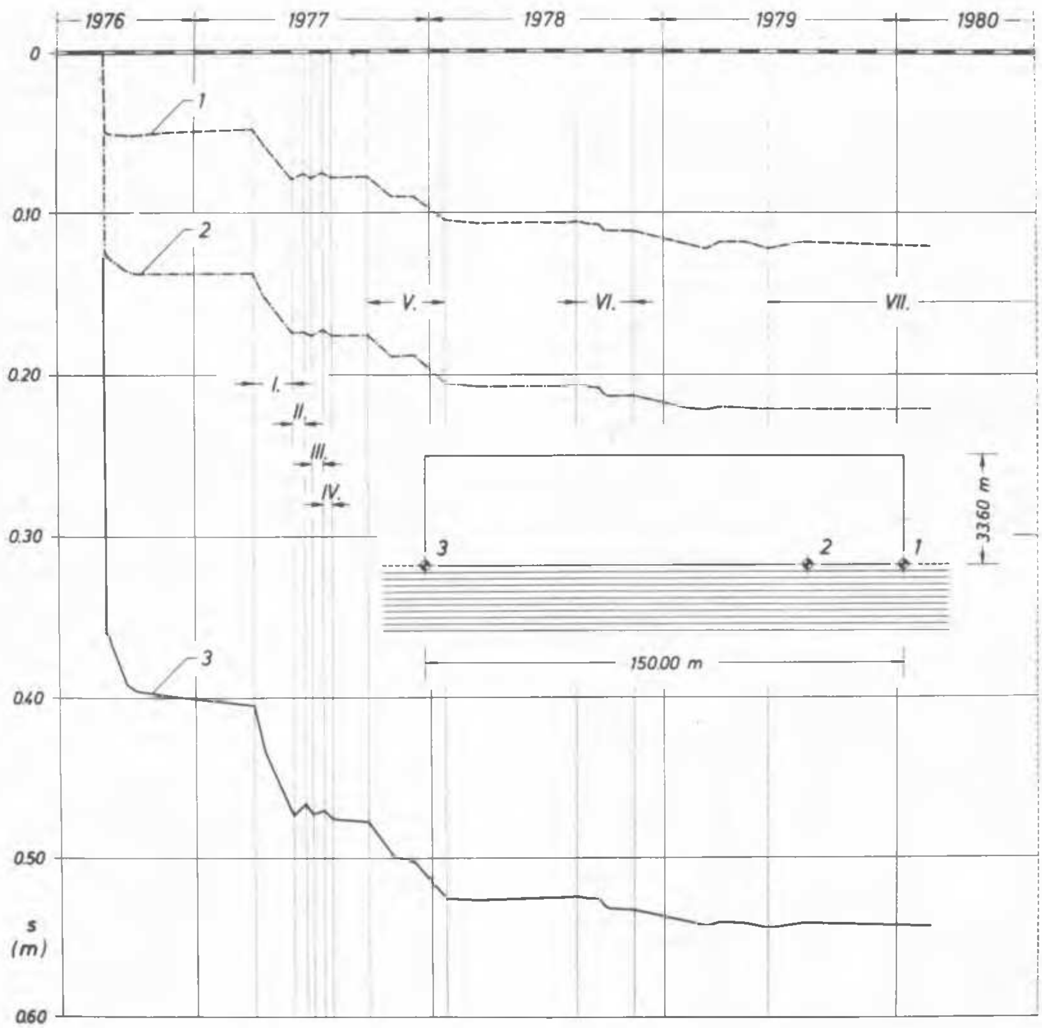

Fig. 8 Horizontal displacements of the structure and particular phases of remedial measures: I) dredging underneath the platform, II) placing of steel flats (floating caissons), III) driving of the replaced vertical piles, IV) removal of steel flats, V) driving of new batter piles, VI) final dredging underneath the platform, VII) finish of remedial measures and the start with the exploitation of the wharf

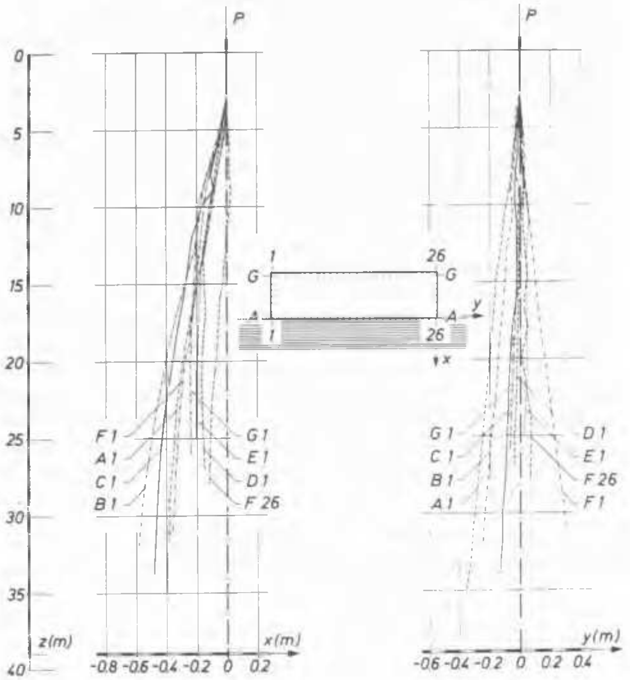

Fig. 7 Deflections of the piles

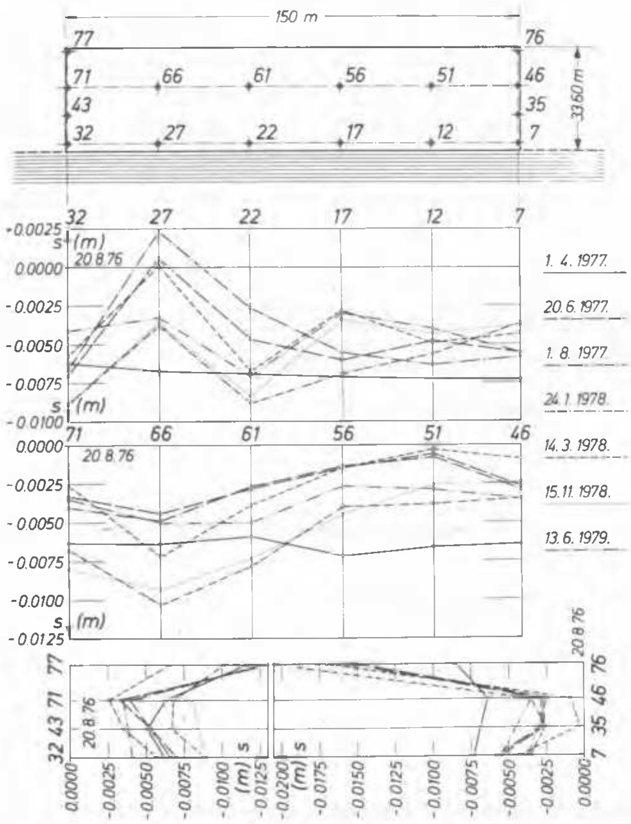

Fig. 9 Settlements of the structure during remedial measures

CONCLUSIONS

Calculation method for critical buckling load of a steel pipe pile with initial curvature is developed for elastic-plastic state of stress by the finite element method.. It is shown that also in the case of deflected piles the calculated bending moments are not particularly sensitive to the values of the modulus of subgrade reaction but they are sensitive to the movements of the surrounding soil. Theoretical calculations are checked against a full-scale test of a deflected pipe pile and by succesful remedial measures for a piled marine structure which had been damaged during harbour dredging.

ACKNOWLEDGEMENT

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