Ground Improvement of the Grain Silo of Hodeidah

Amélioration du Sous-Sol du Silo à Grain de Hodeidah

J.K. HARTIKAINEN Professor of Soil Mechanics and Foundation Engineering, University of Oulu, Finland

SYNOPSIS The first silo of the Yemen General Grain Corporation, with a capacity of 20.000 tons, was built on the shore of the Red Sea in 1978-80. The biggest problem encountered in laying the foundation was the topmost silty sand layer, which was about 10 metres thich and exceptionally loose. What the silo needed was an earthquake-proof base.

The foundation of the silo was laid on a slab measuring 24 x 55 square metres and resting on ground improved by stone columns. The settlement of the silo was calculated at all the most important stages of the work, and the interaction of ground and structure was calculated at the critical stages of construction.

The present paper clarifies the differences between the estimated and the measured settlements and deflections. The progress of the settlement at the beginning of operations corresponded to preliminary estimates, but towards the completion of the project and at the first stage of filling, the settlement was smaller than estimated. The deflections are larger than estimated, but smaller than allowed.

SITUATION AND GROUND CONDITIONS OF THE SILO

The constructed silo (Fig.1) is situated within the port complex of Hodeidah in the Yemen Arab Republic (Fig.2). The geology of Yemen represents a zone of subsidence, which occurred in the northern and central areas of the peninsula. These areas of subsidence were later filled with sedimentary deposits. These cretaceous beds in the area of the project are generally in the order of 100-200 m thichness overlying the Jurassic beds (Geukens, 1966).

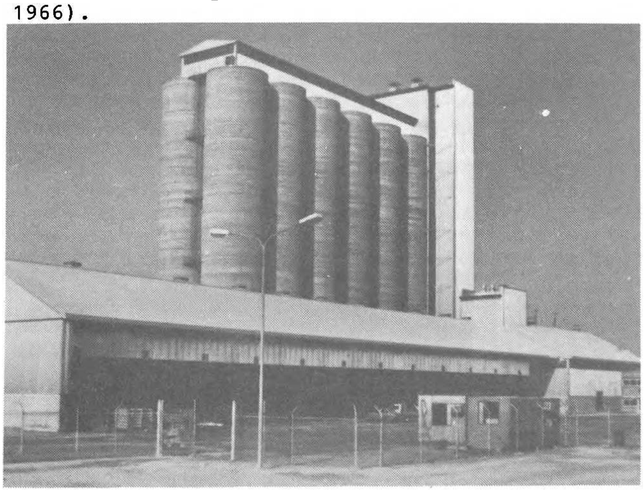

Fig.1 The completed silo in 1980. Bagging plant in front, elevator housing in the rear and conveyor above the silo. The office at the right.

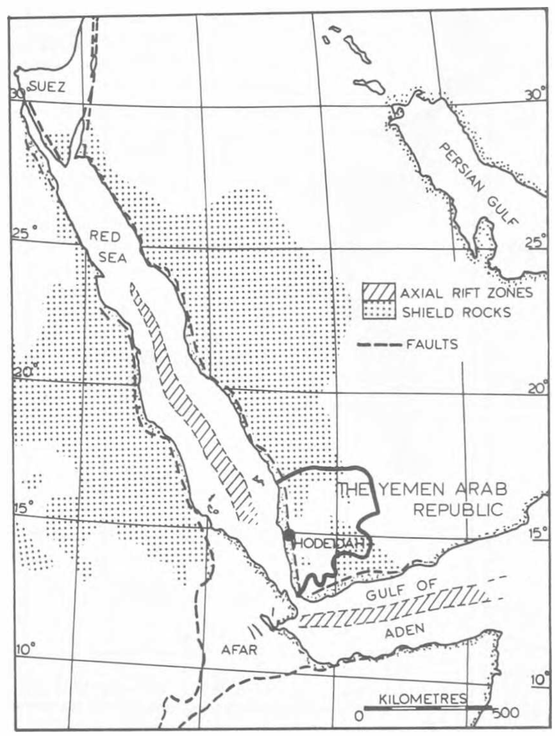

Fig.2 Structural map of the Red Sea showing the axial rift valley and region of continental separation. The Yemen Arab Republic and Hodeidah appear on the map.

The tectonics of the region forms part of the general Afro-Arabian rift system, which extends 6500 km from Turkey to Mozambique and includes the Dead Sea, the Red Sea, the Gulf of Aden and the African Rifts. The latter three branches radiate from an area known as the Afar triangle (Fig.2).

Before the tendering phase, the soil conditions at the site were assessed by five in-situ Standard Penetration Tests (SPT), combined with the taking of disturbed samples for visual classification and some of them for sieve analysis.

The SPT-results are shown in Fig.3. The upper layer is very loose, giving N-values from 4 to 8. At the -10.0 level, a gradual increase in resistance of the SPT occurs. Nowhere below this level do the N-values fall below 18. Below the -17.0 level, the N-values exceed 30. The strata generally have a granular structure, consisting mainly of sands and silts. Especially the loose top layer is granular, and it can be described most aptly as silty sand.

REQUIREMENTS FOR FOUNDATION DESIGN

The dimensioning of a silo foundation, as of all foundations, has to be done in such a way that the working loads are carried with an acceptable margin of safety and without causing excessive settlement. On loose ground, these requirements were difficult to fulfil because of the heavy loads of the silo and the high earthquake risk.

The basic data for earthquake dimensioning are based on the Imperial College report "Assessment of seismic risk in the Hodeidah area of the Yemen Arab Republic". In this report, a Red Sea zone 900 km long and 300 km wide, as shown in Fig.2, is cited as a major source of earthquakes. This zone extends about 75 km to the east and 225 km to the west of Hodeidah and is situated midway in the approximate north-south line. According to an acceptable 10 % probability of an earthquake exceeding the Richter magnitude of M = 5.75, a ground acceleration of 0.2 g and a corresponding ground velocity of 0.2 m/s are used for the dimensioning.

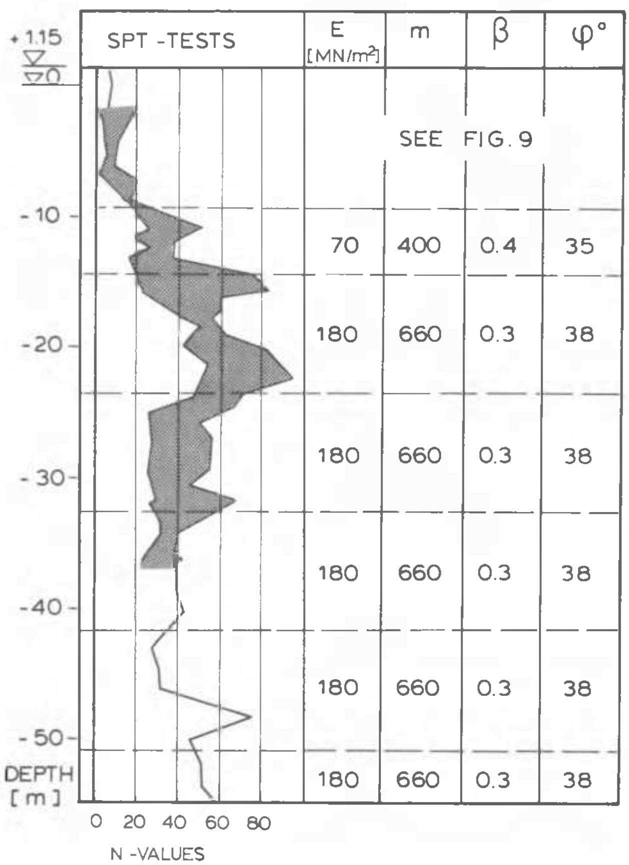

Fig.3 SPT results of five tests at the site and compressibility and strength coefficients assumed for control calculations.

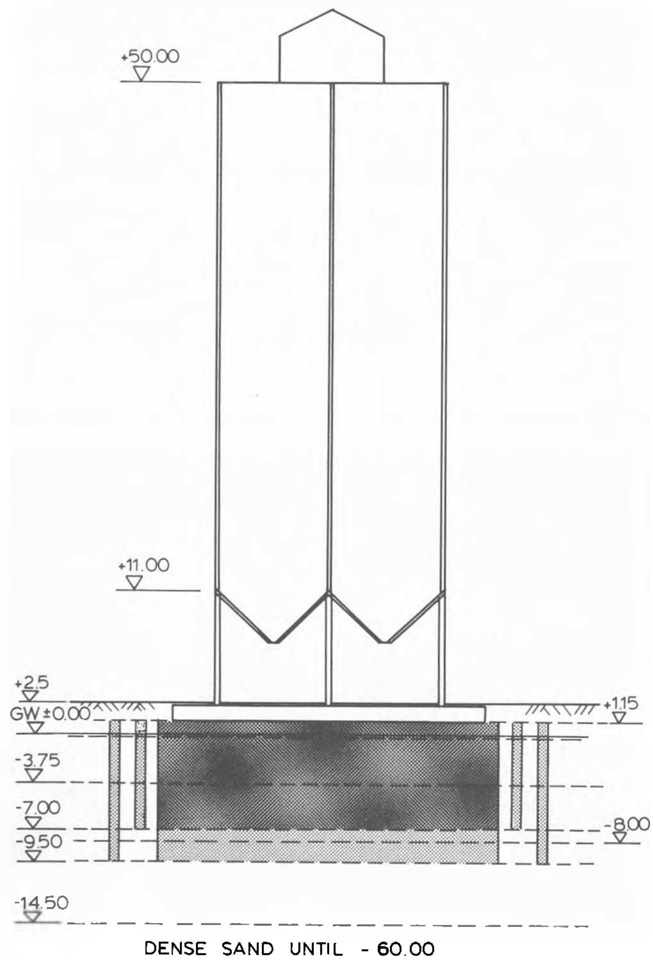

Fig.4 General section of the silo and its base.

GENERAL PLAN

The whole silo structure consists of 2 x 6 main silo bins with an inside diameter of 8.5 m and of 5 inside silos between the main silos. The length of the slip-formed concrete silos is 36...39 m, rising to the +50.0 level. Beneath the silos are hoppers and rounded walls, which support the silos rigidly to the 24 m x 55 m bottom plate (Fig.4). Above the silos is a conveyor. The total height of the whole structure is about 55 m. On the sea side, there is an elevator housing. A bagging plant and an office were also included in the project.

FOUNDATION ENGINEERING

The letter of invitation demanded that the contractor do the foundation engineering design using driven piles; and the design was done for tapered Raymond piles $\varnothing_{max}$ = 415 and L = 17.5 m and precast driven piles 275 x 275 and L = 18 m. Needed in the dimensioning were 432 Raymond piles with Q_u = 2.6 MN, Q_{max} = 1.15 MN, M_{max} = 0.15 MNm or 532 precast driven piles with Q_u = 2.0 MN, Q_{max} = 1.0 MN and M_{max} = 0.09 MNm. The dimensioning called for considerable reinforcement in the piles because of the great bending moment caused by the lateral forces at earthquake shock. Consequently, in the strenuous contract competition, the pile foundation solutions were perceived to be too expensive. In addition, in the event of an earthquake, the liquefaction of loose surrounding soil is liable to occur. Since the compaction achieved by pile driving prevents liquefaction only 2 m outside the piles, this may not suffice to allow the 10 m thick loose soil layer to transfer the horizontal forces to the surrounding soil. For these reasons, as a solution to the foundation problem, the improvement of the ground by utilizing stone columns was recommended.

DIMENSIONING OF THE FOUNDATION

The maximum contact pressure under a 24 m x 55 m raft foundation during a possible earthquake was calculated using a mass-spring-damper system as described in Fig.5. Because the silo is very rigid in comparison with its foundation, only one degree of freedom has been assumed. The earthquake wave is assumed to be sinusoidal with a maximum acceleration of a_o = 0.2 g = 2 m/s^2 and a maximum velocity of v_o = 0.2 m/s. Then the period is T = 2 $\pi v_o/a_o$ = 0.628 rad/s. The maximum contact pressure of the foundation raft has been analysed as a Winkler foundation.

The maximum inclination of the structure can be solved from the differential equation:

$$(J + M h_o^2)\ddot{\phi} + c_o \frac{B^2 L}{6} \dot{\phi} + k_o \frac{B^2 L}{6} \phi = \qquad (1)$$

$$M h_o \dot{Y}_o \sin\omega t$$

where J is rotational inertia of structure
M is mass of structure
L is length of structure
B is width of structure
k_o is coefficient of subgrade reaction
c_o is damping coefficient.

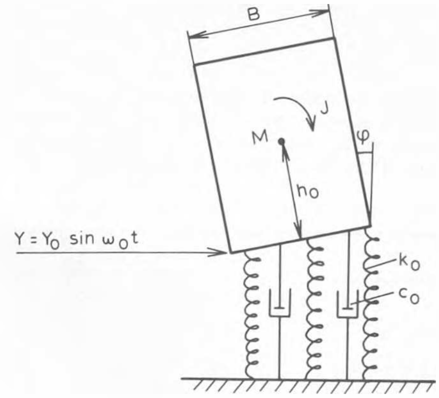

Fig.5 Mass-spring-damper system.

Maximum pressure p_{max} at time t can now be calculated from the equation:

$$p_{max} = \frac{M\,g}{B\,L} + \frac{R}{Z}(c_o \dot{\phi} + k_o \phi). \qquad (2)$$

By using k_o = 8.3 MN/m^3 and c_o = 140 Mg/m^2 in computing the maximum edge pressure, p_{max} ∿ 430 kPa was found for both centric and excentric loading with 80 % filling of the silos. The total weight of the structure is 135 MN and of the grain filling 200 MN. Hence the weight of the grain filling in centric loading is 160 MN and in excentric loading 86 MN. Then the natural period of the silo is 0.7...0.8 s. By using the lateral earthquake load as static load, the maximum edge pressure in centric loading was p_{max} = 450 kPa and in excentric loading p_{max} = 400 kPa. Therefore, none of these values makes any big difference in dimensioning for bearing capacity, because the factor of safety according to DIN 4017 (1979) is close to five for inclined loading in all cases when a low value for friction angle ϕ = 32^o is used.

GROUND IMPROVEMENT

The upper 10 m of loose silty sand is quite easy to liquify under earthquake shock (Fig.6). To prevent this, the geotechnical dimensioning of ground improvement can be done in two ways. The sand has to be compacted to a sufficient density to counter liquefaction and the discharge passages reserved for pore pressure through the stone columns and the stone fill beneath the raft permit the pore water pressure to dissipate, which also prevents liquefaction.

According to Seed and Idriss (1971) liquefaction does not occur at 0.2 g earthquake when soil is compacted to a relative density of Dr $\geq$ 70 %. This density corresponds to a cone resistance of Cone Penetration Test (CPT) q_c = 6 MPa at the

0.0 level and q_c = 8 MPa at the -9.5 level (Fig.9, broken line). According to the few grading curves given (Fig.6, dark raster), which showed a silt content of less than 10 %, it was planned to put stone columns in a 2 x 2 m square grid. When this working method was tested in the test area, it was perceived that it was impossible to obtain acceptable results with this method. Especially between the levels 0.0. - 1.0 and -4.0 - -6.0, there were very loose layers where the silt content was high (Fig.6, light raster); and none of the samples taken during the original site investigation was taken from these layers.

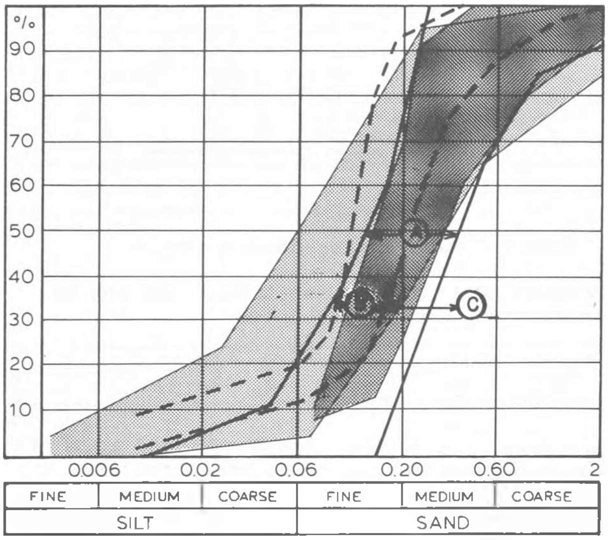

Fig.6 Grading curves from the soil investigation (Zone of dark raster) and from the samples taken during the soil improvement work (Zone of light raster) compared with the zone of easiest liquifaction of soil material (Zone A) and with the suitability zones of deep compaction (Zone B difficult and zone C easy to compact, Brown, 1977).

After noticing the difficulties in compaction by the original grid, it was decided to make the stone columns in two phases. The first phase involved the original grid and the second involved putting a shorter stone column at every midpoint of the original grid (Figs. 7 and 8). Working in two phases made it possible to obtain good compaction results (Fig.9). The second phase is highly effective for two reasons:

(i) Compaction is effective when the soil material between the columns is pushed against the columns made earlier.

(ii) Excess pore-water pressure can be dissipated through the columns laid during the first phase of the operation.

The compaction results were controlled by the Cone Penetration Test (CPT) and the results are shown in Fig.9. They indicate that a 70 % relative density is, on the average (Fig.9, full line), clearly exceeded.

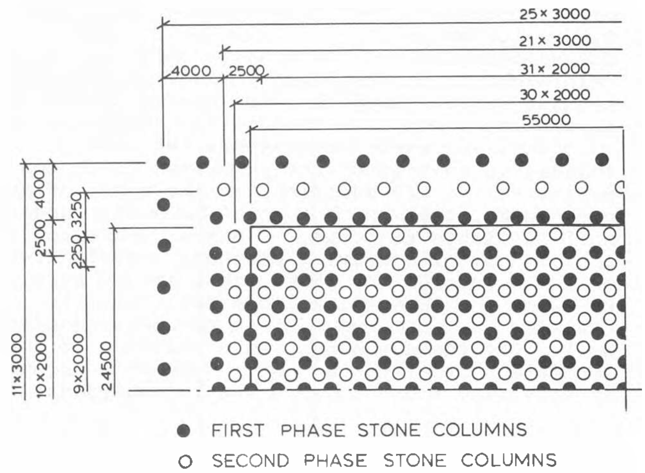

Fig.7. Ground improvement layout.

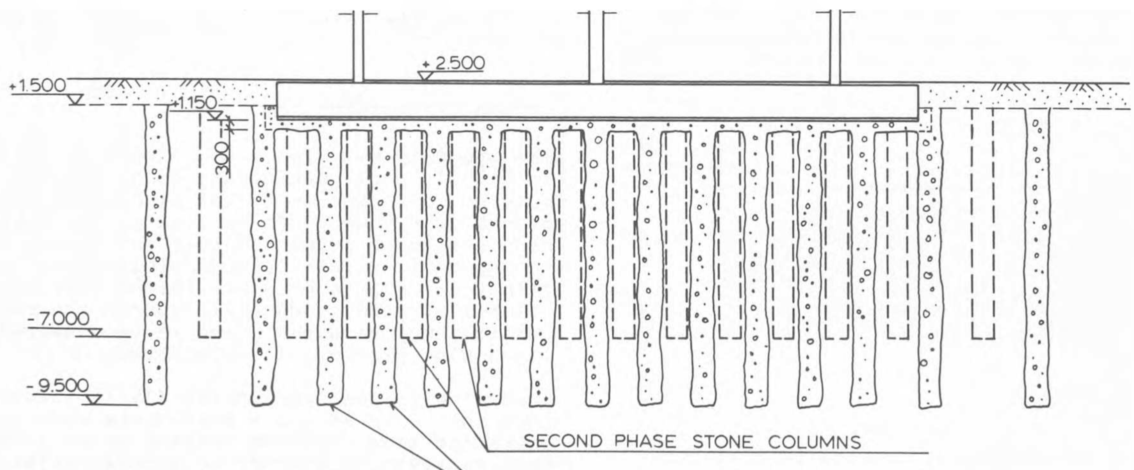

Fig.8 Ground improvement section.

A further analysis revealed that stone columns acting as gravel drains could prevent liquefaction even if loose lenses might occur. The analysis was done after Seed and Booker (1976) taking the duration of the strong phase of shaking during an earthquake of 5.75-magnitude according to Valera and Donovan (1977). The average duration of an 5.75-magnitude earthquake is 12.5 s and the maximum duration 20 s, corresponding to 6 and 9 uniform stress cycless at a shear stress level of 0.65 x τ_{max}. The calculated excess pore-water pressure is shown in Fig.10. The relatively small excess pore-water pressure will not essentially reduce the stability against liquefaction. The factor of safety against liquefaction is in the range of 2.0 (Wong, Seed and Chan, 1975).

SETTLEMENT CALCULATIONS AND MEASUREMENTS

The preliminary design was done to the allowable maximum settlement of 100 mm and to an angle of differential settlement of less than 1:750. The preliminary settlement estimates were made assuming the compressibility modulus in the treated zone of E = 40 MPa and in the untreated zone E = 70 MPa. These values gave for the total loading of p = 250 kP_a a total settlement of about 100 mm, of which one half should occur in the treated and the other half in the untreated zone.

During and after the ground improvement, several settlement calculations were done by the Geotechnical Laboratory of the Technical research Centre of Finland as well as, by the contractor and the subcontractor using Baumann's and Bayer's (1974) method. These analysis gave total settlement values ranging from 73 to 138 mm. The most probable estimate for the total settlement was 80 mm. To be on the side, a low molulus of compressibility of E = 23.5 MPa for the treated zone was taken for further calculation. This gave a total settlement estimate of about 120 mm.

The interaction between the silo construction and the ground soil - contact pressure, bending moments and settlements of silo construction - was calculated in several stages, of which the most important are:

(i) foundation raft and superstructure ready up to the funnel level and the the first half of the silos concreted
(ii) second half of the silos concreted
(iii) silo construction ready and filled.

It took one and a half years from concreting the foundation raft to filling the silo. Because of the creep in slow loading, the elasticity modulus of the concrete was reduced from E = 26 kN/mm^2 to E = 10 kN/mm^2. The moment of inertia used for the whole structure was J = 520 m^4, according to a simple analysis. The settlement calculated with these values is shown in Figs. 11, 12 and 13 as a preliminary estimate. This calculation was done for the loading, from which the weight of the foundation raft, p = 30 kP_a, is omitted. This is because the bolts for the settlement measurement are installed on the completed foundation raft.

When the settlement measurements were started and as work continued, it was seen that the settlements at the beginning were in quite good

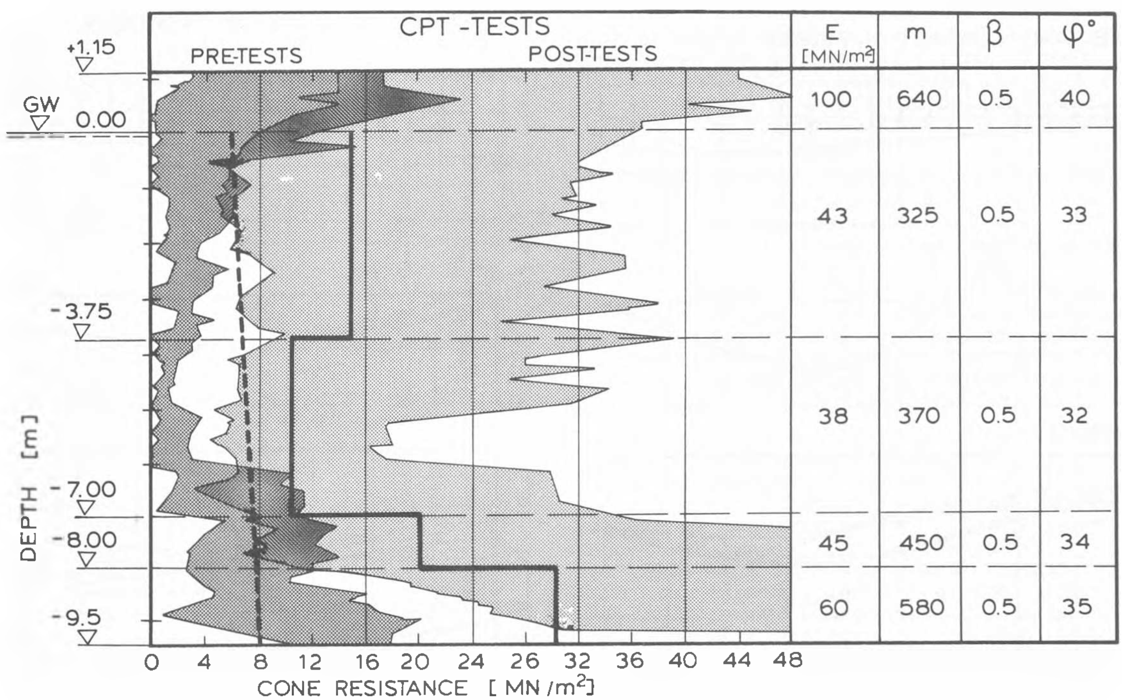

Fig.9 CPT-results from 10 pretests and 20 post-tests. Comparison between them shows clearly the influence of ground treatment. Compressibility and strength coefficients represent the average values in the treated zone, including stone columns. Compressibility modulus of stone columns is assumed to be E = 60 MPa.

agreement with the preliminary estimates. When the loading was increased, the difference between estimated and measured values started to increase, with the result that measured settlement was smaller. This is due to the fact that when a constant compressibility modulus is used, correct settlements cannot be obtained. The error increases in deeper layers because of the heavy overburden pressure. That is why Janbu's (1970) formula should be used for the stress dependent compressibility modulus.

Accordingly, acontrol calculation was done using in Figs. 3 and 9 given m- and β-values to find a good agreement between the measured and

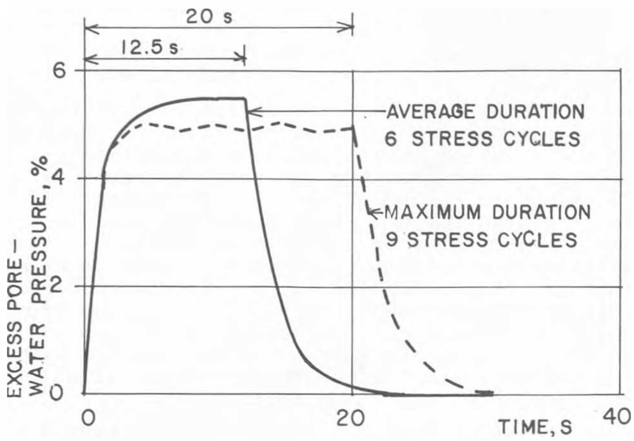

Fig.10 Pore-water pressure caused by the average and maximum duration of 5.75 M earthquake in the soil between stone columns.

calculated values. For the treated zone, m- and β-values are calculated through E values, which are found from average CPT-results and assuming the stone columns to have a compressibility modulus of E = 60 MPa according to Engelhardt and Kirsch (1971).

The total settlement arrived at by this calculation for the significant point is 56 mm, 36 mm occuring in the treated zone and 20 mm in the untreated zone. This can be compared to the most probable estimate of total settlement, 80 mm, of which the unmeasured part of the foundation raft is about 10 mm. One half of 70 mm is close to 36 mm. Hense, using a constant compressibility modulus makes no difference in the treated zone the thickness, of which is 0,4xB. But, as it can easily be seen in Fig. 14, when the overburden pressure and the pressure caused by the foundation raft are compared, the stress state has a strong influence on the compressibility modulus in deeper layers. Therefore, by using a constant modulus of compressibility, the settlement in deeper layers can be easily overestimated by 100 %.

Resort to a constant modulus is based on the fact that the common methods for calculating the interaction between ground and structure use a constant E-modulus. The control calculation is done by Kany's method (1974). For this calculation the moment of inertia of structure was reduced from J = 520 m^4 to J = 200 m^4, which was achieved by a finite element analysis and by the method of Gossla (1977). In order to minimize the inoccuracy done by using a constant E-modulus the calculation method is modified more stress dependent so that at every calculated stage and for every different soil layer, a

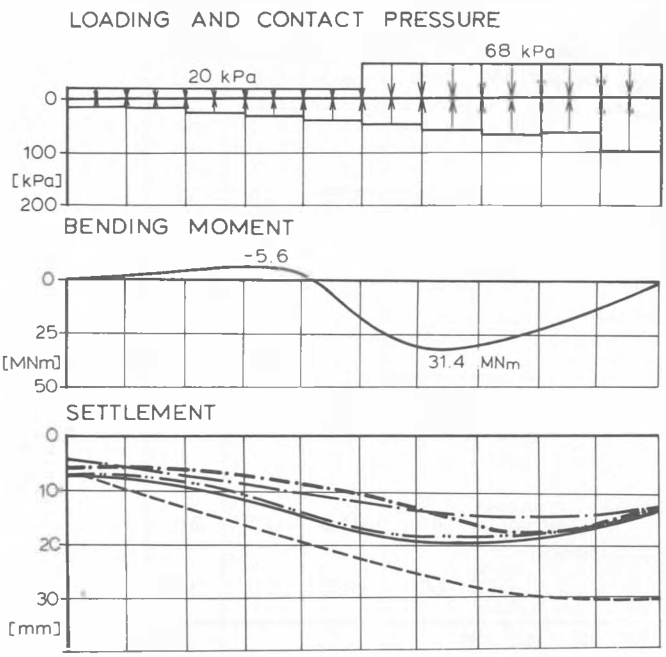

Fig.11 Interaction between the silo construction and ground after completion of the slip-forming of the first half of silo bins. (See Fig.13).

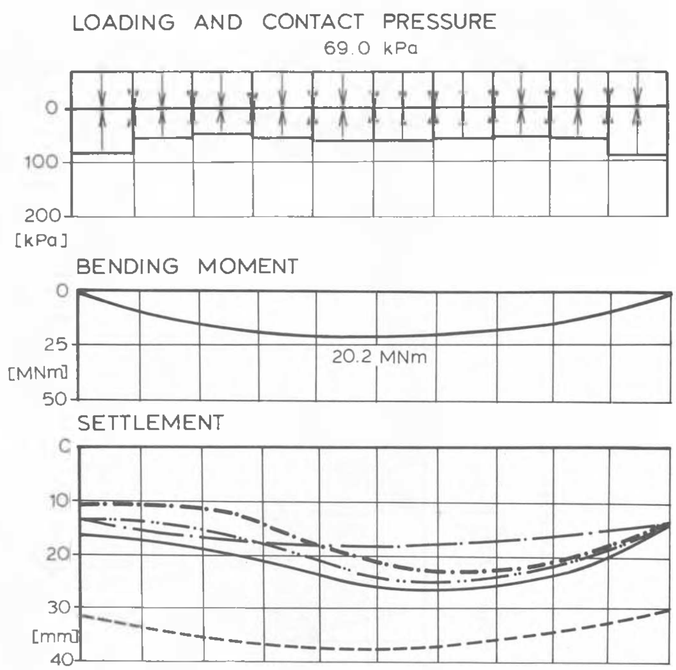

Fig.12 Interaction between the silo construction and ground after completion of the slip-forming of the whole silo. (See Fig.13).

different E-modulus is used, corresponding to the stress state at this stage. The calculation shows that the actual structure is still a bit more flexible than the calculated one. For better agreement, the study should proceed with a finite element analysis of the interaction between ground and structure. In any case, both the total settlement and the differential settlement of the silo structure fall very neatly within the original limits (Figs. 11, 12 and 13).

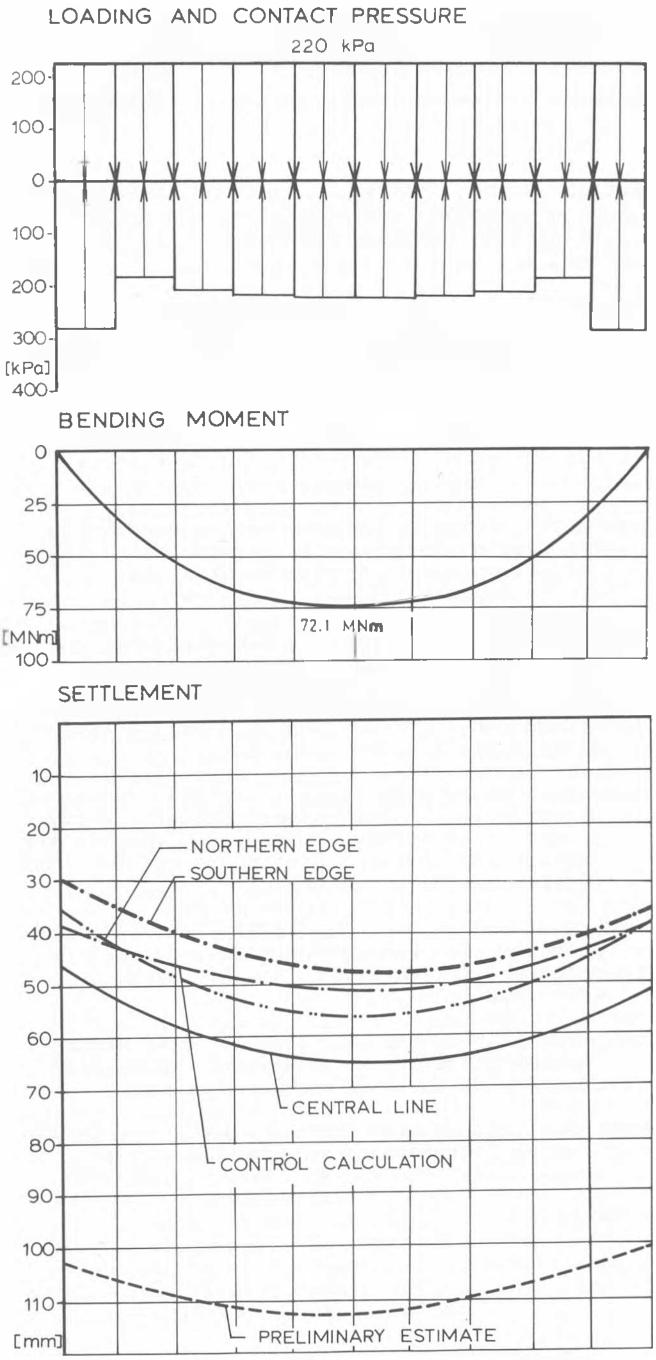

Fig.13 Interaction between the silo construction and ground after filling of silo bins.

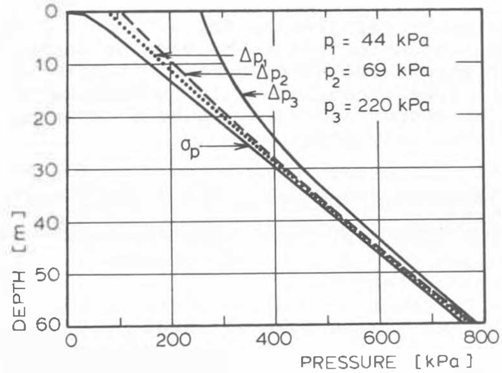

Fig.14 The effective overburden pressure compared with the vertical stresses, which were caused by the different stages of silo construction in the ground.

CONCLUSIONS

The low cost of the soil-improvement solution in comparison with the pile solution made it possible to win the hard international contract competition. In spite of the difficulties encountered in the deep compaction operations and the increase of about 60 % in the quantity of stone columns, the cost was still only about one-fourth of the estimated cost of Raymond piles.

The economic solution is based on the technical fact that the horizontal forces brought into play by a possible earthquake are much easier to transfer from the raft foundation to the compacted ground than through vertical piles to a less compacted soil layer.

The stone-column treatment used is very effective against the liquefaction danger. It prevents liquefaction in two ways:

(i) when sand is compacted to a density sufficient ($D_r \geq 70$ %) (Fig.9) to counter liquefaction, and

(ii) when liquefaction threatens somewhere because of loose lenses, the system of stone columns and the stone fill beneath the raft will keep excess pore-water pressure at an insignificant level (Fig.10).

Ground improvement in general, especially deep compaction by stone columns, is highly sensitive to relatively small variations in ground conditions. For this reason, soil investigation should be accurate. For example, such a large number of samples for grain-size analysis has to be taken that they must surely represent also the most fine-grained and so most difficult soil layers for deep compaction. It also showed that the Standard Penetration Test (SPT) is too rough an investigation method for deep compaction. The Cone Penetration Test (CPT) seems to be more sensitive and therefore much more suitable an investigation method for ground treatment.

A very slight increase in silt content from 10 % to 40 % made the soil considerably more difficult to compact. In this case, 60 % more stone columns than called for by the original plan (Figs. 7 and 8) were needed to achieve the desired compaction result.

A settlement calculation for a large and heavily loaded foundation has to be made by using Janbu's (1970) stress dependent compressibility modulus. The settlement can be easily overestimated by 100 % in deeper layers by using a constant compressibility modulus.

The silo structure seems to be more flexible than is often assumed. There are two reasons for this:

(i) The creep in slow loading reduces the elasticity modulus of concrete quite considerably and
(ii) the moment of inertia of the silo structure can easily be overestimated by a simple analysis. A finite element analysis and the method of Gossla (1977) seem to yield values of flexibility in sufficiently close agreement for ground-structure-interaction calculations.

Although the silo is more flexible, the differential settlement is still nicely within the original limit of 1:750. The advantage of more flexibility is that it decreases the bending moment of the whole silo structure and consequently also the need of reinforcement.

The general compressibility modulus methods for interaction calculations use a constant E-modulus. With such methods, it is impossible without modifications to follow the settlement and deflection behavior of the structure accurately at different stages of construction. The changing stress state in the ground during the construction period must be taken into consideration at least by changing the compressibility modulus according to the stress state.

The dimensioning bending moments and shear forces at all the important stages of construction were determined by interaction calculation. It was shown that the strains caused by concreting the silo with two slides were not critical. It was thus possible to fix the two slides together without any trouble. This was a noticeably economic measure, since only half the slip-forming equipment was required.

Dimensioning for a possible earthquake involves challenging philosophy, because without installing vibratory gauges on the foundation and measuring the earthquake shock on the actual foundation, it is quite impossible to determine whether an earthquake is inside or outside the limits for which the structure is dimensioned.

ACKNOWLEDGMENTS

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