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Preloading for Large Storage Building

A. C. Stamatopoulos
Kotzias-Stamatopoulos Co. Ltd., Athens, Greece

P. C. Kotzias
Kotzias-Stamatopoulos Co. Ltd., Athens, Greece

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A. C. Stamatopoulos, P. C. Kotzias

Partners, Kotzias-Stamatopoulos Co. Ltd., Athens, Greece

SYNOPSIS The properties and method of preloading a 11 m erratic deposit of soft clays and loose sands, are described. Under the preload embankment of 12 m the deposit compressed an average of 7%. Settlement developed rapidly and excess porepressure dissipated within few days, never exceeding 2.1 m of water head. After preload removal borings were performed to compare results with those obtained on the untreated site. This comparison showed that the compressibility and strength of both the clayey and sandy components improved substantially and that the subsurface became more homogeneous. The projected structure built on the improved ground has performed satisfactorily.

SITE AND PROJECT DESCRIPTION

Site is 150 kms NW of Athens, Greece, on a coastal plain of intermediate seismic activity. Building location, about 300 m from the sea, was originally covered by marshland. Surface elevation, referenced to low sea level, was +0.3 m. A steel rolling plant, a ferroalloy smelting plant and other industrial installations within a distance of 1 km, had been founded on 12-16 m piles.

Site preparation included removal of 0.6 m topsoil and filling to elevation +2.0 m, with compacted sandy gravel, weighing 21 kN/m^3.

Building has a length of 144 m, width 33 m and height 16 m. Contained storage is ore in heaps up to 14 m high, weighing 21.6 kN/m^3 and having an angle of repose of 40°. Roof is supported on 3-hinged steel frames, at 12 m spacings, founded on two continuous reinforced concrete footings along the 144 m sides. A double telescoping conveyor suspended from the roof should not suffer differential settlements between successive frames, of more than 0.02 m.

SUBSURFACE CONDITIONS

Five boreholes showed recent random deposits of soft and compressible soils followed below elevation -10.9 m by very stiff clay. The type and variability of soils is illustrated by the results of the two typical borings shown in fig 1. Soft soils consisted mainly of silty sand, silty clay and sandy clay, SM, CL and SC according to the unified soil classification system (Lambe and Whitman, 1969), respectively. The proportion of boring length that penetrated non plastic and plastic soils was 51.7 and 48.3%, respectively. The standard penetration resistance as defined by ASTM (1976), in blows/0.3 m, (51 results) was found to vary between zero and 21, with mean 6.0; about one third of the results were 0, 1 and 2. The natural water content (62 results) had a mean of 31.8%. Also, 19 field permeability tests gave a coefficient of permeability, in 10^{-5} m/sec, from 0.1 to 4.0 with mean 2.9 and coefficient of variation 2.8. The modulus of compressibility was found to vary between 2.5 and 7.0 MN/m^2, with mean 4.0 MN/m^2. The mean shear strength from triaxial compression tests was 54 and 13 kN/m^2 for consolidated undrained (CU) and unconsolidated undrained (UU) tests, respectively. The intergranular angle of friction was found equal to approximately 35°.

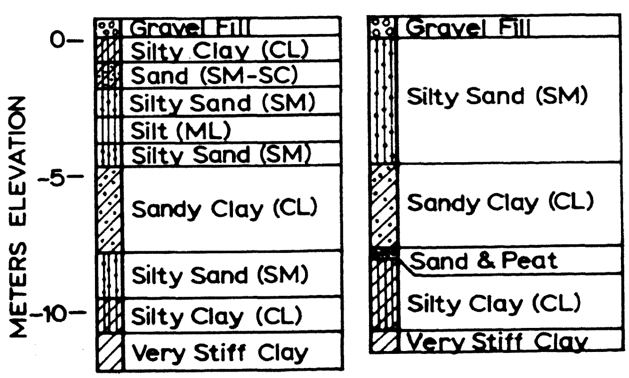

Fig 1 Typical Soil Borings

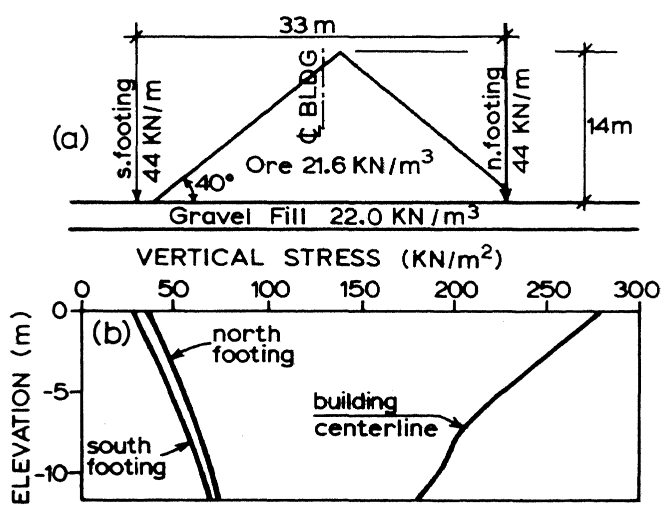

Fig 2 Building Loads and Vertical Stresses

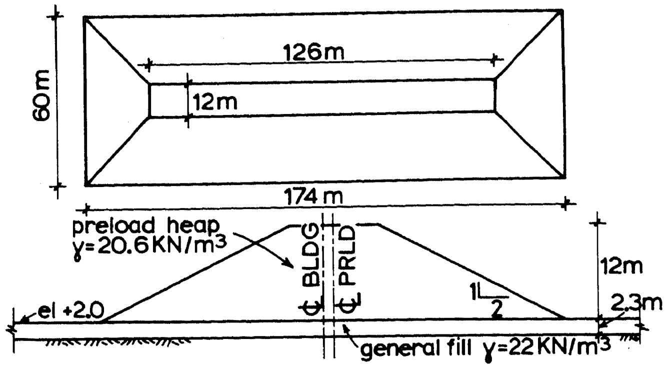

Fig 3 Plan and Section of Preload Heap

DECISION TO PRELOAD

The loads of the building with contained storage and general fill (fig 2a) cause the vertical stresses shown in fig 2b. Settlement computations gave 0.72 m along the building centerline and 0.20 m and 0.18 m along the north and south footings, respectively. As these settlements were judged to be inadmissible, it was decided to preload by a 12 m embankment of total volume 65,000 m^3, weighing about 1.8 as much as the permanent structure (fig 3). Vertical stresses under preload were calculated to be generally higher than under the permanent structure, in the ratios shown in fig 4. Preload settlements were estimated at 0.76 m along the centerline and 0.48 m along the footings.

The alternative to preloading would have been 15 m long piles under the footings and 12 m long stone columns under the floor of the building; the resulting cost would have been about six times as high as that of preloading.

TIME PREDICTION AND CONSTRUCTION SCHEDULE

The coefficient of consolidation c_v for the overall formation of soft soils, was estimated from the equation $c_v = kD/\gamma_w$, where k = coefficient of permeability, taken on the basis of field tests equal to 2.9 x 10^{-5} m/sec, D = modulus of compressibility, taken on the basis of laboratory consolidation tests equal to 4.0 MN/m^2 and γ_w = unit weight of water = 9.81 kN/m^3. The value of c_v = 0.012 m^2 sec^{-1}, thus determined, lead to the prediction that primary consolidation will follow closely the placement of preload. But for reasons of caution, it was decided to allow more time for consolidation than indicated by this calculation, especially as it was recognized that within the overall layer of compressible soils there might exist sizable clay pockets. The original tentative plan was to build the embankment to one half of its final height (but 71% of the volume) in 30 days, to pause from the 31st to the 60th day, to complete the embankment from the 61st to the 80th day, to consolidate from the 81st to the 180th day and to demolish the embankment from the 181st to the 210th day. These times were counted from the end of site preparation to elevation +2.0 m.

FILL MATERIAL AND CONSTRUCTION OF THE PRELOAD HEAP

Construction of the heap was preceded by site preparation over an area 225 m x 110 m (fig 5). The material used for site preparation and also for the first one meter of the preload heap, was clean river gravel, assuring unobstructed drainage of consolidation water and satisfactory subgrade for the floor of the building. While work was in progress, it was decided to change the type of fill, for economy, and so the bulk of the heap was formed of gravelly-sandy clay, with about 60% passing sieve no 200, liquid limit 25-45% and plasticity index 15-25%; natural water content was within 3% of optimum.

Material was placed in lifts of 0.5 m and some compaction was provided by hauling trucks and the bulldozer and grader that were levelling the surface. Wet unit weight was approximately 22 kN/m^3 for the river gravel and 19 kN/m^3 for the clay.

The rate of construction was initially slow and later erratic. The uneven progress was due to delays in developing borrow pits, to contractor's organization and also to adverse weather. When the heap reached elevation +10.5 m construction was interrupted by rains for 20 days, thus

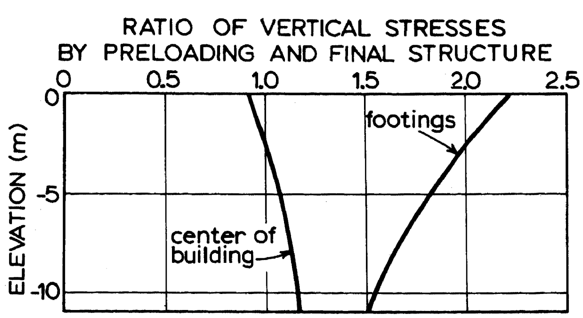

Fig 4 Ratios of Preload to Building Stresses

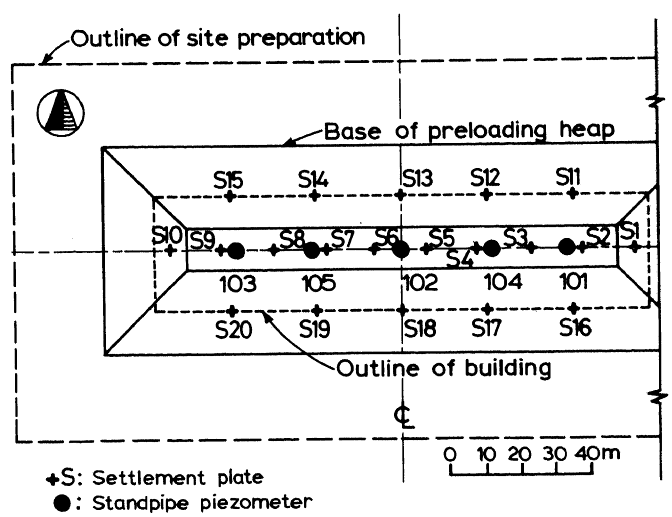

Fig 5 Plan of Preload and Instruments

creating a de-facto second pause (fig 6).

INSTRUMENTATION

Twenty settlement plates and 5 standpipe piezometers were installed at the locations shown in fig 5. Damages during the first weeks of embankment construction were repaired rapidly so that the continuity of readings was not lost. The chances of further damages were limited by placing sections of concrete pipe around the instruments. Only near the end of preloading 4 settlement plates became inoperative, two (nos 2, 13) because of collision of construction equipment and two (nos 16, 17) because of excessive deviation from the vertical.

Readings were taken 2-3 times per week and plots of results were kept strictly to-date.

SETTLEMENT AND POREPRESSURE READINGS

Fig 6 gives an overall view of loading, settlement and porewater pressure on the same (linear) time scale. Plotted settlements are the mean values of observations along the center (nos 1-10) of the building and along the north (nos 11-15) and south (nos 16-20) sides. Fig 7 shows the mean settlements along the center, north footing and south footing, plotted versus the logarithm of time, for the period of final consolidation. The resulting straight lines conform to case C of Stamatopoulos and Kotzias (1983). The time rates of settlement during final consolidation can be calculated in terms of the time t, from the equations of the three lines, by taking the derivative ds/dt; values in meters/day are 0.044/t, 0.033/t and 0.026/t, for the center, south footing and north footing, respectively.

The time rate of settlement along the center of the building (fig 6c) was about 155 mm/day at times of intense loading but decreased drastically within few days after load discontinuation.

The plot of piezometric levels (fig 6d) resembles that of the time rate of settlement, both rising during rapid loading and dropping during pauses. This similarity of response indicates that piezometers worked satisfactorily. The readings at no 104 were somewhat less than those at the rest of the piezometers, probably because of a layer of more than average permeability at 104, facilitating pore pressure dissipation. In any case, even at times of rapid loading, piezometric levels did not exceed elevation +2.5 m, which is only 2.1 m above the initial level.

CALCULATION OF STABILITY

The factor of safety FS against a base failure was calculated using Bishop's simplified method of slices. Before construction, the material above elevation zero had been assumed to be of river gravel, with random inclusions of fines; a field reconnaissance of the source had indicated that natural cuts stood vertically about 3 m and thus it was decided to allow a cohesion of 10 kN/m^2, in addition to the friction of 30° that also seemed reasonable. The minimum FS, ignoring seismic forces, was found to be 1.22.

The stability analysis was repeated using the true rather than the design fill material, and also taking into account the pore pressure readings; the angle of friction of foundation soils was assumed 32°, i.e., somewhat less than the value of 35° found from triaxial compression tests. The resulting FS was 1.34. This result, coupled with the quick response to loading, allowed the discontinuation of final consolidation after only 46 days from its beginning

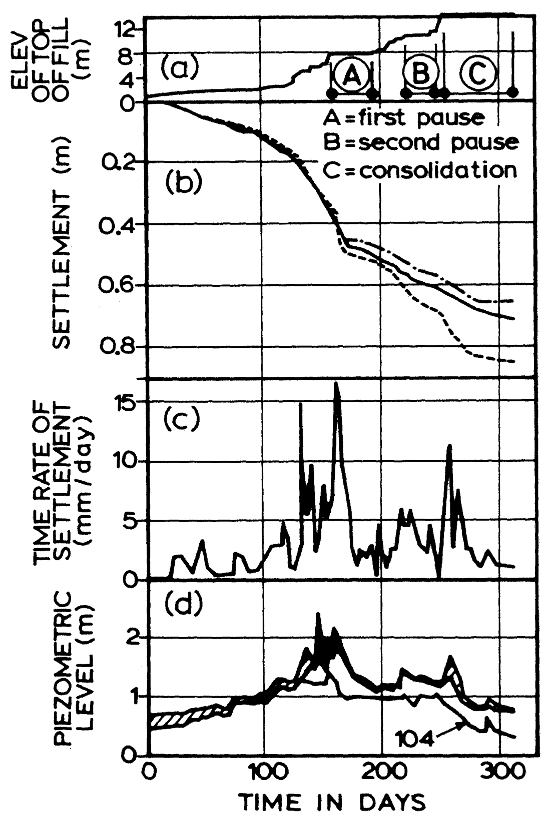

Fig 6 Load, Settlement, Time-Rate of Settlement and Piezometric Level Versus Time

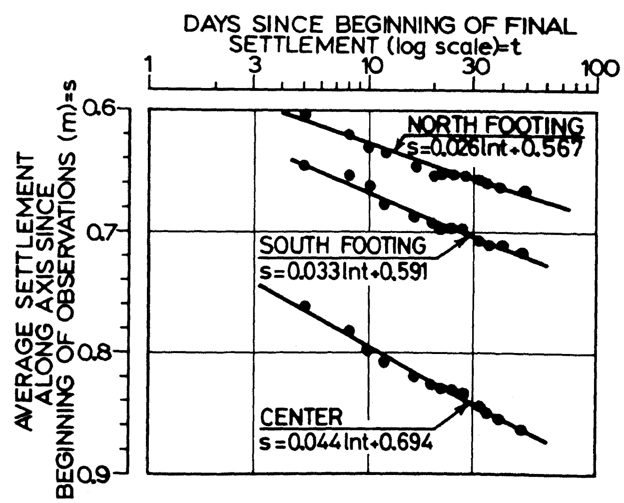

Fig 7 Consolidation Settlement Versus Log Time

instead of the 100 days originally envisaged.

SOIL IMPROVEMENT

Following removal of preload, 5 borings and related tests were carried out, starting at elevation +2.0 m and ending at -13.0 m, to gauge soil improvement attained by preloading. Basic findings follow:

1. The upper boundary of soft soils moved from about elevation -0.3 m to -1.1 m; the lower boundary remained unchanged.

2. The mean standard penetration resistance, in blows/0.3 m, increased from 6.0 to 16.1 i.e., by 168%. The zero and other very low values (16 results of 0, 1 and 2) that were observed before preloading were completely eliminated, the lowest value observed after preloading being 7. The increase was about the same for cohesive and cohesionless soil.

   It was further observed that the standard penetration resistance before preloading has a skewed distribution, probably because values are constrained to be positive, whereas after preloading it is close to normally distributed. The coefficient of variation decreased from 0.9 to 0.4, indicating that in addition to improving the soft soils, preloading has also made them more homogeneous.

3. The in situ permeability decreased 9-fold, from a mean value of 2.9×10^{-5} m/sec to 0.33×10^{-5} m/sec. Here again there is an improvement of the coefficient of variation from 2.8 to 2.0.

4. The mean value of the water content dropped from 31.5 to 26.9%, which corresponds to a change of the dry density approximately from 1460 kg/m^3 to 1560 kg/m^3. This result implies a reduction in volume by about 7%, which is about equal to the magnitude of settlement (fig 6b) divided by the thickness of the soft soils ($0.8/10.6 \simeq 0.07$).

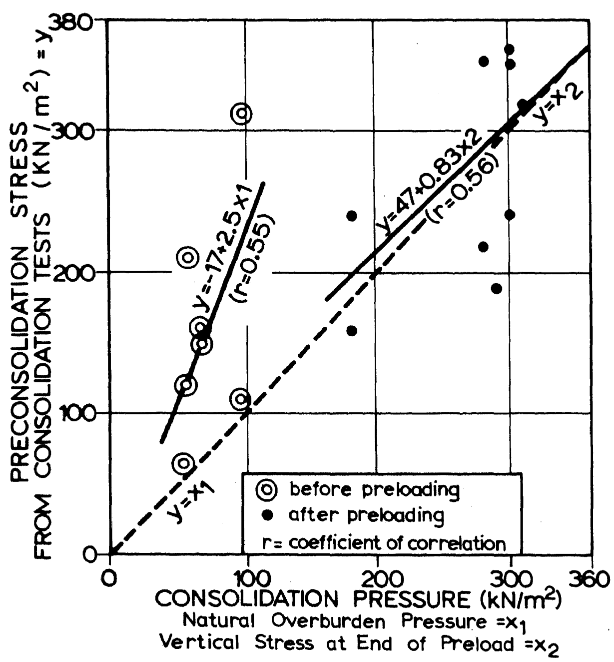

Fig 8 Preconsolidation Stress Versus Consolidation Pressure

5. The mean modulus of compressibility increased from 4.0 to 6.0 MN/m^2. The coefficient of variation improved from 0.48 to 0.37.

   Fig 8 shows the variation of the preconsolidation stress, as determined from the void ratio-log pressure plots of the results of consolidation tests (Lambe and Whitman, 1969), with the consolidation pressure. The preconsolidation stress before preloading is considerably higher than the natural overburden stress, a fact that suggests aging or overconsolidation. The preconsolidation stress after preloading is close to the vertical stress induced by preloading, a fact that confirms that soil improvement was carried to a satisfactory degree.

6. The shear strength of clay from UU tests increased 3-fold during preloading. In the case of the CU tests the mean value changed from 54 to 85 kN/m^2, i.e., it increased by 57%.

PREDICTION OF FUTURE SETTLEMENTS AND BUILDING PERFORMANCE

The future settlements of the permanent structure were estimated taking the constrained modulus of the improved soil equal to 6.0 MN/m^2. Values were 0.35 m along the center and 0.09 m and 0.08 m along the north and south footings, respectively. In view of the increased degree of homogenuity created by preloading it was concluded that the requirement of a maximum differential settlement between frames of 0.02 m will be met.

Building superstructure was completed during the last months of 1982 and ore started being stored immediately afterwards. The operation of all structural and mechanical elements has since been satisfactory.

ACKNOWLEDGMENTS

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