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# Settlement of a building founded on soil improved by stone columns

## Tassement d'un bâtiment fondé sur un sol amélioré par des pieux en gravier

Mulabdic Mensur – Assistant Profesor, Faculty of Civil Engineering, University of Osijek, Croatia

**ABSTRACT:** An industrial building, very sensitive to differential settlements, was founded on soft soil only after the soil was improved and a rigid raft foundation added. Stone columns 0.8 m in diameter, 13 m long and 1.4-1.7 m apart, were used for soil improvement. The distance between stone columns is related to the different load intensity over the raft. The measurements of settlement of the raft foundation were conducted by a specially developed type of profilometer, in order to verify the compliance with the strict angular distortion limit set as 1:1000. The total measured settlement was in excess of the predicted settlement, but differential settlements were under the limit. A back-analysis was performed so as to clarify the lower values of predicted total settlement.

**RÉSUMÉ:** Un bâtiment sensible aux tassements différentiels n'a pas pu être fondé sur un sol meuble sans améliorations du sol et sans un radier rigide. Des pieux en gravier, 0,8 m en diamètre, ont été exécutés jusqu'à une profondeur de 13 m dans une grille de 1,4 à 1,7 m, suivant la charge due à l'ouvrage, répartie de façon non uniforme sur le plan. En vue du contrôle des conditions de tassement différentiel (1:1000), il a été procédé à la mesure du tassement de la surface inférieure du radier, à l'aide d'un profilomètre présentant une précision suffisante de mesure des déplacements verticaux. Les tassements totaux ont été supérieurs aux tassements de calcul, tandis que les tassements différentiels ont demeuré dans les limites admises. On a étudié aussi le modèle de calcul de tassement de l'ouvrage pour expliquer la différence considérable entre les tassements calculés et les tassements mesurés.

### 1 DESCRIPTION OF FOUNDATION

The water treatment plant was founded on a rigid raft foundation on soft soil deposit. Different solutions for foundation had been studied (piling, soil improvement), and the foundation soil improvement by the stone columns was chosen.

The soil consists of soft clay of high plasticity down to about 17 m, where a thick layer of dense gravel is first encountered. A thin layer of organic clay was detected at depth interval of 2.7-3.5 m. Ground water table is normally between 1 and 2 m below the surface. The undrained shear strength of clay was determined by piezocone test and by Marchetti dilatometer test as  $c_u = 25-50$  kPa increasing up to 11 m. Further down, it had constant value of 50 kPa. CPTU test gave somewhat lower values compared to the dilatometer test results and at certain depths showed values as low as 15 kPa. The SPT test gave values of  $N=3-5$  (4 in most cases) in clay at 2 m depth intervals.

The Fig. 1. shows the top view of the foundation raft, distribution of loading and columns distance. Unfavorable distribution of load and strict demand for angular distortion limited to 1:1000 required rigid raft foundation and different column distance and depth over the foundation area. Inside walls generate significant contribution to the raft stiffness. The rigid raft foundation consists of 60 cm reinforced concrete placed over 20 cm lean concrete, underlain with about 60 cm thick layer of crushed stone placed over the stone columns. The stone columns diameter is 70-80 cm and length ranges from 7 to 13 m. In the zone were both 10 m and 13 m long columns were used, every second column had length of 10 m. The 7 m long columns were put outside raft by the most loaded columns as additional support. Columns were spaced in square pattern with axial distances 1.4 x 1.65 m to 1.7 x 1.65 m.

Columns were constructed by Keller GmbH, Vienna, using their patented dry vibro-displacement method. Well-graded gravel from crushed stones with grains from 4-60 mm was used for filling. Column quality was controlled by recording electrical energy consumption for gravel densification with depth. Column bearing capacity was not tested. During construction of columns, soil surface heave was observed and measured in the amount of

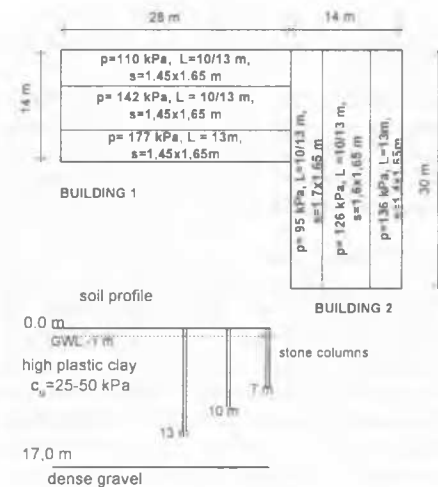


Fig. 1. Foundation raft top view with distribution of loading, indication of column spacing and length and soil profile

60 cm. Therefore the top 60-70 cm of soil-gravel mixture was stripped and replaced by gravel that was compacted over the stone columns.

### 2 MEASUREMENT OF SETTLEMENTS

Design solution predicted maximum total settlements for the improved soil of about 6.5 cm for the building 1 and 5 cm for the building 2. Settlements of improved soil were calculated according to Priebe (1992). Maximum settlements for unimproved soil were calculated as 18 cm for the building 1 and 15 cm for the building 2. That gives coefficient of soil improvement of about 2.6.

Two methods of measurement of vertical movement of the building were used: topographic measurement of some points in the building (at corners of building and external points of the

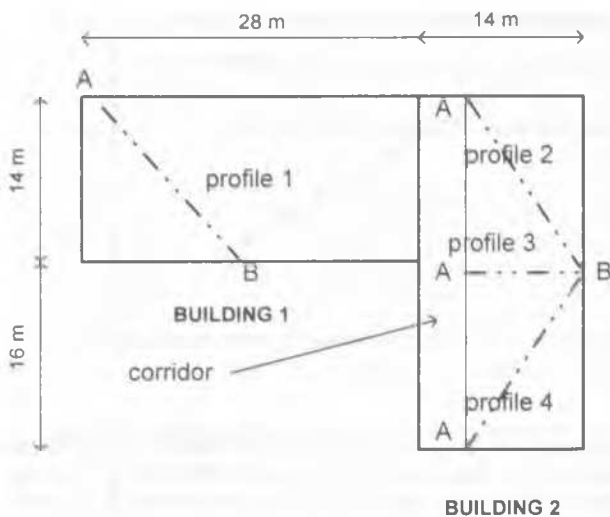


Fig. 2. Profile tubing positions under the buildings

profiles for differential settlement measurement) and measurement of differential settlements under the building.

Measurement of differential settlement limited in terms of distortion to 1:1000 and monitoring of points under the building were required. Therefore special measurement of vertical movement of the foundation was conducted using instrument called profilometer (hydrostatic profile gauge). It measures the water head between the position of the pressure transducer introduced and positioned in a profiling tube under the building and the water level in the water container placed outside the building at elevation above the measuring points. Precise pressure transducer is connected to water container by thin tube saturated with water.

There are several types of profilometer on the market. Due to special requirement regarding distortion much better precision was necessary than that offered by the profilometers available in the market. Also special demands were put on the guiding tubes through which the instrument moves from one position to another.

In the profilometer constructed for this purpose, the pressure transducer is put inside the torpedo body and connected separately to the air pressure and water pressure from the container. Official calibration of the instrument confirmed that movements could be measured with accuracy of  $\pm 1$  mm. The measurement range is 1 m. Digital read out unit is used for transducer reading. In order to assure the same position of the instrument at each point inside the tube in successive measurements at different times, the profile tubing (guiding tube for torpedo) is made of aluminum square profile. The profile tubing was fixed in a lean concrete (bellow the slab and above gravel subbase). Torpedo is sufficiently heavy and fitted with caster for smooth movement through the guiding tube. Markers on the connecting cable were set in 1 m intervals. Measurements were made in points set 2 m apart. Fig. 2. shows one measuring profile under the building 1 and three profiles under the building 2. Position and number of profiles were dictated by the slab and building configuration.

### 3 MEASURED SETTLEMENTS

Settlements were measured as total vertical movements by topographic measurement at corners of the buildings and profiling vertical movements at points 2 m apart of each other along profiles (see Fig.2). End points (A and B) on profiles were monitored for total vertical movement so that every point inside the profile could be interpreted in terms of total vertical movements.

Due to some delay in construction the total loading condition has not been reached yet. At present (January 2001) about 90% of final load is realised (that stage is included in Fig. 3 showing

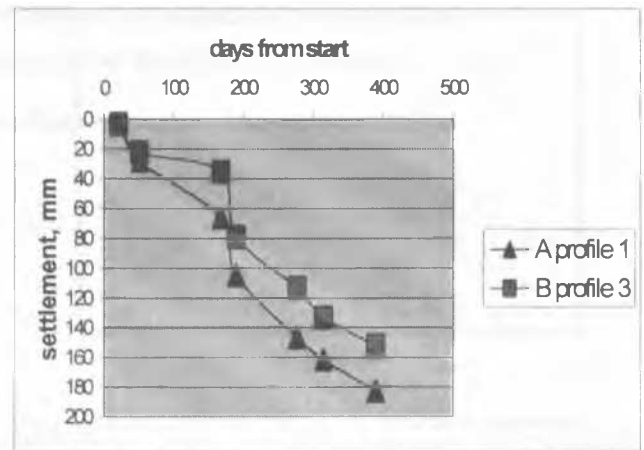


Fig. 3. Total settlement measured at the end points of the profiles 1 and 3; load was increasing in time

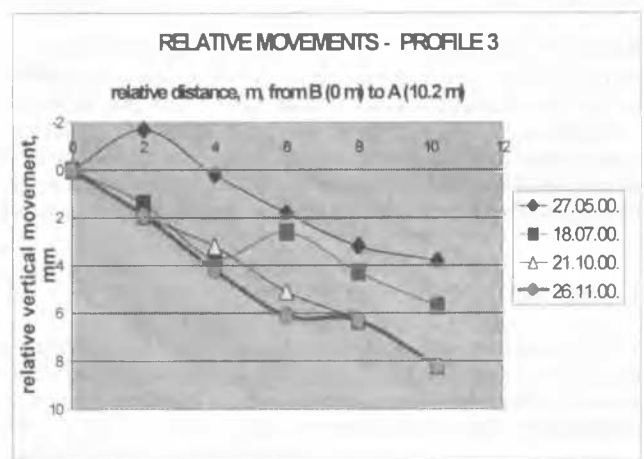


Fig. 4. Relative movements of points on the profile 3

total settlement, but not in Fig. 4 showing differential settlements).

Generally, total settlements reached about 15-18 cm by now. Differential settlements are different for different profiles but distortion is not greater than 1:1000, as requested.

## 4 ANALYSIS OF THE SETTLEMENTS

### 4.1 Soil characteristics

Design solution for the stone columns was based on soil characteristics presented in the Geotechnical Report for Site Characterization (Prizma, 1998) and on design approach defined by Priebe (1992).

Soil characteristics were mainly established by penetration testing. Testing procedures and interpretation of the results were in accordance with Larsson *et al.* (1991, 1995) for piezocone testing and Marchetti (1992, 1997) for dilatometer testing.

Also, a type dynamic penetration was performed in order to penetrate into gravel, with developed local experience in terms of correlation of velocity of penetration and  $N_{(SPT)}$  value for gravels. Classical geotechnical borehole was made in order to take and test undisturbed samples of soil in the laboratory. Two undisturbed samples taken from depths of about 3 m and 7 m were tested in oedometer test in laboratory. Oedometer moduli were in good agreement with the values interpreted from Marchetti dilatometer test conducted at these two depths. Addi-

tional information about soil mechanical properties was obtained by SPT test conducted in the borehole at 2 m depth intervals.

#### 4.2 Soil improvement

Few months after installation of the stone columns piezocone, dilatometer and dynamic penetration testing was performed at two positions between columns in order to determine if there was any improvement in the clay mechanical characteristics. All the parameters were almost the same as in natural deposit. Only resistance to dynamic penetration was on increase.

Soil improvement was checked by SASW method - Spectral Analysis of Surface Waves (Stockoe *at al.*, 1994). Ratio of the shear velocity in improved (profile ran over the stone columns and soil in-between) and nonimproved soil, as an average value for the first 13 m depth, was 3-4 (these measurements are taken at very small soil deformations).

#### 4.3 Settlement analysis

As shown in Fig.3, total settlements have reached value of about 16 cm for 90% of loading, which is about 2.5 times higher than calculated values for improved soil. Design solution calculation shows that maximum settlement would be about 6.5 cm and about 5 cm for building 1 and building 2, respectively. Pulko (2000) and investigation at Georgia Tech (Design guidelines ..., 1983) showed that Priebe method (Priebe, 1992) tends to somewhat overestimate improvement compared to other theoretically more founded solutions, and concentration factor is even more overpredicted.

Separate analyses of the settlement of the building on natural soil indicate the total settlement of 25 cm. It was noticed that the soil moduli for vertical deformation in Keller design solution (1999) were taken as double values of what dilatometer indicated for the depth interval 7-13 m. Also, the same solution predicted that the soil under the columns will settle for a few millimeters only. A separate calculation found that this settlement would be about 4 cm.

The improvement factor established by the design solution was about 2.6. Design guidelines for the stone columns (Georgia Tech, 1983) reports on experience gained by contractors, according to which the settlements are typically reduced by a factor of two. The same document analyses criteria for column bearing capacity in regard to undrained shear strength of soil, angle of internal friction for stone column and possible bulging (near the surface and deeper down in a soft layer thicker than column diameter). Checks on the existing columns for these criteria determined that factor of safety in terms of bearing capacity was close to one for the most loaded piles.

Also, the factor of safety was much dependent on stress concentration factor,  $n$  ( $n$  was taken as 3 and 5).

If the deformation of the soil under the columns is disregarded because of the short time for consolidation, and improvement factor of two in the improved soil zone assumed, the measured settlements are still twice the expected. That raises the question of possible local bulging failure occurring in layers with low undrained shear strength. The total load on a column in a zone loaded in average with  $177 \text{ kN/m}^2$  is about 424 kN, which is a rather high value. Decrease in column diameter, which can happen at some depths, increases stress concentration factor and lowers bearing capacity of a column. Friction along the column, taking half of average undrained shear strength up to 13 m depth ( $0.5 \times 35 = 17.5 \text{ kPa}$ ), is not sufficient to bear load of 424 kN without end bearing and with a reasonable factor of safety. That means that "floating" columns should have a very good bottom contact with soil.

Other possible contribution to large settlements would be the fact that the basic soil stiffness has been overestimated. That means that soil moduli determined by dilatometer (and confirmed at some points by oedometer test) should be about 2 times lower. Marchetti (1997) and Robertson and Schmertmann (1988)

advocate good agreement of settlements computed with dilatometer moduli and measured settlements for many buildings on different soils.

A general conclusion could be that large measured settlements could be a consequence of combined effects related to settlement of improved soil and possible local bulging in columns, at depths where undrained shear strength is low. Consolidation settlement could, in turn, be in increase compared to the calculated settlements due to overestimation of soil moduli. Also, distribution of surface loading with depth is influenced by the presence of firm gravel layer, and combined with the size of loaded area compared to depth of the soft soil it deviates from the classical Bousinesque solution by increasing so calculated values.

Unfortunately, financial limits of the project budget did not allow measurement of the settlement at different depths, which could have helped to explain some of the issues.

## 5 CONCLUSIONS

Settlement of a building on soft soil improved by the stone columns was measured and compared to design calculated values. Main reason for soil improvement was to reduce total and especially differential settlements. Measured total settlements on improved soil were about 2.5 times higher than calculated. Differential settlements were small and remained within distortion limit of 1:1000.

Specially constructed profilometer was used to measure vertical movement under the building inside the profiling tube under the raft. The instrument accuracy is  $\pm 1 \text{ mm}$ .

SASW method proved capable of making a good contrast between small deformation modulus in unimproved and improved soil. Also, dilatometer moduli were regarded as good for standard calculation of settlement, and compared well to oedometer values.

Soil characteristics, possible local bulging failure in columns and possible deviation of standard design from real performance of the soil could be regarded as combined source of discrepancy in measured and calculated settlements.

This case suggests that even an experienced designer and contractor should take care when calculating settlement reduction and soil improvement factor for the stone columns. Also, detailed reliable profile of soil undrained shear strength is required. Trial testing of the stone columns bearing capacity and measurement of soil deformation in depth would contribute to better understanding of the performance of the stone columns.

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