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# Differential Settlement of Foundations on Loess

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# DIFFERENTIAL SETTLEMENT OF FOUNDATIONS ON LOESS

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ABSTRACT

Experience gained during several decades shows that the loess soil in some cases undergoes structural collapse and subsidence due to inundation and that in some other cases the sensitivity of loess to the collapse is considerably less pronounced. In this paper the behaviour of three 12 story buildings A, B and C, of the same static system and the identical shapes have been analyzed. The measurement of settlements of building A over a period of 10 years indicate that the values were situated between the limits 9 cm to 13 cm, and that they are larger than the calculated values, but there was no damage reported in this case. However, the measured settlements of building B over the same period of time were considerably larger, reaching 46 to 51 cm, causing severe damages of the building. In order to find the explanation for such behaviour of loess subsoil, the additional field and laboratory testing of loess have been carried out. Some of the obtained results are presented in this paper.

## INTRODUCTION

Several years ago 20 residential buildings were built on the location near Belgrade (Serbia). All of them were statically identical and 12 stories high, with the depth of foundation D = 2.0 m, with the width of strip foundations B = 1.80 – 2.40 m and with the contact pressure $\sigma$ = 170 kN/m$^2$. By field and laboratory investigation it was found that the soil profile on each microlocation was composed of loose, open structured macroporous landloess, and that its thickness was 20 m or more.

On the basis of the laboratory investigations of soil properties the firm responsible for the project found that the calculated settlement, varying between 6 and 10 cm, could be accepted for all buildings. However, the measured settlements over the period of more than 10 years, were considerably greater than the calculated values, reaching in some cases 50 – 55 cm. The differential settlements were also unexpectedly large, producing severe damages. It is also important to notice that the settlement of some buildings was slightly greater than calculated, and that in such cases the damage was not reported.

Difficulties with building on loess and the results of numerous studies indicate that this soil is of an unusual kind (Moretto and Bolognesi, 1957; Larionov, 1965; Milovic, 1969; Northey, 1969; Milovic, 1978; Lutenegger et al., 1979; Milovic, 1980). In order to better understand the behaviour of loess subsoil under the applied load, in the frame of one Research Project dealing with the foundation problems on loess soils, the enlarged program of investigations was realized.

## LABORATORY TEST RESULTS

### Unconfined compression test results

Experience gained in the past years confirm that the method of sampling has a very significant influence on the quality of loess samples and of sensitive clays samples. The mechanical disturbance of loess samples taken by thin walled piston was registered (Milovic, 1971). For that reason a comparative laboratory testing was made on piston samples and on samples obtained from hand carved blocks, removed from test pits. Typical unconfined compression test results are shown in Figs. 1-5. The curve in Fig. 1 shows the relationship between the vertical stress $\sigma_1$ and vertical deformation $\varepsilon_1$, for block and piston samples.

The results presented in Fig. 1 were obtained on block samples with $\gamma_d$ = 12.9 kN/m$^3$ and water content w = 12 %. The unconfined compression strength was $q_u$ = 27 kN/m$^2$ and modulus of deformation E = 2.5 MN/m$^2$. From the same depth piston samples had $\gamma_d$ = 15.1 kN/m$^3$, w = 13 %, $q_u$ = 74 kN/m$^2$ and E = 4.1 MN/m$^2$.

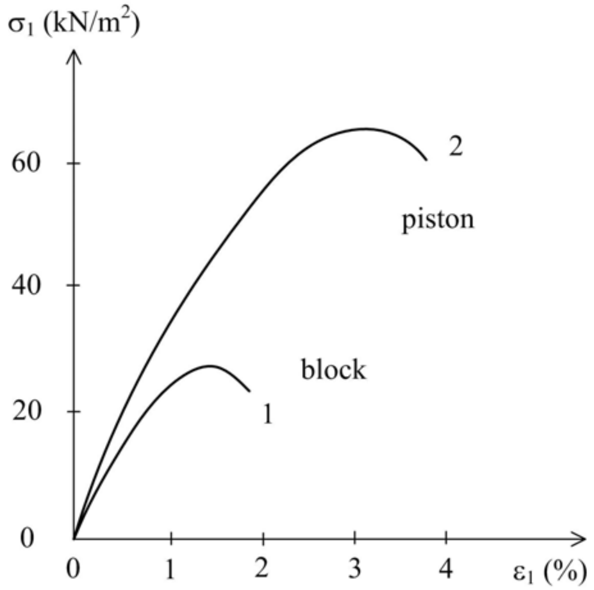


*Fig. 1. Unconfined compression test results*

Figure 2 shows the results on block and piston samples with considerably greater dry density.

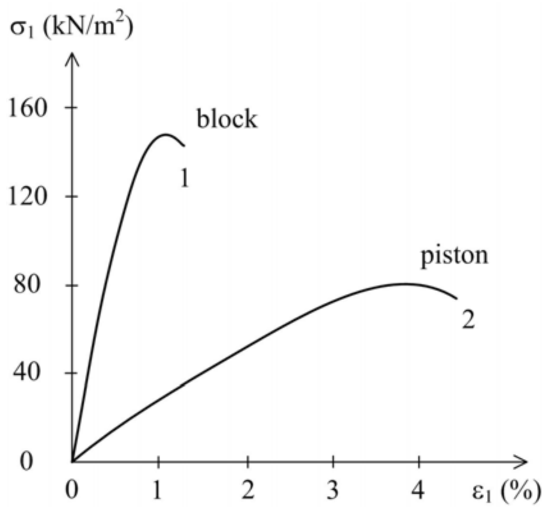


*Fig. 2. Unconfined compression test results*

Figure 2 shows the results obtained on block samples with $\gamma_d$ = 17.3 kN/m$^3$, w = 20.6 %, $q_u$ = 150 kN/m$^2$ and E = 13.3 MN/m$^2$. From the same depth piston samples had $\gamma_d$ = 16.5 kN/m$^3$, w = 23 %, $q_u$ = 86 kN/m$^2$ and E = 2.3 MN/m$^2$. The obtained results clearly show that the method of sampling is of primary importance in determining geotechnical properties of loess soils.

Several groups of block loess samples were tested with lateral strain measurements in unconfined compression tests. Bishop's strain indicator was used to register the stresses which initiated the lateral deformations. Bishop's ring and loess sample are shown in Fig. 3.

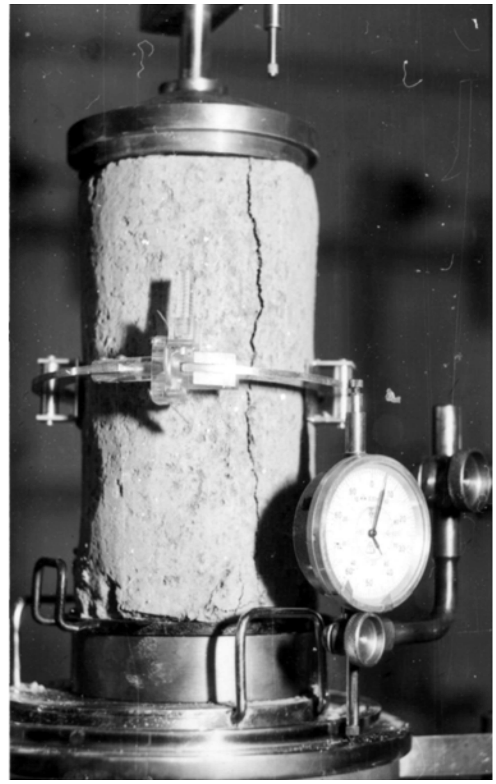

*Fig. 3. Bishop's strain indicator*

The results of these tests are shown in Figs. 4 and 5. In Fig. 4 the values of the vertical stresses which initiate horizontal deformations are shown by points on curves. The samples were with $\gamma_d$ = 12.0 – 12.5 kN/m$^3$ and w = 9 – 24.5 %.

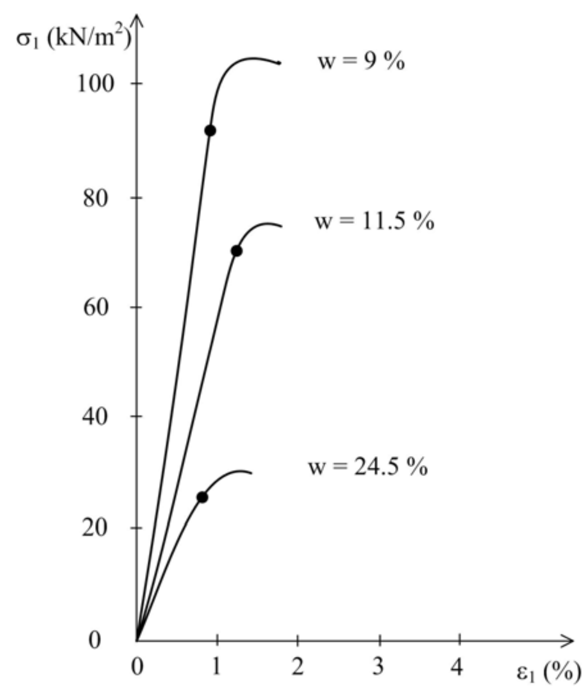


*Fig. 4. Stresses which initiated the lateral deformations*

The curves shown in Fig. 4 indicate how the value of the unconfined compression strength depends on the water content, when the value of the density is the same. In Fig. 5 are shown the results for block samples with $\gamma_d$ = 15.1 – 16.2 kN/m$^3$, with the same water content w = 22.5 %.

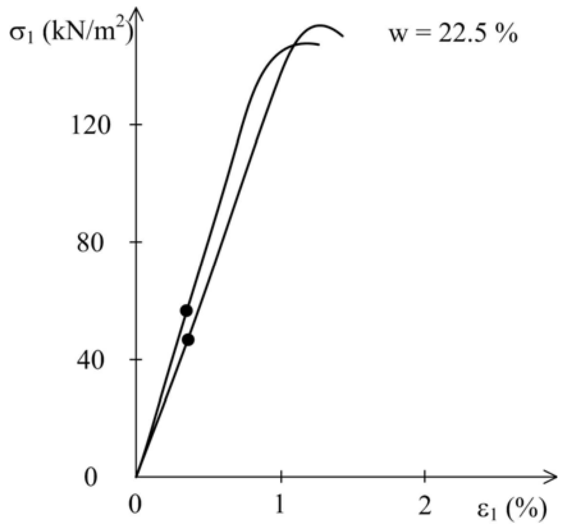


*Fig. 5. Stresses which initiated the lateral deformations*

The obtained results show that for the samples with the relatively high dry density the lateral deformations started at low vertical pressures. However, for the samples of low density the lateral deformations were registered under the stresses which were close to the ultimate stress $q_u$.

A number of unconfined tests were performed on block loess samples, cut in vertical and in horizontal direction, in order to determine the degree of anisotropy. Typical results are shown in Fig. 6.

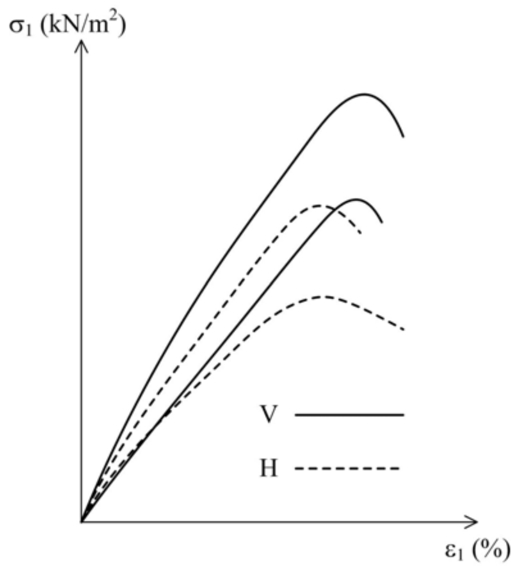


*Fig. 6. Unconfined compression test results for block samples*

Full lines indicate the curves obtained on vertical samples (V) and dash dot lines the curves on horizontal samples (H). The tested samples were with $\gamma_d$ = 16.5 – 17.5 kN/m$^3$ and w = 21.2 ± 1 %, $E_V$ = 16 ± 5 MN/m$^2$ and $E_H$ = 13 ± 1 MN/m$^2$. The degree of transverse anisotropy n ranged between 1.30 and 1.60.

Consolidation test results

In order to indicate the influence of the initial dry density of loess samples on the values of modulus of compressibility $E_{oe}$, a relatively great number of the consolidation tests was carried out by one dimensional compression of the loess samples, which were laterally restrained. Undisturbed samples, cut from blocks, were saturated at very beginning of the testing. The values of the modulus of compressibility were determined for the range of stresses $\sigma$ = 80 – 120 kN/m$^2$ on loess samples with $\gamma_d$ = 12 – 16.5 kN/m$^3$.

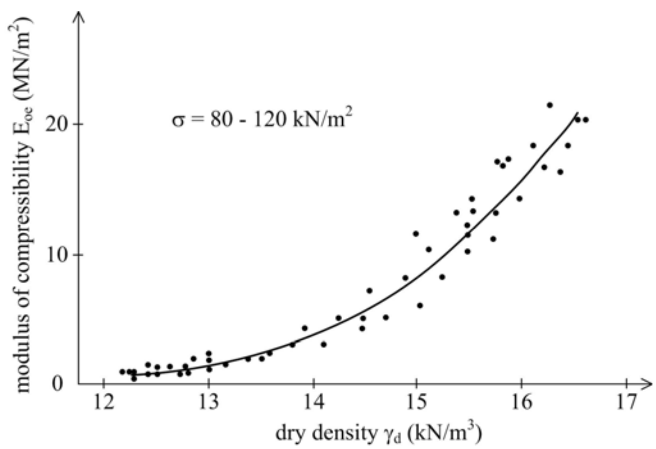


*Fig. 7. Modulus of compressibility $E_{oe}$ for saturated loess samples*

It is of interest to notice that for the values of dry density greater than ~ 14 kN/m$^3$ the values of the modulus of compressibility remarkably increase.

The change of the void ratio and the degree of subsidence were also determined by one dimensional compression tests and then the coefficient of subsidence was obtained by the following expression:

$$i_m = \frac{e_n - e`_n}{1 + e_n} = \frac{\Delta e_n}{1 + e_n} \tag{1}$$

where $e_n$ is the void ratio before saturation at the vertical stress $\sigma_n$, and $e_n'$ is the void ratio at the end of subsidence under the same vertical stress $\sigma_n$, as shown in Fig. 8.

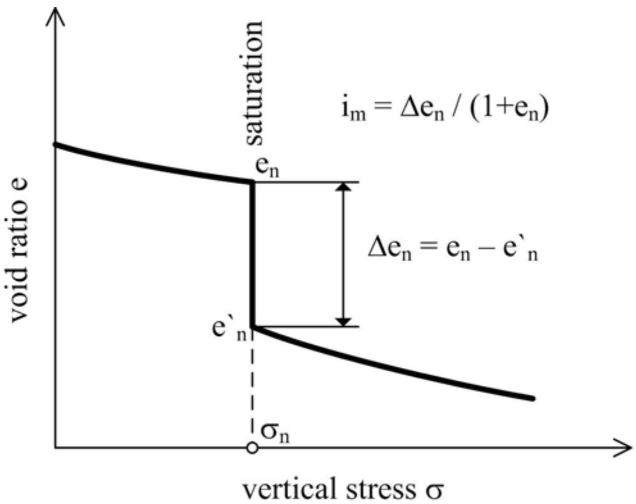


*Fig. 8. Typical subsidence consolidation test result*

Coefficient of subsidence $i_m$ can be used for the prediction of the additional settlement of loess soil due to an increase of water content. Therefore, for the relatively large number of undisturbed less samples the standard test procedure for evaluation of the hydroconsolidation potential was used. A sample was loaded to some value of $\sigma_n$ and after the consolidation of the specimen was reached, the water was added. These tests have been performed on loess samples, covering the large range of dry density and initial water content. In Figs. 9 - 13 are presented the curves of the coefficients of subsidence $i_m$ for the undisturbed loess samples with various values of dry density, covering the range from 12.5 to 15.5 kN/m$^3$. Curve 1 in Fig. 9 is related to the samples with $\gamma_d$ = 12.5 – 13 .0 kN/m$^3$ and w = 13.2 %. Curve 2 indicates the values of $i_m$ for the samples which were saturated at a given stress level.

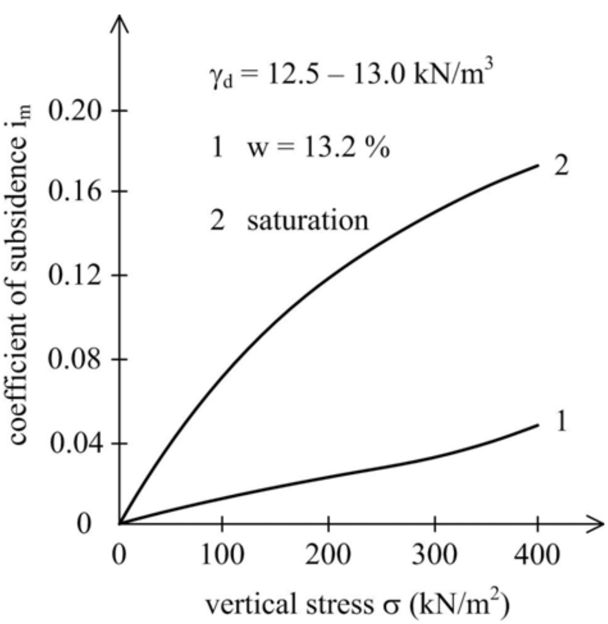


*Fig. 9. Coefficients $i_m$ for $\gamma_d$ = 12.5 - 13.0 kN/m$^3$*

In Fig. 10 are presented curves of the coefficients $i_m$ for the samples with $\gamma_d$ = 13.5 - 14.0 kN/m$^3$. Curve 1 is related to the samples with w = 13 % and curve 2 indicates the values of $i_m$ for the saturated loess samples at a given stress level.

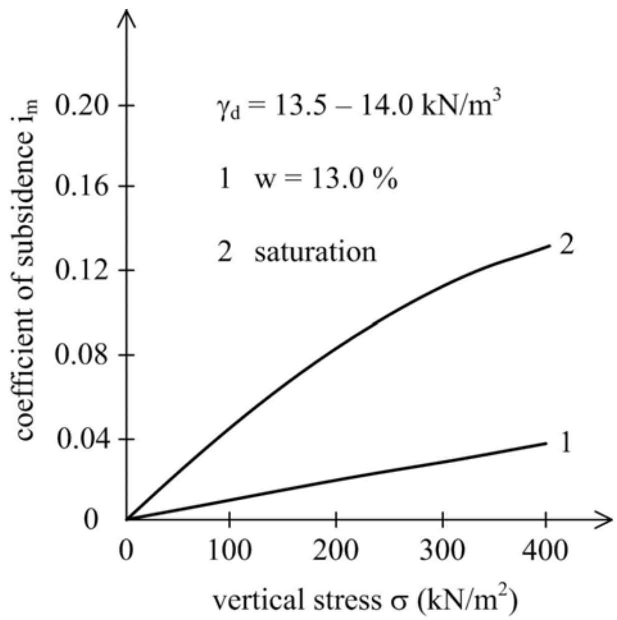


*Fig. 10. Coefficients $i_m$ for $\gamma_d$ = 13.5 - 14.0 kN/m$^3$*

In Figs. 11 - 13 are shown the curves of $i_m$ for the loess samples with $\gamma_d$ = 14.0 - 14.5 kN/m$^3$, $\gamma_d$ = 14.5 - 15.0 kN/m$^3$ and $\gamma_d$ = 15.0 - 15.5 kN/m$^3$, respectively. The water content before saturation was w = 12.6%, w = 13.2% and w = 13.8%.

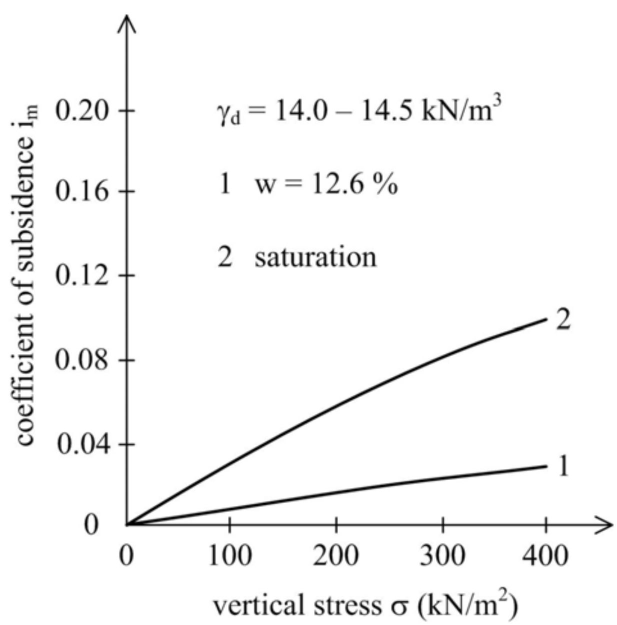


*Fig. 11. Coefficients $i_m$ for $\gamma_d$ = 14.0 - 14.5 kN/m$^3$*

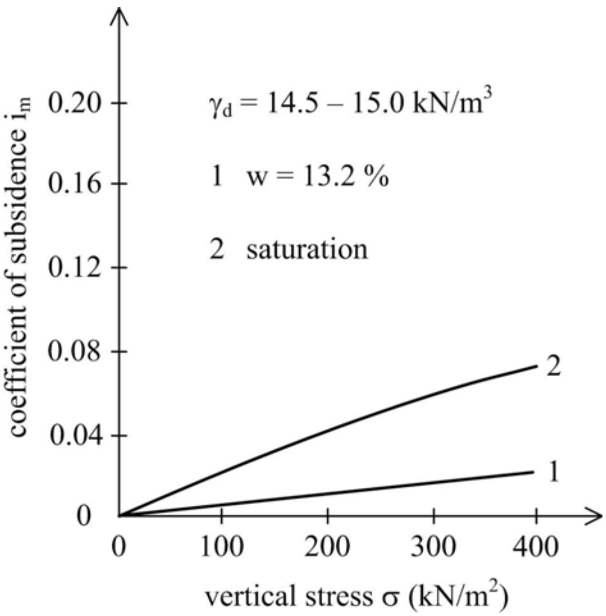


*Fig. 12. Coefficients $i_m$ for $\gamma_d$ = 14.5 - 15.0 kN/m$^3$*

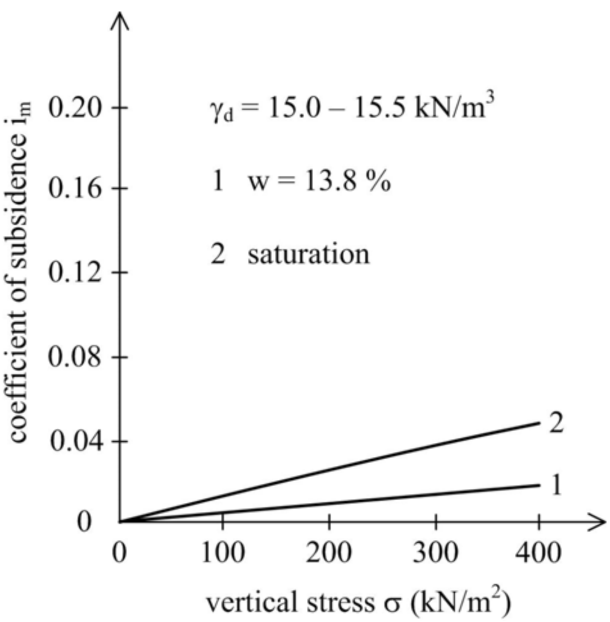


*Fig. 13. Coefficients $i_m$ for $\gamma_d$ = 15.0 - 15.5 kN/m$^3$*

The obtained results clearly show that the coefficient of subsidence $i_m$ depends to a large extend on the initial value of $\gamma_d$, on the natural water content and on the applied load $\sigma$ during the saturation. Beside of this, it is also shown that the structural collapse due to saturation is well pronounced on the samples with low dry density and low initial water content. However, the samples with low dry density and high water content exhibit large settlements before saturation. On the other hand, the samples with high dry density, approximately higher than 15 kN/m$^3$, undergo relatively small settlements due to saturation.

FIELD TEST RESULTS

Static penetration tests in the field were carried out in loess deposits with natural water content and in saturated loess. Some typical results are shown in Figs. 14 and 15.

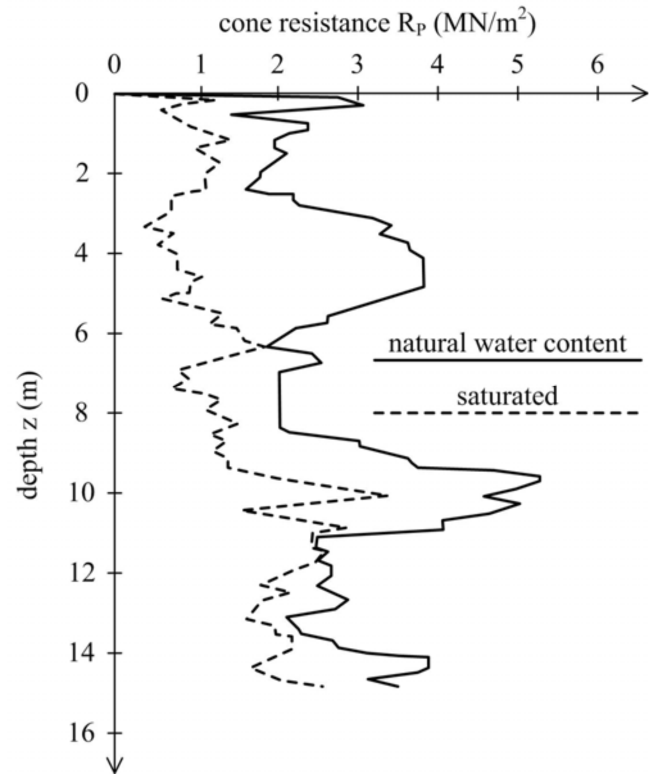


*Fig. 14. Cone penetration test results*

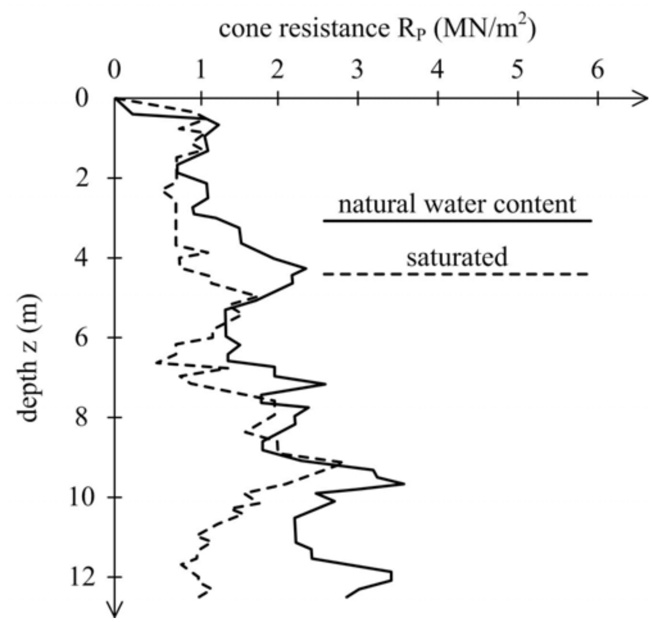


*Fig. 15. Cone penetration test results*

Full line in Fig. 14 shows the cone resistance of the loess deposit with natural water content, and dotted line represents the cone resistance of the saturated loess soil. In Fig. 15 similar results of the static penetration tests are presented. From several test pits the undisturbed block samples of loess were obtained and laboratory tests were carried out. The dry density of these samples was $\gamma_d$ = 12.2 - 13.5 kN/m$^3$ natural water content w = 15 – 21 % and modulus of compressibility $E_{oe}$ = 2 – 2.5 MN/m$^2$. However, after saturation these values were $E_{oe}$ = 1.3 – 1.5 MN/m$^2$.

The obtained results of the penetration tests illustrate the effect of water content on the stress – deformation relationship and confirm the validity of the consolidation test results. On several locations field load tests were performed in order to establish the relation between stresses and displacements. Typical results are presented in Fig. 16.

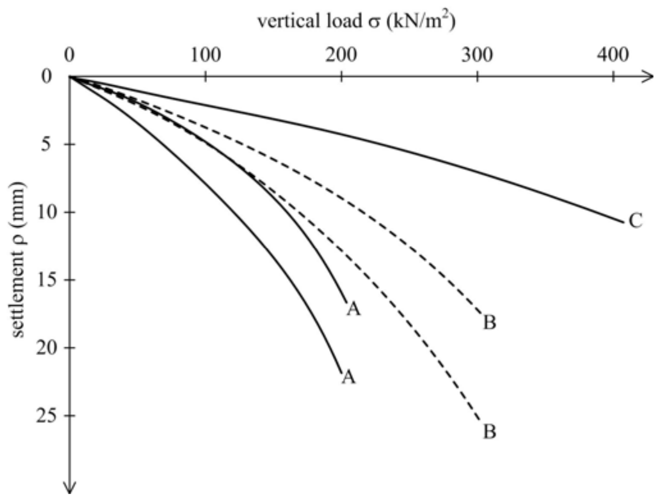


*Fig. 16. Field load test results*

Curves A are related to the loess deposits with $\gamma_d$ = 12.6 – 13.1 kN/m$^3$ and natural water content w = 20.5 – 22 %. Curves B were obtained for loess deposits with $\gamma_d$ = 13 – 14.7 kN/m$^3$ and natural water content w = 19.1 – 24.7 %. Curve C indicates load – settlement relationship for loess with $\gamma_d$ = 16 – 17.2 kN/m$^3$ and water content w = 19 – 20.7 %. One may say that in all cases water content is practically almost the same, whereas the initial dry density varies between the very low and very high values. The presented curves indicate the influence of the initial dry density on the amount of subsidence, what is in a good agreement with the results shown herein.

## THEORETICAL STUDY

The problem of calculating the stresses and displacements was studied by the finite element method and by Fourier's series. In the theoretical consideration it was assumed that the strip foundation was perfectly rigid, perfectly flexible, including also the foundation of any rigidity. It was also assumed that the soil profile was the elastic and isotropic or, alternatively, anisotropic half space. In some cases the compressible layer of limited thickness was also considered. Geometry of the problem is shown in Fig. 17.

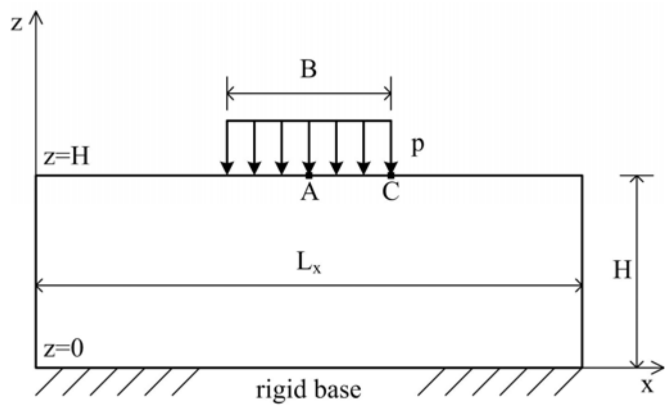


*Fig. 17. Geometry of the problem*

Relative rigidity K of the foundation is given by:

$$K = \frac{(1-\mu_s)^2 \cdot E_b \cdot h^3}{6 \cdot (1-\mu_b)^2 \cdot E_s \cdot B^3} \tag{2}$$

where: $E_b$ = modulus of elasticity of concrete, $E_s$ = modulus of elasticity of soil, $\mu_b$ = Poisson's ratio of concrete, $\mu_s$ = Poisson's ratio of soil, h = thickness of strip foundation and B = width of foundation.

In the finite element analysis, in the case of the elastic and isotropic soil mass, the stiffness matrix is given by:

$$D_e = \frac{E}{(1+\mu)(1-2\mu)} \begin{bmatrix} 1-\mu & \mu & 0 \\ \mu & 1-\mu & 0 \\ 0 & 0 & \frac{1-2\mu}{2} \end{bmatrix} \tag{3}$$

and in the case of an anisotropic soil the stiffness matrix is given by the following expression:

$$D_e = \frac{E_v}{(1+\mu_h)\cdot(1-\mu_h-2k\mu_v^2)} \cdot \begin{bmatrix} k(1-k\mu_v^2) & k\mu_v(1+\mu_h) & 0 \\ k\mu_v(1+\mu_h) & 1-\mu_h^2 & 0 \\ 0 & 0 & m(1+\mu_h)(1-\mu_h-2k\mu_v^2) \end{bmatrix} \tag{4}$$

where: $E_v$ = modulus of elasticity in the vertical direction, $\mu_v$ = Poisson's ratio in the vertical direction, $E_h$ = modulus of elasticity in the horizontal direction, $\mu_h$ = Poisson's ratio in the horizontal direction, $k = E_h/E_v$, $m = G_v/E_v$ and $G_v$ = shear modulus in the vertical direction.

The dimensionless coefficients I for the calculation of componental stresses and displacements have been determined with various values of Poisson's ratio, with several values of foundation rigidity and for two degrees of anisotropy. Some of the obtained results are presented graphically. More results can be found elsewhere (Milovic and Djogo, 2005; Milovic, 1992; Milovic and Djogo, 2005). In Fig. 18 are shown the coefficient $I_w$ for several values of the ratio H/B, where H is the thickness of the compressible layer and B is the width of the flexible strip foundation.

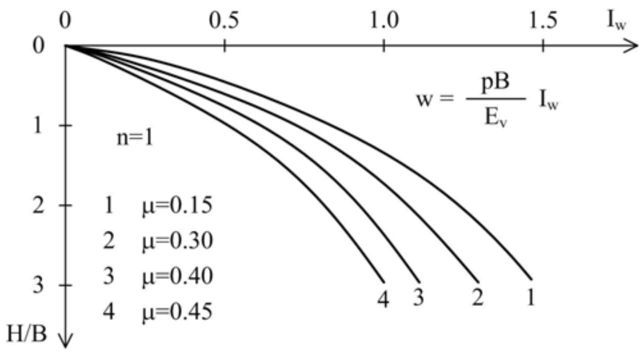


*Fig. 18. Coefficients $I_w$; n = 1*

In this case it was assumed that the soil was elastic and isotropic, with $n = E_v/E_h = 1$. In Figs. 19 and 20 are presented the values of coefficient $I_w$ for the anisotropic soil with n = 0.50 and n = 2.0, respectively.

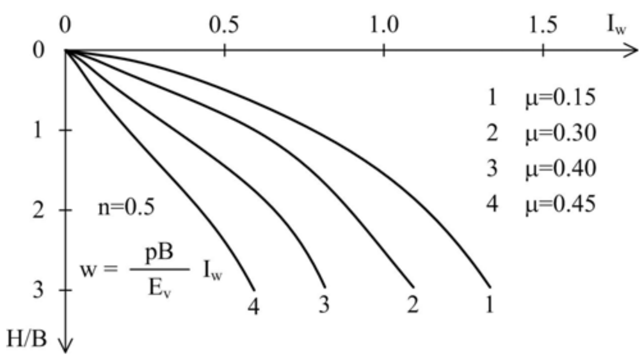


*Fig. 19. Coefficients $I_w$; n = 0.5*

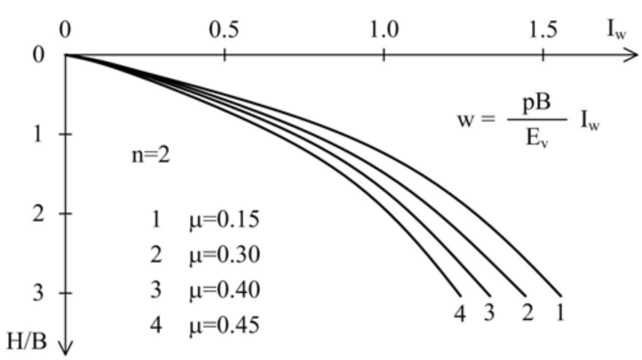


*Fig. 20. Coefficients $I_w$; n = 2*

Fig. 21 shows the coefficients $I_w$ for isotropic and anisotropic soil with the Poisson's ratio μ = 0.30.

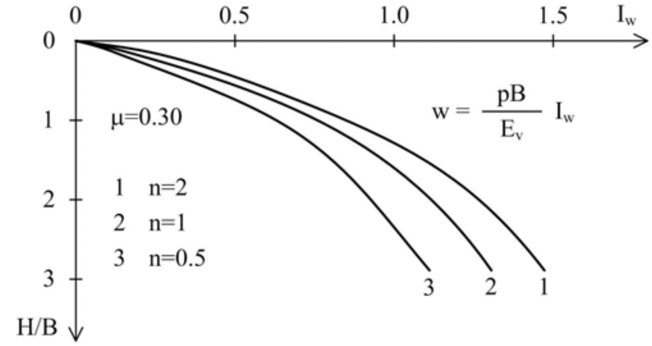


*Fig. 21. Coefficients $I_w$; μ = 0.30*

The coefficients $I_w$, determined for various values of rigidity K, are shown in Fig. 22.

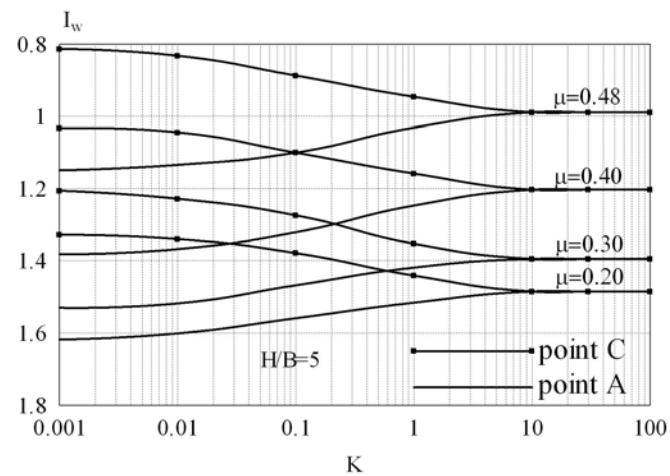


*Fig. 22. Coefficients $I_w$; strip foundation for various valuse of rigidity K*

Using the presented curves it is possible to determine the differential settlements of the strip foundation of any rigidity K.

The problem related to the determination of stresses and displacements in the elastic and isotropic soil of finite thickness has also been studied by Fourier's series. Componental displacements u and w, produced by a vertical uniform load over a flexible strip, are expressed by the following trigonometric series:

$$u = \sum_{m=0} U(z,m) \cdot \sin(\alpha \cdot x) \quad (5)$$

$$w = \sum_{m=0} W(z,m) \cdot \cos(\alpha \cdot x) \quad (6)$$

$$\alpha = \frac{m \cdot \pi}{L_x} \tag{7}$$

By developing these displacements functions in the Fourier's series the solution for componental stresses and displacements has been obtained. Using these solutions dimensionless coefficients have been determined for several values of the ratio H / B and for four values of the Poisson's ratio. Some of the obtained results are shown in Figs. 23 and 24.

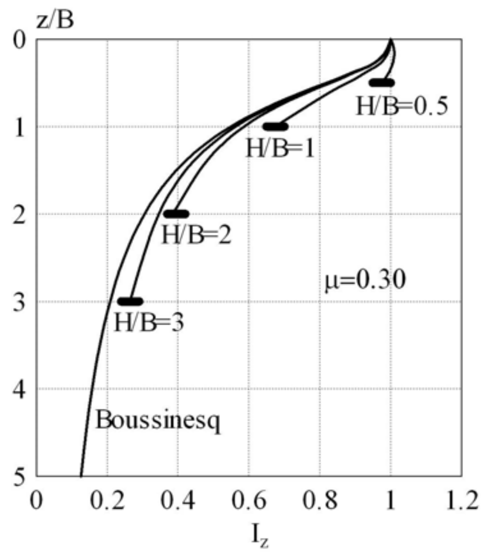


*Fig. 23. Coefficients $I_z$ for flexible strip foundation*

As can be seen, the vertical stresses $\sigma_z$ are bigger than those obtained by Boussinesq's half – space. In Fig. 24 are presented the values of the coefficient $I_w$ for several values of H/B and for three values of Poisson's ratio μ.

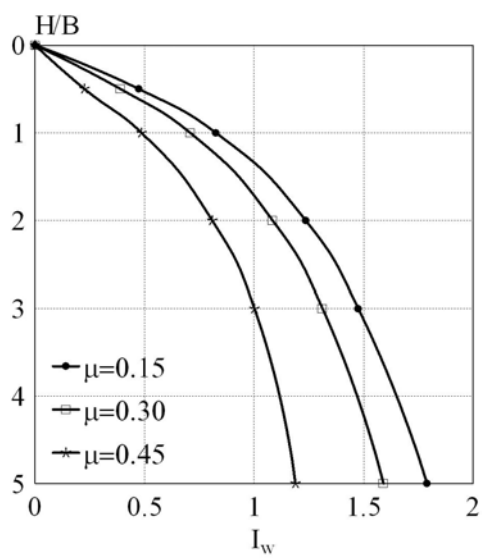


*Fig. 24. Coefficients $I_w$ for flexible strip foundation*

It is of interest to emphasize that the results obtained for stresses and displacements by finite element method and by Fourier's series are in a very good agreement.

## PREDICTED AND MEASURED SETTLEMENTS

The measured settlements of twenty 12 story buildings over a relatively long period of time show that in some cases there is no acceptable agreement between the calculated and observed settlements. On the basis of the available results of measurements it was concluded that for five buildings the measured settlements were considerably larger than the predicted values. However, in other cases the calculated settlements were in reasonable agreement with the measured values.

On the location of each building several boreholes were performed, reaching the depth of approximately 20 m, and the undisturbed samples of loess were taken from various depth with thinwall piston.

The laboratory test results have shown that the soil profile was uniform and that the foundation subsoil consisted of approximately 20 m macroporous landloess underlain by stiff clay. Dry density $\gamma_d$ of piston samples varied from 15.8 to 16.6 kN/m$^3$ and natural water content w from 13.2 % to 17 %. Using these data it was obtained that the calculated settlements were $\rho$ = 7 – 10 cm. On the basis of these acceptable values the firm responsible for the constructions assumed the same system on strip foundations for all buildings.

The Building A was performed without serious problems, and the settlements stayed in the frame of the calculated values. However, it is important to note that the excavation for foundation pit was not additionally wetted. Tanks to the unchanged water content in the foundation subsoil, the values of the differential settlements were acceptable, and in this case there was no reported damage. In Fig. 25 are presented the values of the measured settlements for each corner of the building.

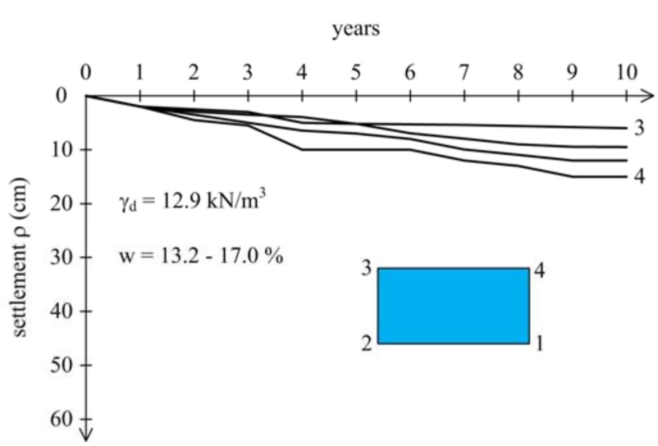


*Fig. 25. Time – settlement curves; Building A*

As can be seen, the values of the measured settlements are slightly bigger than the calculated values, because they were determined with the higher values of $\gamma_d$, obtained on piston samples.

Taking into account that the piston samples of loess were mechanically disturbed (Milovic, 1971), in the vicinity of the Building B several pits were excavated and the undisturbed loess samples, cut from hand curved blocks, were obtained. The values of dry density were $\gamma_d$ = 11.8 – 12.6 kN/m$^3$ and water content w = 14.8 – 16.5 %. It is worth noting that these values are considerably lower than $\gamma_d \sim 16$ kN/m$^2$, obtained for piston samples. The observed settlements of Building B show that the total settlement and particularly the differential settlements were excessively large. In Fig. 26 are shown the values of the measured settlements.

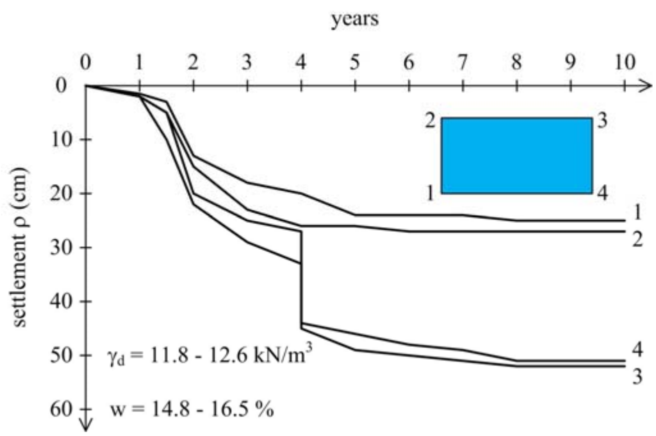


*Fig. 26. Time – settlement curves; Building B*

As can be seen, in the first period of exploitation the settlements of the benchmarks 3 and 4 were larger than those with benchmarks 1 and 2. The amount of settlements was 20 to 30 cm, with the differential settlement of about 10 cm. On the basis of the available information and technical documents it was concluded that for some buildings the excavated foundation pit was open during the rainfall and the water penetrated in the loess subsoil and drastically changed the natural water content. The results of the additional cone penetration tests, performed in loess soil with natural water content and in the saturated loess are shown in Fig. 27.

The canalization pipes and water piping system were damaged by the differential settlements, provoking the additional infiltration of water into the loess subsoil. This change of water content caused a considerable increase of settlements in the area of benchmarks 3 and 4 up to 52 cm, and also the increase of differential settlements up to the value of approximately 27 cm. These settlements caused severe damages of building B. The reparation of walls, stairs, elevators and other damaged parts of building required a considerable amount of time and money.

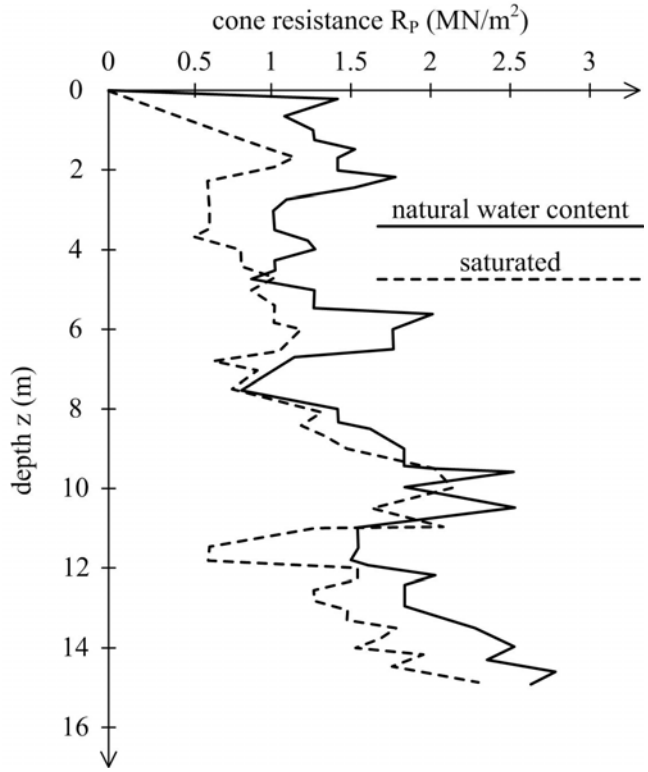


*Fig. 27. Cone penetration test results*

The Building C, founded on loess subsoil, represents one another type of behaviour, quiet different from the buildings A and B. In this case the measured settlements are much bigger than the predicted values. In Fig. 28 are shown the values of these settlements.

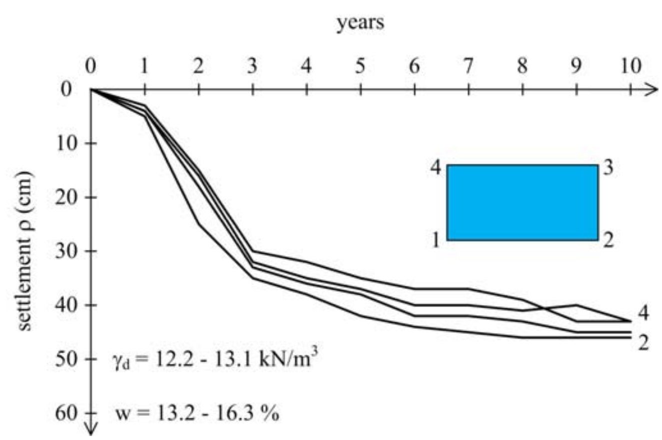


*Fig. 28. Time – settlement curves; Building C*

The excavation of the foundation pit probably was uniformly wetted by intensive rainfall and the settlements were very large and reaching 41 to 46 cm. However, the differential settlements remained between the acceptable limits, which did not cause severe damages.

## CONCLUSIONS

Loess deposits are very sensitive to the mechanical disturbance, and the inadequate method of sampling can lead to wrong laboratory results, which are often on the unsafe side.

The sensitivity of loess to subsidence due to wetting or saturation depends to a large extent on the initial dry density, initial water content and stress level during saturation.

The prediction of the behaviour of structures founded on loess deposits must be based on the settlement calculation in natural moisture conditions and also in wetted or saturated state.

In settlement calculation the anisotropic properties of loess must be taken into account.

The selection of design geotechnical parameters is an essential component of design.

This case study emphasized once more that the method of calculation may play less significant role in settlement prediction than the selection of the geotechnical parameters.

The use of shallow foundations as well for small or high buildings founded on loess is always risky because, sooner or later, water can penetrate into foundation subsoil. In the extreme case saturation can cause inclination or even collapse of the structure, or seriously endanger its serviceability.

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