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Settlement Back-Analysis of Buildings on Soft Soil in Southern Germany

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ABSTRACT

This paper presents 10 case-histories of buildings on soft clay in southern Germany. A lot of field observations show that the calculated settlements using the routine analysis are on average 50 % larger than settlements actually measured in this area. A back-analysis is carried out to verify the soil parameters which are intended to investigate in the subsurface exploration phase and later in a laboratory test program. Recommendations for the engineering practice are suggested to review the determination of compressibility parameters and, consequently, to improve the settlement prediction.

INTRODUCTION

This paper demonstrates 10 observation results of soil consolidation under buildings with raft foundation on soft clays in southern Germany, especially near the lakes in front of the Alps. The observations were performed in the city of Constance and its environs (see Fig. 1). The measurements were carried out far enough to form a reliable basis to study the primary settlement and the secondary settlement to some extent. The main purpose of this paper is to demonstrate the considerable discrepancy between the measured and the calculated settlement as well as to explain it using the back-analysis associated with a laboratory test program, where standard consolidation test (STD) and constant rate of loading consolidation test (CRL) are carefully carried out. The influences of the load increment and the loading rate on the soil deformation behavior have been intensively discussed.

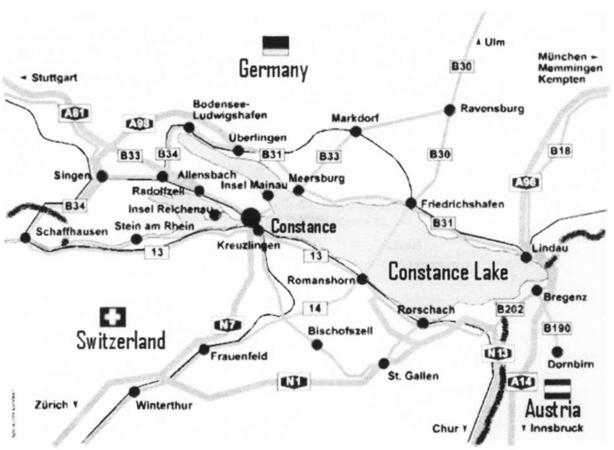

Fig. 1: Map of the Constance Lake

Finally recommendations for the engineering practice are suggested that may lead to better design parameters for settlement estimation in this type of soils.

SOILS AND GEOLOGY

The soils near the lakes in front of the Alps belong to the lacustrine and fluvial young sedimentations from the Holocene and the last ice age, see for example Fig 2.

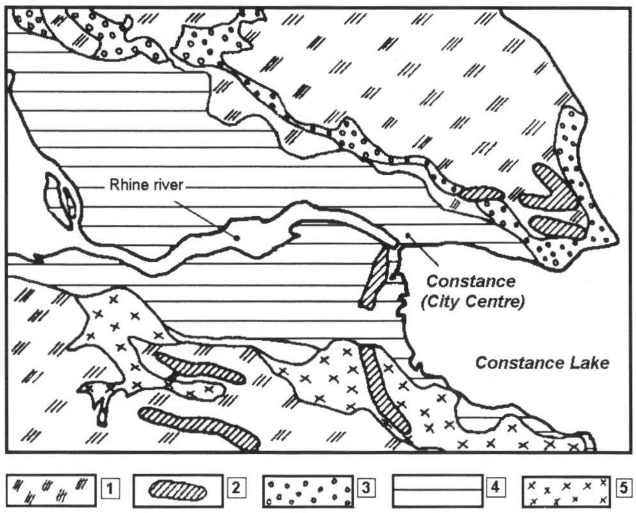

Fig. 2. Constance lacustrine deposits where: 1) lacustrine clay (last ice age), 2) glacial drift (moraine), 3) gravel-sand mixtures, 4) lacustrine clay (Holocene), 5) Loam (Holocene)

In geotechnical terms those sensitive sub soils are silt clays to sandy silts of low plasticity with soft to very soft consistency, water content is up to 25-50 %, drained shear parameters are φ'= 22.5-25° and c' = 0 kN/m², undrained shear parameter is c_u = 10-40 kN/m². Because of their low strength and high compressibility the lacustrine clays in southern Germany are considered as difficult soil in foundation engineering and loads of normal buildings are typically carried by raft foundations.

For the purpose of classification, grain size analysis and Atterberg limits tests of the lacustrine soft clay from several sites were carried out. The results with additional data from *Gebreselassie (2003)* are presented in Fig. 3 and Fig 4.

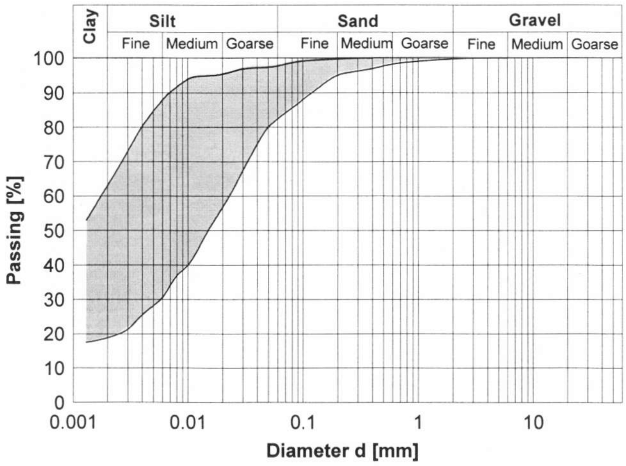

Fig. 3. Range of the grain size distribution of Constance clay

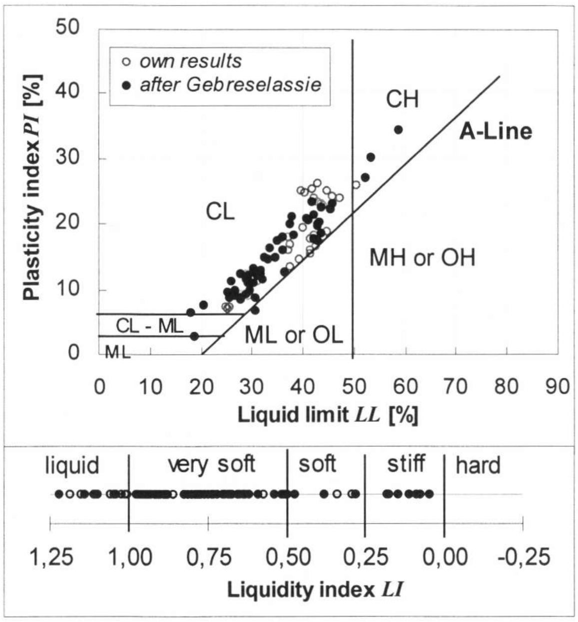

Fig. 4. The consistency of Constance clay

In addition, results of the undrained shear strength measured by vane tests from locations in three towns near the lakes in front of the Alps (*Weissenbach and Kempfert (1995)*) are shown in Fig. 5. In Constance region a ratio of c_u / σ'_{vc} between 0.22 to 0.26 has been determined by *Scherzinger (1991)* that matches well with the equation:

$$c_u / \sigma'_{vc} = 0.23 \pm 0.04 \qquad (1)$$

found out by *Jamiolkowski et al. (1985)* for normally consolidated clays having PI < 60 %.

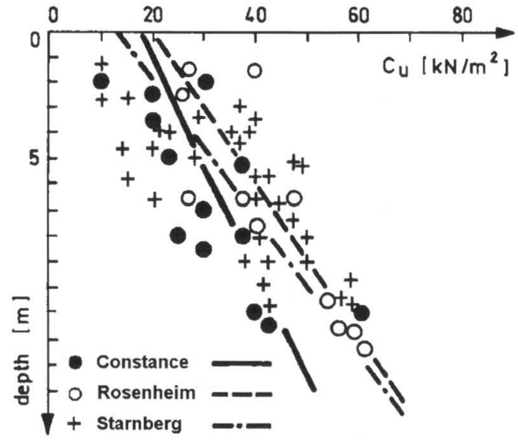

Fig. 5. Undrained shear parameter of the lacustrine clay from locations in three towns near the lakes in front of the Alps

Own compressibility parameters with further results from high quality samples obtained using a special sampling device developed by *Scherzinger (1991)* are shown in Fig 6.

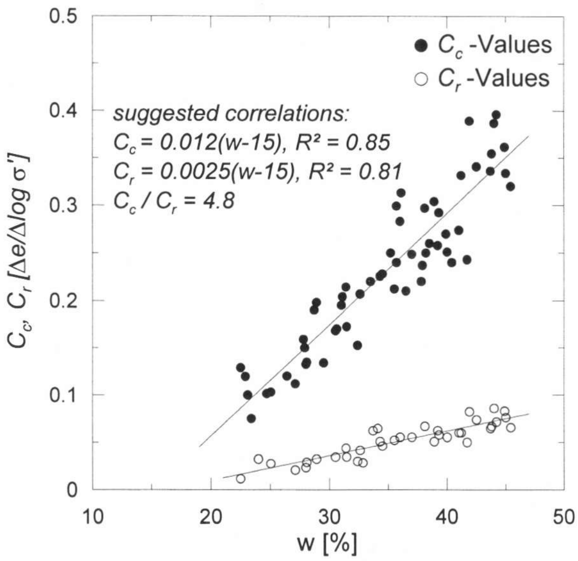

Fig. 6. Compression index C_c and recompression index C_r of the Constance lacustrine clay

The presented subsoil properties are over large regions so uniform that the settlements of buildings from several towns in this region can directly be compared to obtain a general conclusion for the deformation behavior of soft soil in southern Germany.

REVIEW OF SETTLEMENT OBSERVATIONS

For a valuable back-analysis one of the following equations can be used, *Lambe's (1973).*

Observation + Soil Parameter ⇒ Validated Theory (2)

Observation + Theory ⇒ Empirical Soil Parameter (3)

Nevertheless, the conventional theories are widely used in the geotechnical practice to estimate the primary and secondary settlement as well as the rate of consolidation. Therefore, the authors applied the equation (3) using the available long-term settlement measurements and the common practical methods to obtain representative deformation parameters taking into account that the choice of an "appropriate theory" to describe the soil response is one of many unavoidable idealizations in each back-analysis, *Lerouil and Tavenas (1981).*

Back-analysis procedures

The field observations were analyzed using the method developed by *Asaoka (1978)*. This method enables to determine the final settlement s_∞ and the coefficient of consolidation c_v for a settlement-time observation. After evaluating the final settlement for all measurement points of every single project, a settlement isolines map was established to interpolate the final settlement $\bar{s}_\infty$ at the so-called characteristic points in which the settlement is usually calculated in the German practice, see fig 8. This is the way to determine the settlement of a rigid foundation because in exact elastic solutions the settlements in the characteristic points for rigid and flexible foundation are identical, *Graßhoff (1955).*

Using the "measured" c_v-values, the field primary settlement s_p could be estimated. Using the same ratio C_c/C_r obtained in laboratory (see Fig. 6), the measured value s_p can be substituted into the well known equation (4) to estimate an average field compression index C_c.

$$s = s_p = \frac{H}{1+e_0}\left[C_r \cdot \log\frac{\sigma'_{vc}}{\sigma'_0} + C_c \cdot \log\frac{\sigma'_{vc}+\Delta\sigma}{\sigma'_{vc}}\right] \qquad (4)$$

where H is the thickness of the compressible layer, e_0 is the initial void ratio, σ'_0 is the effective initial stress and σ'_{vc} is the overburden pressure which is assumed to be equal to the preconsolidation pressure in normally consolidated deposits. The average increase of stress in the compressible layer due to the applied surface load $\Delta\sigma$ was estimated by *Simpson's* rule.

It should be mentioned here that the measurement of settlement in all projects was performed after constructing the raft foundation on a compacted granular fill. Hence considerable amount of the immediate settlement in addition to the elastic settlement due to the recompression occurred prior to any measurement. Consequently, the largest part of settlement occurs entirely along the compression curve and the error resulting by assuming a value $C_c/C_r = 4.8$ can be neglected.

In addition all measurements beyond the primary consolidation time t_p were used to estimate an average field coefficient of secondary settlement C_α using the equation

$$C_\alpha = \frac{1}{H}\left[(s - s_p)/\log(t/t_p)\right] \qquad (5)$$

in which s is the measured settlement corresponding to time t (where $t > t_p$) and H is the thickness of the compressible layer.

In this way the field primary and secondary settlement as well as the actual rate of deformation can be determined and compared to the parameters from laboratory tests.

Case 1: Students hostel, Constance

The first example of the back-analysis deals with a students dormitory in Constance. The building consists of 9-storey and an underground floor and was built on 36 m thick lacustrine clay layer. As shown in Fig. 7 the water content is closed to the liquid limit.

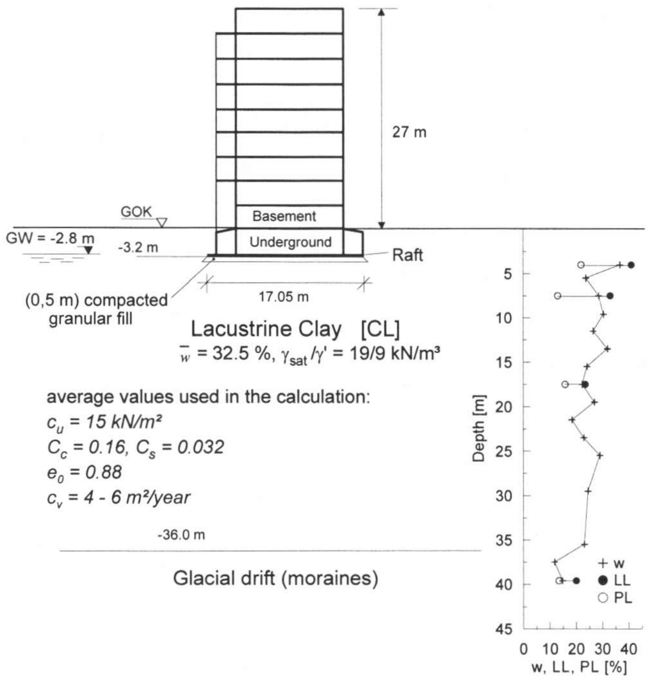

Fig. 7: Profile, soil conditions and cross section, case 1

Settlements were measured at six points for 884 days after the raft foundation was in place. Fig. 8 shows the observed time-settlement curves and the settlement contours.

Using *Asaoka' s* method the settlement at all measurement points was determined so that an average final settlement of about 6.15 cm at the characteristic points could be interpolated. This value is 35 % smaller than the calculated settlement ($s_{cal.}$ = 9.5 cm).

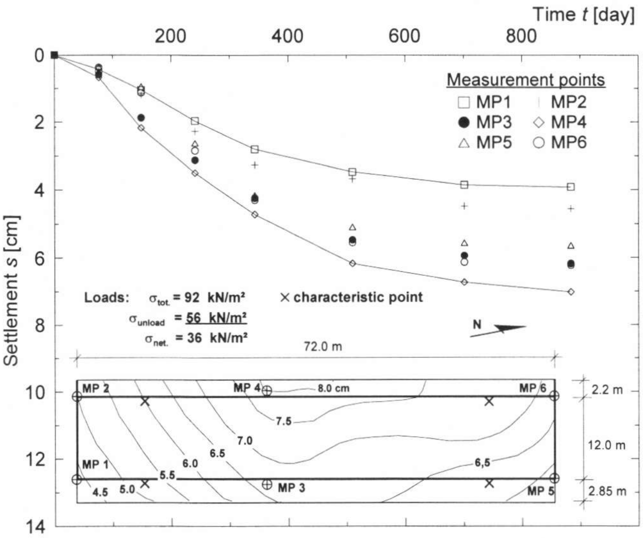

Fig. 8: Settlement record and settlement contours, case 1

Furthermore, a value of 32 m²/year for the coefficient of consolidation was obtained assuming one way drainage and a thickness of 7.4 m for the compressible layer. Substituting the measured settlement (s_p = 6.15 cm) into equation (4) an average field compression index of C_c = 0.104 can be derived. Moreover, using the actual c_v–value a field consolidation time of t_p = 748 days was determined for this project. The measurements subsequent to this time were substituted into equation (5) so that an average field coefficient of secondary settlement C_α = 0.0048 was evaluated.

In the next sub-paragraphs, only the soil conditions and the settlement records of the subsequent case histories will be demonstrated. The results of the back-analysis will be summarized later.

Case 2: An office building with underground garage,

The second case deals with an office building in Constance. The house has an S-form ground plan and consists of 5-storey in the southern part and 4-storey in the north part with a basement and an underground garage. It lies on a 0.5 m thick raft foundation that was built on a well-compacted granular fill with a thickness of 1 m.

The subsoil conditions were explored by borings and soundings to a depth of about 41 m. A lacustrine soft clay layer with a thickness of 31 to 38.5 m below the ground surface was found (see Fig. 9). The clay soil is normally consolidated and has an average water content of 30 % and an undrained shear strength of 20 kN/m² within the compressible layer.

The settlement has been followed in 12 measurement points by regular measurements over a period of 961 days, see Fig 10. The back-analysis was performed applying the same methods explained above.

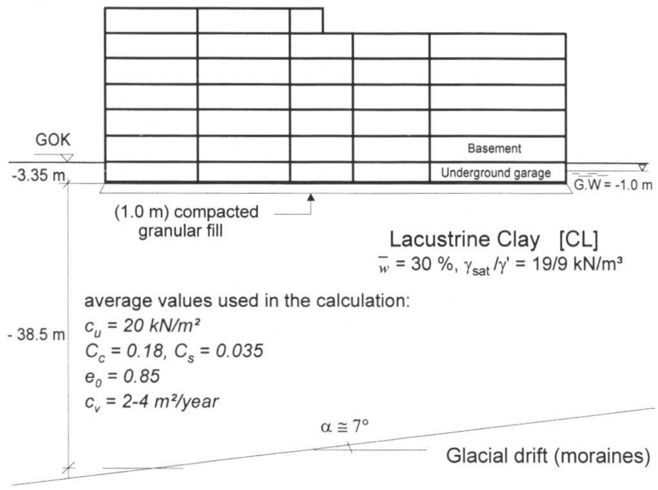

Fig. 9: Profile, soil conditions and cross section, case 2

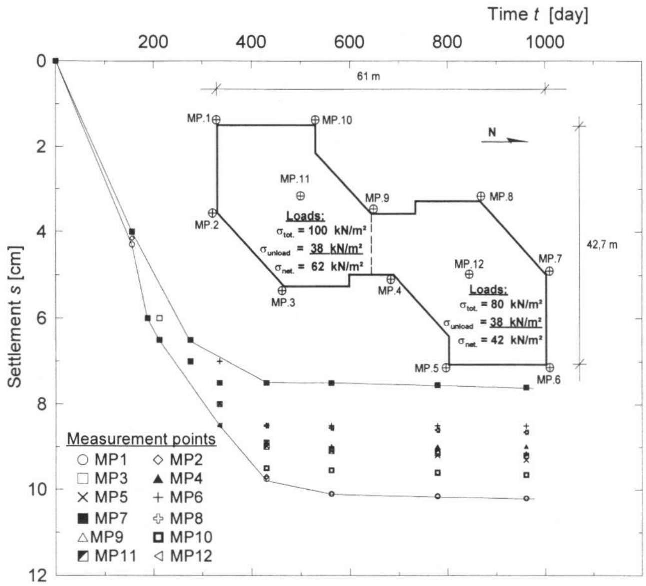

Fig. 10: Settlement record, case 2

Case 3: Apartment and commercial building, Radolfzell

The building in case 3 has 7 floors, basement and underground garage. The building has a base dimension of 21.5 × 21.8 m² with a total load of 126 kN/m². It is founded on a 0.8 m thick raft foundation that rests on a 0.5 thick well-compacted granular fill.

The subsoil conditions were explored by 3 borings and 2 test pits where a lacustrine soft clay layer with a thickness of 11 m was found which is underlain by the glacial drift (moraine). Some geotechnical data of the clay are summarized in Fig. 11.

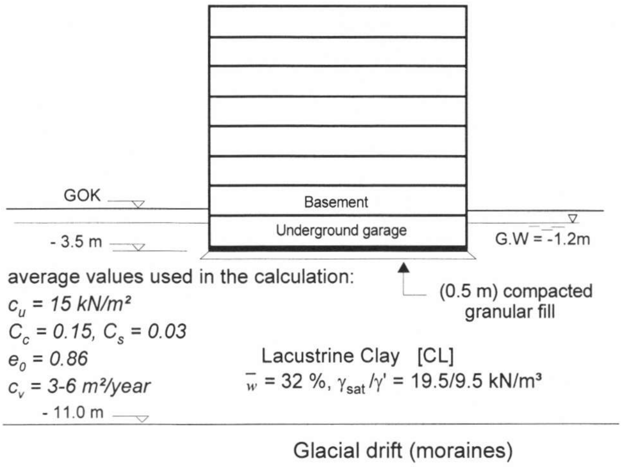

Fig. 11: Profile, soil conditions and cross section, case 3

The settlement was measured at the raft center over a period of 378 days only (see Fig. 12). In this case study the observation was not far enough to analyze the field secondary settlement fairly. Nevertheless, the consolidation of layer was completed within the observation time.

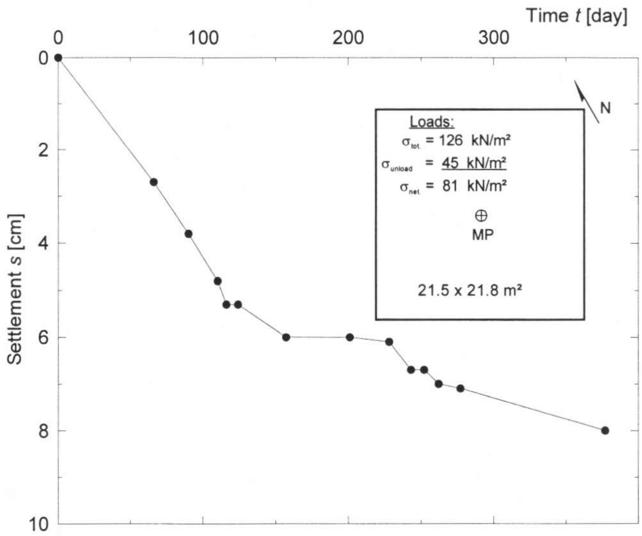

Fig. 12: Settlement record, case 3

Case 4: Administrative building, Constance

In this case an administrative building in Constance is analyzed. The 4-storey house with underground floor has a base dimensions of 17.0 × 12.4 m² with a total load of 72 kN/m². It is founded on a 0.5 m thick raft that rests on a 0.5 thick well-compacted granular fill.

The subsoil consists of soft, low to middle plastic lacustrine clay with a thickness exceeding 30 m. The normally consolidated clay has an undrained shear strength of about 20 kN/m² and an average natural water content of 28 %. The soil characteristics are shown in Fig 13.

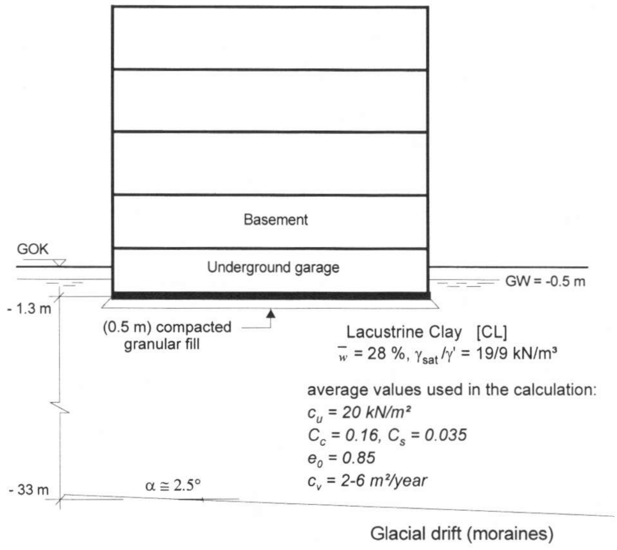

Fig. 13: Profile, soil conditions and cross section, case 4

The settlements have been monitored in the building corners over a period of 2343 days, (see Fig. 14).

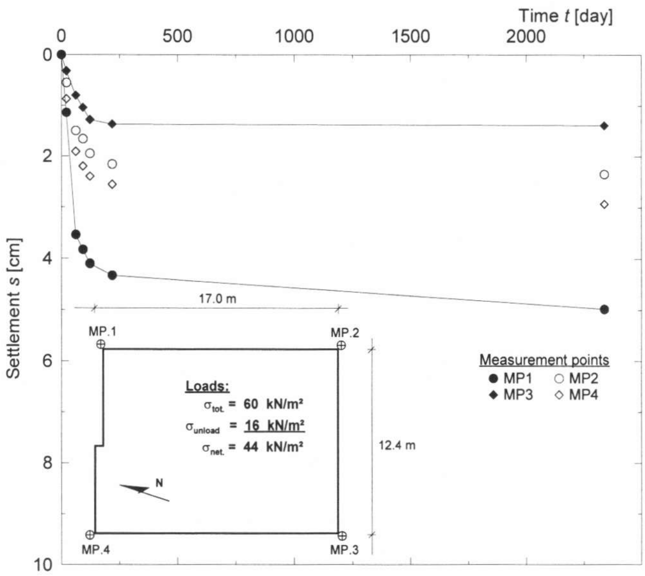

Fig. 14: Settlement record, case 4

Cases 5 and 6: Two office buildings, Constance

The cases 5 and 6 deal with two adjacent office buildings. Each building has three storey, a basement, an underground floor and an attic. On the other hand, their foundations consist of two detached rafts that can be separately analyzed considering the interaction of the two buildings. Each raft is 19 × 28 m² with an average total load of 65 kN/m².

The subsoil conditions and characteristics are summarized in Fig. 15. Below the foundation level a lacustrine soft clay layer to a depth of about 6.2 m was encountered. This layer is also underlain by the glacial drift (moraine).

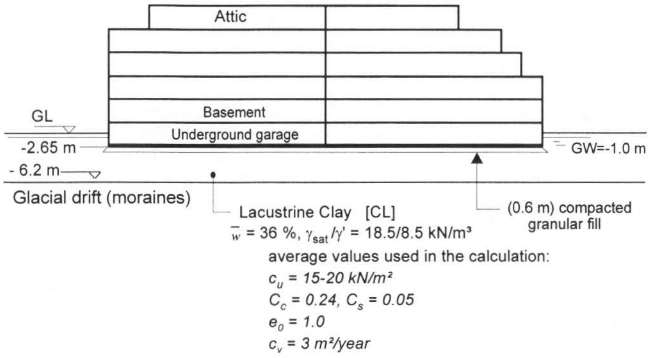

Fig. 15: Profile, soil conditions and cross section, cases 5 , 6

The settlements of the two buildings were monitored at 10 measurement points over a period of 780 days, see Fig. 16.

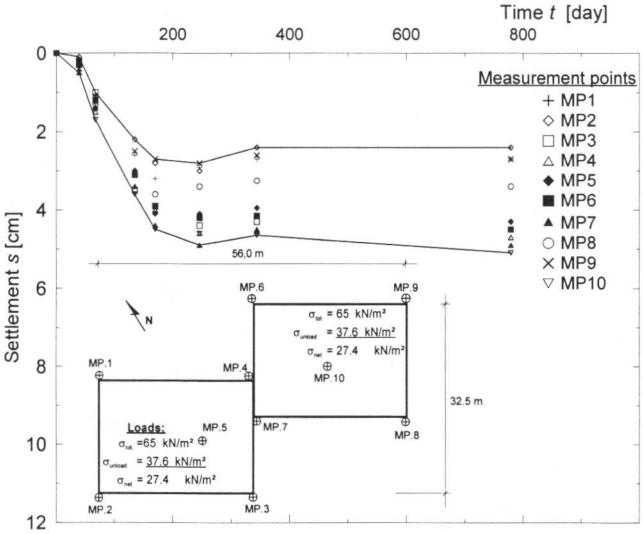

Fig. 16: Settlement record, cases 5 and 6

Cases 7 and 8: Office and storage buildings, Constance

In this example an administrative and a storage building in Constance are presented. Each building has 3-storeys, a basement and an underground floor. The office building has a rectangular form, its raft is 38.5 × 18.6 m ² with an average total load of 71.5 kN/m². The storage house has an L-form with a total load of 68 kN/m². Both buildings were placed on the soft clay layer at a depth of 2.0 m so that the weight of the excavated soil was 27 kN/m² at the foundation level.

The subsoil condition was explored by two borings to a depth of about 33 m. A layer of soft, middle plastic lacustrine clay with a total depth of 18 m was encountered below the ground surface. This layer is followed by soft, low plastic lacustrine clay up to a depth of 28 m where the moraines layer begins, see Fig. 17.

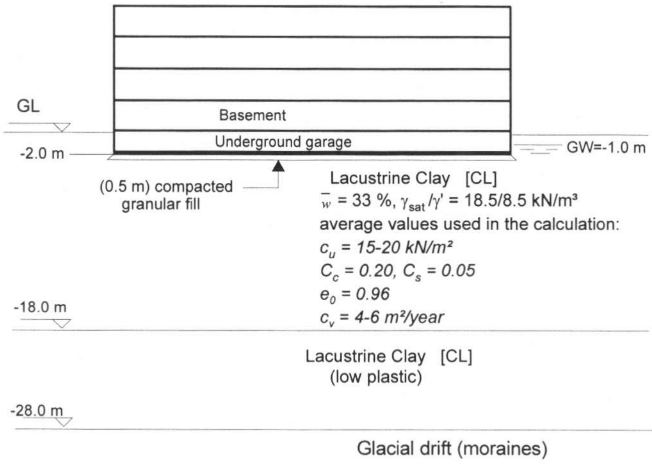

Fig. 17: Soil conditions, cases 7 and 8. Cross section, case 7

Settlements in cases 7 and 8 were measured over a period of 952 days at six and eight points, respectively (see Fig. 18 and 19).

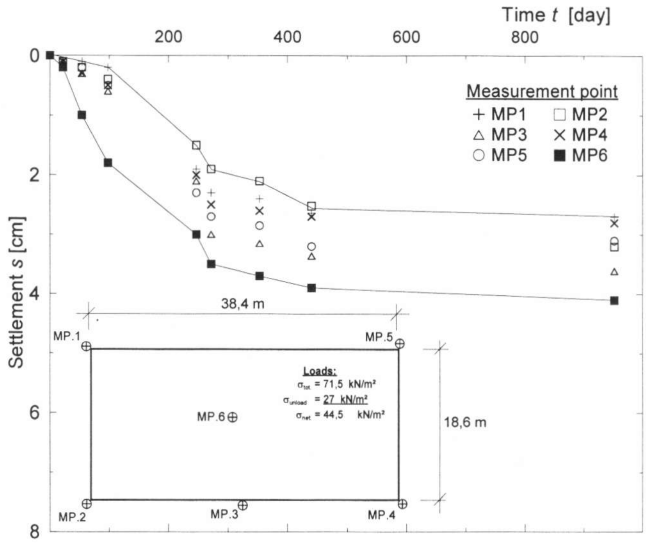

Fig. 18: Settlement record, case 7

Case 9: Office and administrative building, Constance

In the 9th case a 4-storey building with underground floor is analyzed. It has an irregular shape foundation with a total laod of 87 kN/m² (see Fig. 20). The building was placed on the soft clay layer at a depth of 2.5 m The average depth of the groundwater was about 0.5 m below the ground surface so that the weight of the excavated soil is 26 kN/m² at the foundation level.

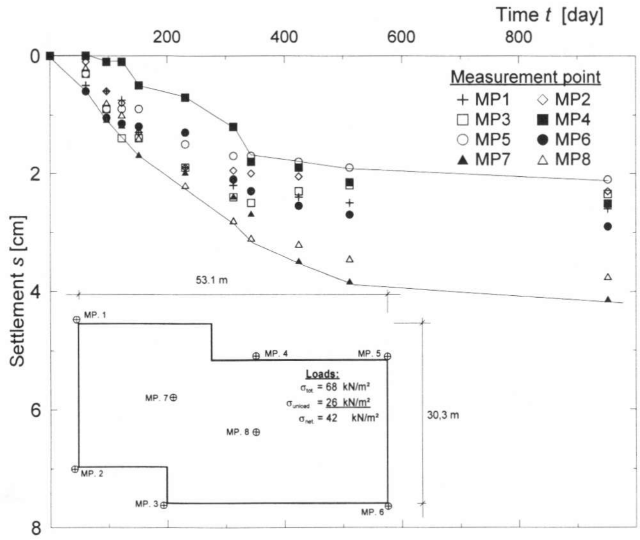

Fig. 19: Settlement record, case 8

The subsoil conditions were explored by two borings to a depth of about 35 m where a deep moraine layer was reached. Between the ground surface and the moraine layer a layer of soft, middle plastic lacustrine clay was encountered. The building in case 9 is located closer to the northern coast of the Rhine river, like the buildings in cases 7 and 8,and the geotechnical properties are similar to those shown in Fig. 17.

Because of its low strength and high compressibility the soft layer was precompressed by applying a preloading of 40 kN/m² caused by 2 m thick granular fill. The preloading was removed after 6 months and an average settlement of 5 cm was measured. Therefore, about 55 % of the total building load lay within the recompression range. This result is obtained taking into account the consolidation process due to the fill during 6 months and the excavation load.

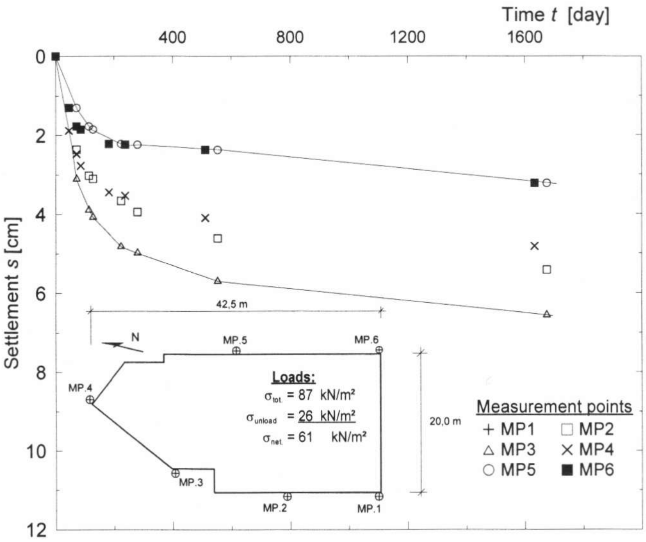

Fig. 20: Settlement record, case 9

The settlement was recorded at 6 points by regular measurements over a period of 1675 days (see Fig 20).

Case 10: Service and administrative building, Constance city

The last example of the back-analysis deals with a service and administrative building in Constance city. The building has 5-storeys and an underground floor. It has approximately a rectangular raft foundation with entire dimensions of 36.6 × 27.9 m² and a total load of 84 kN/m² (see Fig. 21).

The subsoil condition was explored by 4 borings which were extended to a depth of about 20 m and 13 soundings to a depth of 6 m. Also in this site a layer of soft, middle plastic lacustrine clay was encountered up to the end of the borehole. The soil properties are very close to the properties shown in Fig. 17, simply because this building is located at the other side of the street opposite to the buildings in cases 7 and 8.

The settlements were recorded at 6 points by regular measurements over a period of 474 days (see Fig 21). Similar to case 3, the field secondary settlement could not be fairly analyzed because of the relatively short observation time.

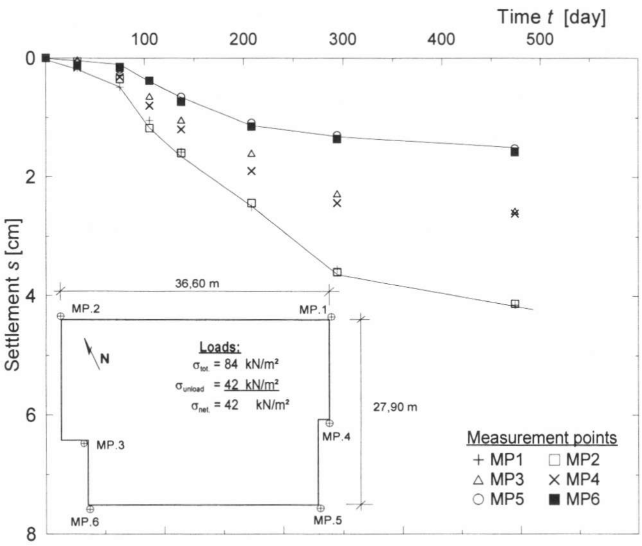

Fig. 21: Settlement record, case 10

RESULTS OF THE BACK-ANALYSIS

By applying the methods explained previously, the field values of the final field settlement $\bar{s}_{\infty}$ and the coefficient of consolidation c_v and consequently the primary settlement s_p as well as the consolidation time t_p for all cases were calculated. By substituting these values in equations 4 and 5, respectively, the average field compression index C_c and the average field coefficient of secondary settlement C_{α} could be estimated.

Results from the back-analysis of the primary consolidation are summarized in table 1.

Table 1. Back-analysis results of the primary consolidation

Case	$s_{cal.}$ [cm]	$c_{v(field)}$ [m²/y]	$t_{p(field)}$ [day]	$s_{p(field)}$ [cm]	$C_{c(field)}$ [-]	$C_{c(field)}/C_{c(lab)}$ [-]
1	9.5	32.0	748	6.15	0.104	0.65
2	12.7	43.4	365	9.1	0.126	0.70
3[1)]	11.6	19.5	275	7.4	0.128	0.71
4	7.5	36.4	677	4.6	0.080	0.62
5	7.1	3.1	318	4.4	0.145	0.63
6	7.1	3.1	330	4.7	0.155	0.67
7	6.8	41.6	445	4.0	0.118	0.59
8	7.1	33.0	560	4.1	0.115	0.58
9	7.3	41.8	670	4.85	0.166	0.75
10	6.7	15.8	380	4.7	0.160	0.69

[1)] The settlement in case 3 was calculated at the raft center and not at the characteristic point since the settlement was measured at this point only.

The estimated values of the field coefficient of the secondary settlement $C_{\alpha(field)}$ together with the field values of compression index C_c shown in table 1 allow to evaluate the field value C_α/C_c. The results of the back-analysis of the secondary settlement are summarized in table 2.

Table 2. Back-analysis results of the secondary consolidation

Case	C_α *(field)*	C_c *(field)*	C_α/C_c *(field)*
1	0.0027	0.104	0.038
2	0.0026	0.126	0.021
3[1)]	0.0027	0.128	0.021
4	0.0016	0.080	0.020
5	0.0034	0.145	0.023
6	0.0036	0.155	0.023
7	0.0027	0.118	0.023
8	0.0027	0.115	0.023
9	0.0039	0.166	0.023
10	0.0038	0.160	0.024
average	0.0030	0.130	0.024

Evaluation of the results

From the above back-analysis of the 10 case histories of buildings with raft foundations on lacustrine soft clay the following conclusions have been derived:

- For the investigated cases the ratio of the primary settlements observed to calculated has an average of 0.67. Similarly, the ratio $C_{c(field)}/C_{c(lab)}$ has also an average of 0.67.
- Except for cases 5 and 6 the field coefficient of consolidation c_v is 4 to 15 times larger than the laboratory values. The laboratory and field c_v-values are identical in cases 5 and 6. This is because a comparatively thin clayey layer is located between the raft foundation and the stiff moraine layer and the consolidation is obviously one-dimensional. Similar field observation was reported by *Terzaghi and Peck (1967)*. The high observed c_v-values in the other cases are perhaps due to the multidirectional consolidation or to the existence of unknown drainage zones (stratigraphy) that may be missed in the subsurface explorations.
- Data about the secondary settlement could be obtained since the consolidation time t_p for all cases was determined within the observation time. The short observation time attributes to the fact that the settlements were monitored by practicing engineers for controlling purpose and not for research purpose. On the basis of the available long-term observation a field coefficient of secondary settlement was determined using the equation (5) for all cases with an average value of $C_\alpha = 0.003$. Thus, the normally consolidated lacustrine clay in south Germany has a low secondary compressibility according to the classification after *Mesri (1973)*.
- The ratio C_α/C_c seems to be constant with an average value of 0.024, which is lightly below the lower limit in the equation

$$C_\alpha / C_c = 0.04 \pm 0.01 \tag{6}$$

proposed by *Mesri and Choi (1984)* for inorganic soft clays.

EFFECT OF LOAD INCREMENT AND LOADING RATE

Incremental loading test (IL-test)

In trying to explain the discrepancy between the measured and calculated settlement the authors supposed at first that the computed compression index C_c is overestimated while this parameter is determined on the basis of common incremental consolidation tests (i. e. the ratio of load increment to existing load $\Delta\sigma/\sigma$ is usually 1 and each load is maintained for 24 h) and it is not based on EOP-consolidation tests (end of primary consolidation). According to the authors experience, the time primary consolidation stage t_p of this soft clay varies between 10 and 240 minutes. Based on this fact numerous consolidation tests on high-quality specimens of lacustrine clay were carried out to determine the ratio $C_{c\text{-}EOP}/C_{c\text{-}24h}$.

It should be noted here that the end of primary consolidation has been determined by measuring the excess pore water pressure at the impermeable bottom of the oedometer cell. As an example both standard- and EOP-pressure-void ratio curves for a typical specimen are shown in Fig. 22.

By means of 24h- and EOP-standard consolidation tests with an increment of $\Delta\sigma/\sigma = 1$ a range of $C_{c\text{-}EOP}/C_{c\text{-}24h}$ between 0.84 and 0.92 was obtained.

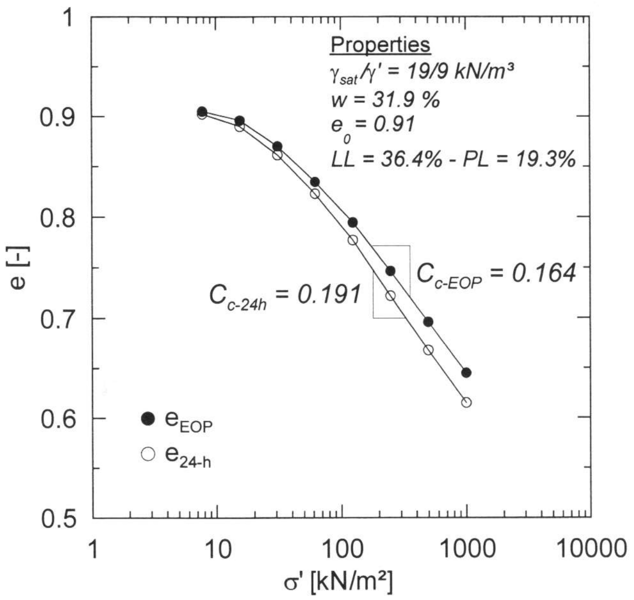

Fig. 22: Typical 24h- and EOP-compression curves of the lacustrine soft clay in southern Germany

Thus, it appears that the overestimation of the field C_c-value could not be clarified using the standard consolidation test only. For this reason a new series of consolidation tests with small load increments was carried out, because the authors believe that the doubling of load was conventionally established for operating purpose in laboratory. On the other hand, such load increment is unexpected in any practical case.

Figure 23 shows test data of two consolidation tests with different load increments on specimens from the same tube sampler. Both specimens were preconsolidated up to a pressure of 62.5 kN/m² by multi-stage loading. From this pressure one specimen has been loaded conventionally (i. e. $\Delta\sigma/\sigma = 1$) and the other specimen with an increment of $\Delta\sigma/\sigma \approx 0.1$ up to the next load of 125 kN/m². All increments were applied under EOP-condition which was controlled by measuring the excess pore water pressure using pore pressure transducer of ±0.6 kN/m² accuracy.

It is evident that the soil response depends on the load increment. If the increment is high the structure of the soft soil may be destructed under massive pressure. The influence of the load increment on the soil behavior can be quantitatively examined by determining the C_c-values for the different load conditions. In regard to the standard procedure (path AD in Fig. 24) the compression index C_c has a value of $C_{c\text{-}24h} = 0.132$ whereas, the EOP-value (path AC) is $C_{c\text{-}EOP} = 0.111$. On the other hand, the C_c-value decreases to 0.103 in the case of a load with increment of $\Delta\sigma/\sigma \approx 0.1$ (path AB).

Taking into account that the ratio of the total load and the initial field stress in the middle of the compressible layer varies between 0.3 and 0.6 in the demonstrated case histories, the following question can be posed: what would happen to the compression index if, for example, the specimen is loaded to half of the load (93.75 kN/m²) at once (path AB′)?. The point B′ will be overlooked by doubling the load and the C_c-value remains the same. However, the use of small increment can better trace the consolidation curve. Moreover, the compression index has a value of 0.094 in this case and the ratio $C_{c\text{-}(\Delta\sigma/\sigma=0.1)}/C_{c\text{-}24h}$ amounts 0.73.

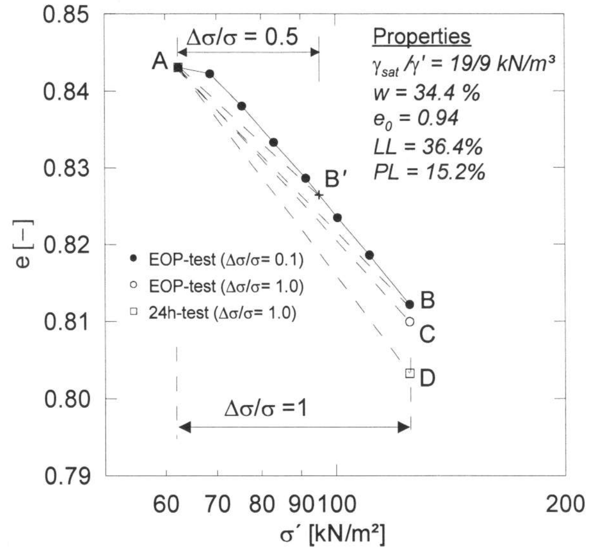

Fig. 23: Consolidation test with two different load increments

Constant Rate of loading test (CRL-test)

Although the consolidation tests with small increments describe well the stress-strain-behavior of the soft soil they can not be applied for the engineering practice because of the excessive testing period. Alternatively, the constant rate of loading test (CRL) may offer several advantages over the conventional incremental test (IL), see for example *Aboshi et al. (1970)*, *Janbu et al. (1981)*. In addition to the short testing time the CRL-test provides a continuous tracing of the consolidation curve. Therefore, it is advisable to use the continuous loading test to estimate the compressibility parameter of the soft soils.

The influence of the rate of loading $\dot{\sigma}$ on the compressibility is a vital question in the CRL-tests. At the present time there is no satisfied criterion to select an appropriate loading rate, because all criteria were established by comparing results of CRL-tests at different loading rates with results of standard consolidation test to obtain the "best" fitting regardless of the shortcomings of the standard consolidation test. The authors use the following simple procedure to estimate the required $\dot{\sigma}$-value for the CRL-test from the field conditions based on the well-known model law of consolidation

$$T_{v1}(lab) = T_{v2}(field) \Rightarrow \frac{c_{v1} \cdot t_1}{H_1^2} = \frac{c_{v2} \cdot t_2}{H_2^2} \tag{7}$$

$$\Rightarrow t_1 = t_2 \cdot \frac{c_{v2}}{c_{v1}} \cdot \left(\frac{H_1}{H_2}\right)^2 \tag{8}$$

where T_v is the time factor. First the expected time of construction t_2 and the field coefficient of consolidation cv_2 must be known. Then the field data in addition to the laboratory data could be substituted into equation (8) to calculate the laboratory time t_1 required for the same degree of consolidation. This proceeding is true if the specimen and the soil layer are acted upon by the same pressure. Thus, the laboratory rate of loading can now be estimated by dividing the total field load over the laboratory time t_1. For example a value of about $\dot{\sigma} = 120$ kN/m² /h for the first case history was calculated. In the same way the laboratory rate of loading from all case histories were back calculated and it varies between 80 to 140 kN/m² /h. Under this rate of loading a series of CRL-consolidation tests were carried out. Results of CRL-test and IL-standard test on specimens from the same tube sampler are compared in Fig. 24.

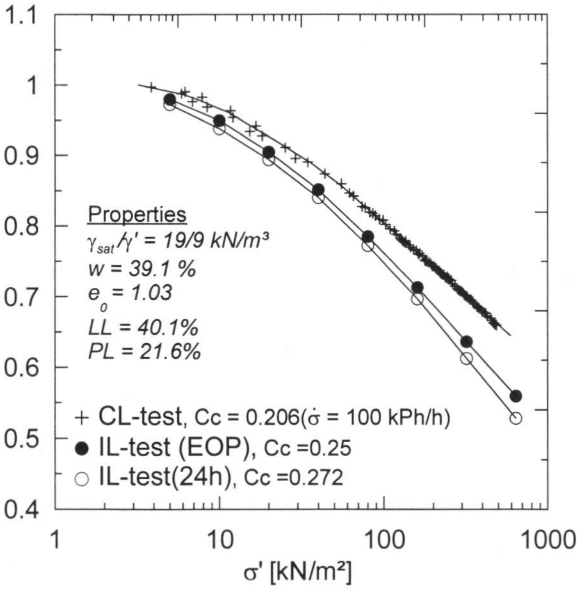

Fig. 24: Comparison of the compressibility of the lacustrine clay in IL-test and CRL-test

The ratio $C_{c\text{-}CRL} / C_{c\text{-}IL(24h)}$ has an average value of 0.74 for this lacustrine clay which is close to the average value of the ratio $C_{c(field)} / C_{c(lab.)} = 0.67$ obtained in the back-analysis.

CONCLUSIONS

A valuable back-analysis of 10 buildings on soft clay in southern Germany show that the calculated settlements using the compressibility parameter from the standard consolidation test are on average 50 % larger than settlements actually measured in this area. This enormous discrepancy is due to the overestimating of the compression index using the standard consolidation tests on soft soil. In order to avoid this discrepancy the authors favor the constant rate of loading test (CRL) to determine the compressibility parameters for soft soils. Otherwise a small increment ($\Delta\sigma/\sigma = 0.1$ to 0.2) with pore pressure measurement to investigate the EOP-phase is recommended if the standard consolidation test will be used.

REFERENCES

Aboshi, H. / Yoshikuni, A. / Mariyama, S. [1970]. "Constant Rate Consolidation Test", Soil and Foundations 10 (1), pp. 43-56.

Asaoka, A [1978]."Observational Procedure of Settlement Prediction", Soil and Foundations, Vol. 18(4), pp. 87-101.

Gebreselassie, B. [2003]. "Experimental, analytical and numerical Investigation of the Excavations in Normally Consolidated Soft Clays", Phd thesis, Institute of Geotechnics, Kassel University, Vol. 14.

Graßhof, H. [1955]. "Setzungsberechnungen starrer Fundamente mit Hilfe des kennzeichnenden Punktes", Der Bauingenieur, Vol. 30(2), pp. 53-54.

Jamolkowski, M. / Ladd, C. C. / Germaine, J. T. / Lancellotta, R. [1985]. "New Development In Field and Laboratory Testing of Soil", *Proc. of 11th ICSMFE*, San Francisco, Vol. 1, pp. 57-153.

Janbu, N. / Tokheim, O. / Sennwset, K. [1981]. " Consolidation Tests with Continuous Loading", *Proc. of 10th ICSMFE*, Stockholm, Vol. 1, pp. 645-654.

Lamb, T. W. [1973]. "Prediction in soil engineering", *13th Rankine Lecture, Géotechnique,* Vol. 23(2), pp. 149-202.

Lerouil, S. / Tavenas, F. [1981]. "Bitfalls of Back-Analysis", *Proc. of 10th ICSMFE*, Stockholm, Vol. 1, pp. 185-190.

Mesri, G . [1973]. "Coefficient of secondary settlement", Journal of the Geotechnical Engineering Division, ASCE, Vol. 99(1), pp. 123-137.

Mesri, G . / Choi, Y. K. [1985]."Settlement Analysis of Embankments on Soft Clay", Journal of the Geotechnical Engineering Division, ASCE, Vol. 111(4), pp. 441-464.

Scherzinger, T [1991]. "Materialverhalten von Seetonen-Ergebnisse von Laboruntersuchungen und ihre Bedeutung für das Bauen in weichem Untergrund", Phd thesis, Institute of Soil Mechanics and Rock Mechanics, Karlsruhe University.

Terzaghi, K. V. and Peck, R. [1967]. "Soil Mechanics in Engineering Praxis", John Wiley & Sons Inc., New York.

Weissenbach, A. and Kempfert, H.-G. [1995]. "German national report on braced excavation in soft soil", *Proc. Int. Sym. On underground construction in soft ground*, Balkema. pp. 33-36.