DR Koncerthuset in Copenhagen - a concert hall on three legs

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ABSTRACT
In 2002 the construction phase of DR-Koncerthuset (the Copenhagen Concert Hall, the Danish Broadcasting Corporation) began. After 7 years of construction the concert hall opened in 2009. The concert hall is a more than 10.000 ton structure resting mainly on three large stair towers like the legs of a three legged chair. With one of the legs leaning outwards the foundation designers was met with severe requirements.
The foundation project was treated as CC3 and geotechnical category 3. As the geotechnical design took place at the same time as the construction work began, a detailed investigation was initiated with a tight time schedule in order to provide sufficient detailed information in due time to cope with the challenging statics of the concert hall and derived foundation requirements.
The Concert hall is designed as direct foundations in glacial deposits of mainly stiff to hard clay till. In order to make use of the high strength of the deposits the geotechnical supervision of the foundation work was integrated with a detailed investigation comprising more than 100 geotechnical boreholes, 24 plate load tests, triaxial tests, and consolidation tests. In addition, the risk of differential settlements caused by variations in the consolidation properties was evaluated by settlement monitoring during construction.
This paper deals with selected geotechnical design considerations, the prediction of the settlement of the main foundations, the interpretation of the results of the settlement monitoring, and how the whole geotechnical design and supervision was integrated.

Keywords: Settlements, Heavy loads, Monitoring.

1 INTRODUCTION

In 1999 the Danish Broadcasting Corporation (DR) commenced the construction of DR-byen. This included The Copenhagen Concert Hall, which is a concert hall with a main auditorium seating 1800 people. The main auditorium is basically a large hollow structure resting on 3 legs of which one of the legs has an outwards inclination thus creating "unstable" statics, illustrated in figure 1.

The three legs are constructed as stair towers. In figure 2 the inclined tower is seen to the right, just next to the stairs leading up to the foyer.

The stair towers are resting on three large footings F09, F10 and F11, where F11 is carrying the inclined stair tower. The foundation layout is shown in in figure 3.

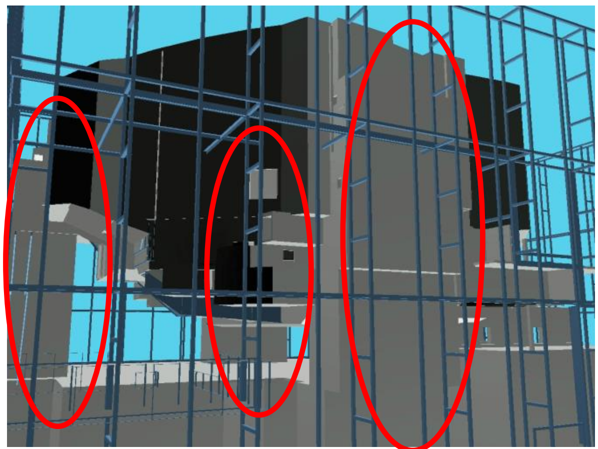

Figure 1: The concert hall structure illustrated in 3D. The red line indicates the location of the three main stair towers (grey) carrying the main auditorium (black). The left most is an inclined stair tower.

Figure 2: The foyer with the inclined stair tower resting on F11 to the left. The main auditorium is seen over the foyer, and is covered with shells (Illustrations from Ateliers Jean Nouvel).

The foundation was established in approx. level -4, which is about 6 m below ground level in stiff glacial deposits.

One of the issues was settlements caused by the very large foundation loads. Initial settlement calculations indicated a risk of large settlements. For this reason a comprehensive field and laboratory work was introduced along with a revised method for settlement evaluation.

This initial settlement assessment was based on information from a few traditional geotechnical boreholes only. For this reason further analysis and investigations were necessary.

Furthermore the detailed design was taking place as the construction pit was excavated, and the detailed geotechnical investigations adapted to the foundation design was only possible in a short window after the excavation and the general blinding layer was finished – this adding additional challenges to the project.

2 GEOLOGY

The geology on the site is dominated by glacial till deposits, which are heavily overconsolidated. These deposits are mainly clay till but are intercalated by sand till and to a minor degree by melt water sand. The glacial deposits are underlain by limestone in level app. -10.00, which on the upper 1.00 – 2.00 m is hardness H1 – H2 and hereunder hardness H3 or more, but locally the limestone has been encountered from aprox level -9.

The construction site is located on the edge of "Rådhusdalen", a highly permeable erosional valley in to the prequatenary limestone.

3 THE FOUNDATION PROJECT

The three large footings have side lengths in the range 7.80 -23.50 m. More specifically their geometry and loads (SLS) are listed in Table 1.

Table 1: Dimension of SLS loadings on footings

	Dimension	**Load Vertical**	**Moment**	
	(m x m)	(kN)	$M_{S,SLS}$ (kNm)	$M_{L,SLS}$ (kNm)
F09	14.80*23.50	84288	5181	16058
F10	7.80*15.10	49017	3122	53966
F11	8.70*15.15	68134	10142	68143

The footings were concreted directly on the cleaned bottom after removal of some soft spots discovered from plate load tests and vane tests.

The foundation level for the three main footings was generally in approx. -4.00, about 6 m below ground level, and approx. 5 m above top of limestone encountered from approx. level -9.

An overall layout of the foundation geometry is shown in Figure 3. The three main footings are located outside Rådhusdalen, but a very comprehensive groundwater lowering system was established with nearby reinjection. The edge of Rådhusdalen is seen on figure 3 were the contours indicate a steep surface of the limestone. The groundwater lowering was keep to a minimum and only to a very small depth below foundation level (less than a meter).

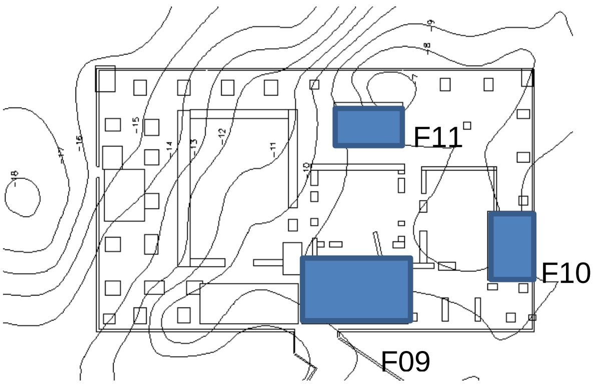

Figure 3: The foundation layout with the three main foundations F09, F10 and F11. The contours shows initially assumed level of limestone. Further boreholes showed limestone from approx. -8,5 to -9 for F10 and F11, and approx. -10 for F09. Rådhusdalen is seen at the left part where the surface of the limestone is steep.

The concreting of the towers and the Concert Hall were done in “lifts” wherefore the load increments were so small, that the load increase may be considered as continuous. The footing were cast ultimo 2003, and the Concert Hall was finished primo 2006, and for this reason the load apply may be considered as very slow.

4 CONSTRUCTION VERSUS DESIGN

The construction of the pit for the basement and foundation work was commenced while the design process was still under way, and only an initial design was present, based on a few boreholes only.

Due to very large foundation stresses and non-conventional structure and statics, the project had to be treated in “Skærpet funderingsklasse” equal to geotechnical category 3.

As a consequence of the very tight project schedule some of the secondary columns and foundations were designed and executed before the detailed design of the concert hall was completed.

Unfortunately a result of the very stiff structure of the concert hall, was that the settlement of the major three footing F09, F10 and F11 would lead to an unaccepted increase of the load on some of these secondary supports. For this reason an assessment of the settlement of the three major footings was of height importance. Particular as the consequence of settlements could be a significant increase of the dimensions of the three major footings.

Detailed investigations were therefore needed, and furthermore the high undrained strength and the large foundation dimensions made traditional foundation inspections hard to carry out using manual methods (hand auger and hand field vane). For this reason all significant footings was both inspected visually and with 1 to 3 geotechnical boreholes, to verify the basis of design. In total more than 100 geotechnical boreholes were executed within the construction area.

This approach provided the basis for detailed investigations, to verify the initial geotechnical design basis.

Caused by a very tight time the installation work for the uplift anchors was commenced right after the pit has been excavated and the blinding layer for the base plate could carry the construction equipment.

The detailed investigations had to be executed in parallel with execution the uplift anchors. This was only possible in close cooperation with the contractor to avoid the geotechnical rigs getting caught in a tight mesh of uplift anchors.

5 THEORY AND COMPUTATIONAL MODEL

The initial evaluation of the predicted settlements was based on the traditional assumption that:

$$K_t = K_{t,0} + \Delta K_t \sigma_{red} \qquad (1)$$

where: K_t is the module of consolidation at the stress $\sigma_{red.}$ $K_{t,0}$ is the module of consolidation at the stress 0, ΔK_t is a constant, and σ_{red} is the minimum effective vertical stress since the ice age (occurs immediately prior to concreting).

From experience obtained at nearby sites it was expected that $K_{t,0} \approx 20$ MPa and $\Delta K_t \approx 2500$ for clay till and $K_{t,0} \approx 50$ MPa and $\Delta K_t \approx 600$ for sand till. The effective weight of the soil was expected to be $\gamma' = 13$ kN/m^3 for clay till and $\gamma' = 11$ kN/m^3 for sand till. σ_{red} is set to $\gamma \cdot z$ where z is the depth under foundation level.

Assuming the footing is modelled by a square footing with side length B yielding the same area as the very footing and assuming a load distribution of 1:2, the consolidation settlement may be determined by:

$$\delta = \int_0^\infty \frac{\sigma(z)}{K_t(z)} dz \qquad (2)$$

$$= \int_0^\infty \frac{p_{sls}}{(B+z)^2(K_{t,0} + \Delta K_{t,0}\gamma z)} dz$$

$$= P_{SLS}\left(\frac{1}{B\left(K_{t,0} - \Delta K_t \gamma B\right)} + \frac{\Delta K_t \gamma \ \ln\left(\frac{\Delta K_t \gamma B}{K_{t,0}}\right)}{\left(K_{t,0} - \Delta K_t \gamma B\right)^2}\right)$$

In the above integration the upper boundary has been set to infinity for the sake of convenience. It shall be emphasized that the ΔK_t term will reduce the settlement contribution rapidly with depth. For this reason the settlement contribution is rather low from top of limestone and below. Integrating to a large depth will tend to give a (slightly) conservative estimate of settlement.

Based on the above information the initial settlement estimate was:

Table 2: Settlement properties from borehole information and calculated settlements

	P_{SLS}	B	$K_{t,0}$	ΔK_t	γ	δ
	(kN)	(m)	(MPa)	(MPa)	(kN/m^3)	(mm)
F09	84288	18.65	20	2500	13	20
F10	49017	10.85	20	2500	13	28
F11	68134	11.48	20	2500	13	35

6 FIELD AND LABORATORY TESTING

As the previous field and laboratory work only comprised a few deep boreholes a more detailed field work was commenced in order to provide sufficient basis for the foundation design.

The main purpose of the field and laboratory testing is of course to obtain more site relevant values of the two parameters $K_{t,0}$ and ΔK_t.

6.1 Field Work

The field testing comprised 17 plate load tests all on Ø30 cm plates. They were carried out in three steps each of 0.5 σ_{SLS}, i.e. to a maximum stress of 1.5 σ_{SLS}.

A plate load test with this small diameter has a rather small influence depth compared to the thickness of the glacial sediments. For this reason the plateload test has been used primary for a local calibration/verification of the stiffness parameters of the till at the actual level, and not as a measure for the total layer of the glacial deposits, which also affects the interpretation as described below.

The evaluation of the tests was based on the middle step to eliminate the bedding effect at the start and to disregard possible curvature above σ_{SLS}. The evaluation was hereafter done by inserting $B = 0.266$ m ($= (\pi/2)^{0.5} \cdot 0{,}3$ m), $\Delta K_t = 2500$ MPa and $\gamma = 13$ kN/m^3 in formula (2) and adjust the value of $K_{t,0}$ until the observed settlement is obtained.

The reason for this procedure is that it is impossible to find both $K_{t,0}$ and ΔK_t from just one set of observations, and since the test mainly involves superficial soil and has a rather limited influence depth, it was decided to lock ΔK_t at its basic value 2500 MPa and determine $K_{t,0}$.

The outcome of this procedure is presented in Table 3: $Kt_{,0}$ derived from plate load tests. and all derived $K_{t,0}$ values except one are well above the basic value 20 MPa.

Table 3: $K_{t,0}$ derived from plate load tests

Footing	Test No.	Remarks	$K_{t,0}$ (kPa)
F11	P05B		53.610
F11	P06		62.815
F11	P07		36.675
F11	P08B		28.413
F10	P09		52.934
F10	P10		60.180
F10	P11		29.163
F10	P12		55.638
F09	P18		41.439
F09	P19		54.421
F09	P20	Uncertain!	98.907
F09	P21		40.601
F09	P23		43.208
F09	P24	Uncertain!	10.335

6.2 *Laboratory testing*

The Oedometric tests have been executed by consolidating the samples to their estimated preconsolidation stress $\sigma_{p,c}$ (≈2400 kPa) and hereafter unload/reload to/from 80, 40 and 20 kPa respectively. This stress path has been chosen to enable determination of both $K_{t,0}$ and ΔK_t in one single test, although it must be expected that $K_{t,0}$ will be underestimated due to sample disturbance.i

The preconsolidation stress $\sigma_{p,c}$ is evaluated by the SHANSEP formula with parameters for Danish clay till as presented by Christensen, et al.(1992)

$$0.4\left(\frac{\sigma_{pc}}{\sigma_v}\right)^{0,85} = \frac{c_v}{\sigma_v} \qquad (3)$$

With measured vane strength $c_v \approx 550$ kPa and a overburden pressure $\sigma_v \approx 20$ kPa (3) yields $\sigma_{p,c} \approx 2900$ kPa. On this background it has been chosen (conservatively) to preload all samples to $\sigma_{p,c} = 2400$ kPa.

Initially one Oedometer test was planned per footing, and the main results from these three tests are presented in table 3:

Table 4: Results from initial oedometer tests

Footing	Boring	Sample	Test results $K_{t,0}$ (kPa)	ΔK_t (MPa)
F09	B8	Pose 1	23702	639
F10	B18	38B	30388	4613
F11	B65	5	37337	1651

The combination of $K_{t,0}$ and ΔK_t for footing F09 imply so poor stiffness properties of the soil that a settlement computation based on these, will lead to a settlement of δ≈50 mm. In the light of the general experiences from the area this result looked untrustworthy, and therefore it was decided, to supplement the F09 test with two more oedometer tests.

Table 5: Main results from additional oedometer tests

Footing	Boring	Sample	Test results $K_{t,0}$	ΔK_t
F09	B90	904	10137	2033
F09	B92	916	1109	921

As it is seen, these supplementary tests did not provide results that look realistic, and especially the $K_{t,0}$ values look quite unrealistic. This is no doubt due to sample disturbance.

7 INTERMEDEATE SETTLEMENT EVALUATION

The Oedometer tests gives the best estimate of ΔK_t, and defines this value to 1062, 4613 and 1651 for F09, F10 and F11 respectively (the first figure is the geometric mean of the ΔK_t values from the three F09 tests). Based on these values and based on the same guidelines as described in section 6.1a re-evaluation of $K_{t,0}$ from the plate load tests is done as shown in Table 6 leading to $K_{t,0}$ values of 39392, 40893 and 46717 kPa respectively.

Hereafter the best sediments estimates based all available test are:

Table 6: Intermediate settlement evaluation

	P_{sls} (kN)	B (m)	$K_{t,0}$ (MPa)	ΔK_t (MPa)	γ (KPa)	Δ (mm)
F09	84288	18.65	39.392	1062	13	25
F10	49017	10.85	40.893	4613	13	14
F11	68134	11.48	46.717	1651	13	31

The tables 2 and 6 show in comparison that the field and laboratory testing confirmed the on hand knowledge of settlement conditions of the soil, but did not provide new information.

8 SETTLEMENT MONITORING

When the concreting of the footings was finished, they were all equipped with four levelling bolts (one in each corner), and regular (biweekly) measurements to these bolts started 20th November 2003.

A couple of years later when one third to one half of the SLS loads were applied to the footings it became clear, however, that the settlements were much smaller than expected from the above model. The 20th September 2005 the situation was:

Table 7: Settlement as per 20th September 2005

	P_{SLS}	Applied Load, Total		Settlements	
Date	Final	2005-09-20		2005-09-20	
	(kN)		% of final load	δ Computed	δ Measured
F09	84288	39304	46.6	12	3
F10	49017	19978	40.8	6	2
F11	68134	22687	33.3	10	3

As it is seen in Table 7 the observed settlements forms only one third to one fourth of the predicted settlements. This calls for a revision of the physical/mathematical model behind the computations.

It shall be noted that the settlements during 2005 were almost constant for the applied load and therefore the small settlement could not be explained as a slow and not completed consolidation.

9 REVISED MODEL

Since the observed settlements are much smaller than the predicted ones, a revised computational model calls for some effect which will lead to increasing soil stiffness with increased load.

The most straightforward way to implement the increase of soil stiffness is to introduce the load itself in the equation governing K_t, implying that the soil has sufficient time to drain of the excess pore pressures due to the increased load and hereby gain additional stiffness, i.e.:

$$K_t = K_{t,0} + \Delta K_t\left(\sigma_{red} + p(z,t)\right) \quad (4)$$

Where $p(z,t)$ is the stress in the depth z caused by the applied load to the time t.

For convenience and simplification the load is assumed applied linearly with time between $t = 0$ and $t = t_s$, i.e.

$$P(t) = \alpha t \ ; \ P_{sls} = \alpha t_s \quad (5)$$

and we get:

$$P(z,t) = p(t)/(B+Z)^2 \quad (6)$$

$$= \alpha t/(B+z)^2$$

leading to:

$$K_t = K_{t,0} + \Delta K_t(\gamma z + \alpha t/(B+z)^2) \quad (7)$$

Hereafter the settlement of a very thin layer, dz, through a very short time, dt, becomes:

$$dd\delta = dp(t,z)/K_t dz \quad (8)$$

$$= \frac{\alpha dt}{(B+z)^2(K_{t,0} + \Delta K_t(\gamma z + \alpha t/(B+z)^2))dz}$$

$$= \frac{\alpha}{\left(K_{t,0} + \Delta K_t \gamma z\right)(B+z)^2 + \Delta K_t \alpha t} \, dtdz$$

leading to a total settlement of:

$$\delta = \int_0^{\infty}\int_0^{ts} dd\delta \qquad (9)$$

$$= \frac{1}{\Delta K_t}\int_0^{\infty} \ln\left(1 + \frac{\Delta K_t\, \alpha\, \mathrm{t}}{(B+z)^2\left(K_{t,0}+\Delta K_t \gamma z\right)}\right) dz$$

This integration can hardly be carried out analytically, and therefore it has been done numerically.

In Table 8 the results of the revised settlement calculation is shown.

Table 8: Settlements per 20th September 2005. Revised computational method

	P_{SLS}	Applied Load, Total		Settlements	
Date	Final	2005-09-20		2005-09-20	
	(kN)		% of final load	δ Computed	δ Measured
F09	84288	39304	46.6	4	3
F10	49017	19978	40.8	2	2
F11	68134	22687	33.3	5	3

For this foundation project the settlements calculated with the revised method are more or less equal to the measured settlements, taken the accuracy of the settlement into account.

10 FINAL SETTLEMENT EVALUATION

Based on the revised computational method the final settlement evaluation was made ultimo September 2005. The outcome was:

Figure 4: Settlement for full load.

	P_{sls} (kN)	B (m)	$K_{t,0}$ (MPa)	ΔK_t (MPa)	γ (KPa)	δ (mm)
F09	84288	18.65	39.392	1062	13	10
F10	49017	10.85	40.893	4613	13	5
F11	68134	11.48	46.717	1651	13	13

Additional settlement measurements carried out from the period 2005-09-20 to 2008-09-08 showed additional approx. 2 mm settlement for all three footings - In total 5-6 mm settlement for all three footings at September 2008 were the load on the three main footings were close to full load.

This further settlement development was partly influenced by the end of groundwater lowering (reducing the effective stresses and thereby the settlement increment) and by load redistribution as some large interims support structures was removed.

However, the settlements measured showed that in case of heavy loads and stiff glacial deposits in Copenhagen and similar stiff deposits, settlements may be overestimated using traditional methods without taking stiffness increase into account.

11 CONCLUSION

The settlement assessment has shown that the stiffness increase during the project was an important factor for the settlement developed. Furthermore if the proper stiffness increase is assessed in proper and robust manner, this will provide a basis of a more optimized foundation design.

12 REFERENCES

Christensen J. L., Schjønning E. and Foged N. (1992) Comparison of clay till strength parameters using BS and Danish test procedures. NGM 92. Ålborg Vol 1. pp 75-80