Ground anchor loads measured on an excavation sheet pile wall

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ABSTRACT: Results of ground anchors load measurement are presented. A 7.5 m deep excavation was carried out for a 2 level basement building in Poland. A sheet pile wall with 1 level of ground anchors was designed to ensure the excavation stability. A number of anchors were equipped with vibrating wire load cells. The loads were measured at two locations with 3 anchors instrumented at each location. At one of the locations an intensive site traffic was observed. At the other location the area behind the wall was operated by building equipment. The differences in soil conditions at both points were moderate. The anchor loads were relatively constant during the anchor life and only slight dependence on the excavation depth and site operations behind the wall was observed. The anchor forces at the site were primarily the effect of the lock-off load. Soil conditions, excavation depth and traffic impact are secondary and they induce anchor load variation by ca 20 %.

INTRODUCTION

Due to the uncertainty in the design of retaining walls (e.g. Long et al. 2012) it is a good practice to perform a monitoring of some elements of such construction. Monitoring system can be designed to control displacements of the wall, the wall and support system behavior in terms of stress, or impact of the excavation on the adjacent buildings. Very difficult quantity to measure is the earth pressure acting on a retaining wall (DiBiaggio 1977, Dunnicliff 1993). This is probably the reason, why the researchers choose to measure rather strain/forces in structural elements. Many papers have been published regarding the measurement of the forces in the struts on the retaining walls (e.g. Powrie and Batten 2000) but only few papers contain data from the measurement of loads in ground anchors (Mayer 2001). Survey done by the author for the first in soils of medium to high strength (Kucybala and Sahajda 2011) showed that the anchors forces were smaller than assumed in the design, albeit the latter had been calculated with non-conservative assumptions. Such results were generally in line with the prop loads measured in the UK (Twine and Roscoe 1999). The results suggested the design methods of flexible anchored walls in non-soft soils are still conservative. This prompted the author to further study the problem. First results are presented in the paper.

SITE DESCRIPTION

The site is located in Poznan, a city in eastern Poland. The building was designed with a 2 level basement and 12 storeys above the ground. The underground construction is trapezoidal in the plan view and occupies total area of 4296 m^2. The excavation is located within an urban area but existing buildings are at relatively large distance from the edge of the retaining wall. The plan view of the excavation with surrounding facilities is shown in Fig. 1. The site is bordered by streets from the north, east and south. On the east side an existing hotel is located at a distance of ca 33 m from the sheet pile wall. An old brick-wall building from the 19^{th} century is adjacent to the street at the south edge of the excavation. The distance between the building wall and the retaining wall is 17 m. No construction exists to a distance of at least 30 m along other edges of the excavation. The area around the site is relatively flat with the ground level 64.80 m ASL. to 65.50 m ASL.

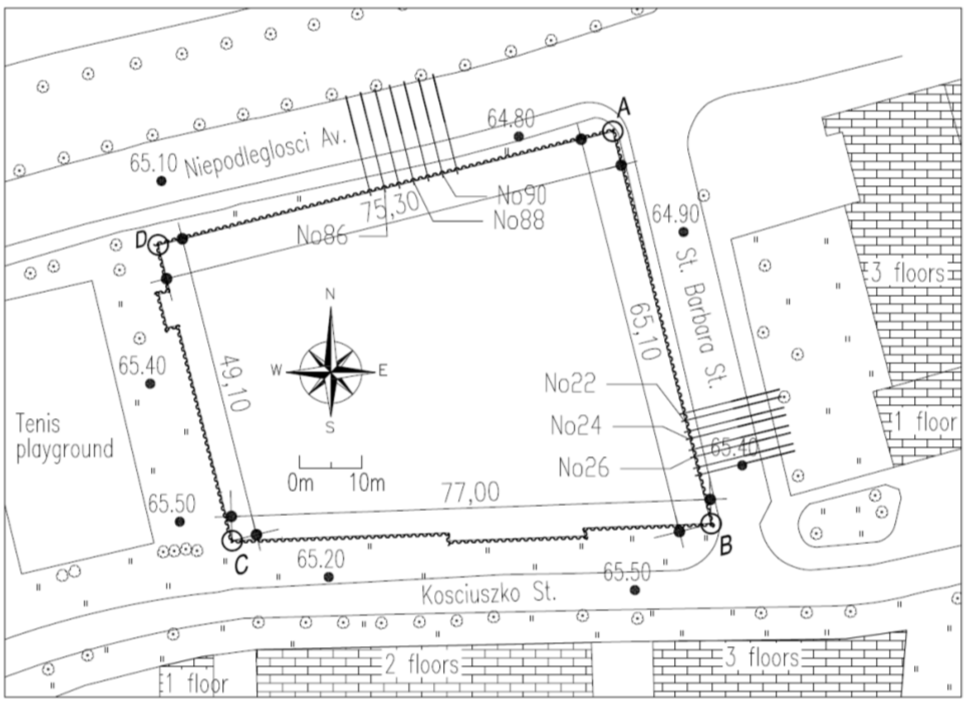

Fig. 1. Excavation with the surroundings and survey layout

The geotechnical conditions across the site are quite uniform with respect to the soil origin but slightly less uniform as regard to the thickness and strength of the layers. According to the geotechnical report a fill consisting of sands and low to middle plastic clays was found to a depth of 4÷6 m BGL. The native soils are fluvioglacial and river sands interbedded with silts and low to middle plasticity clays of glaciolacustrine origin.

The soils were explored by borings and CPTu soundings. No samples were extracted for triaxial or oedemeter tests, which is a common practice in Poland. As a result the strength, stiffness or YSR parameters can only be estimated based on the CPTs. Two of the cone tests were carried out close to the location of the instrumented anchors. Distributions of the cone resistance and friction ratio for the CPT10 from the vicinity of the first survey location are presented in Fig 2.

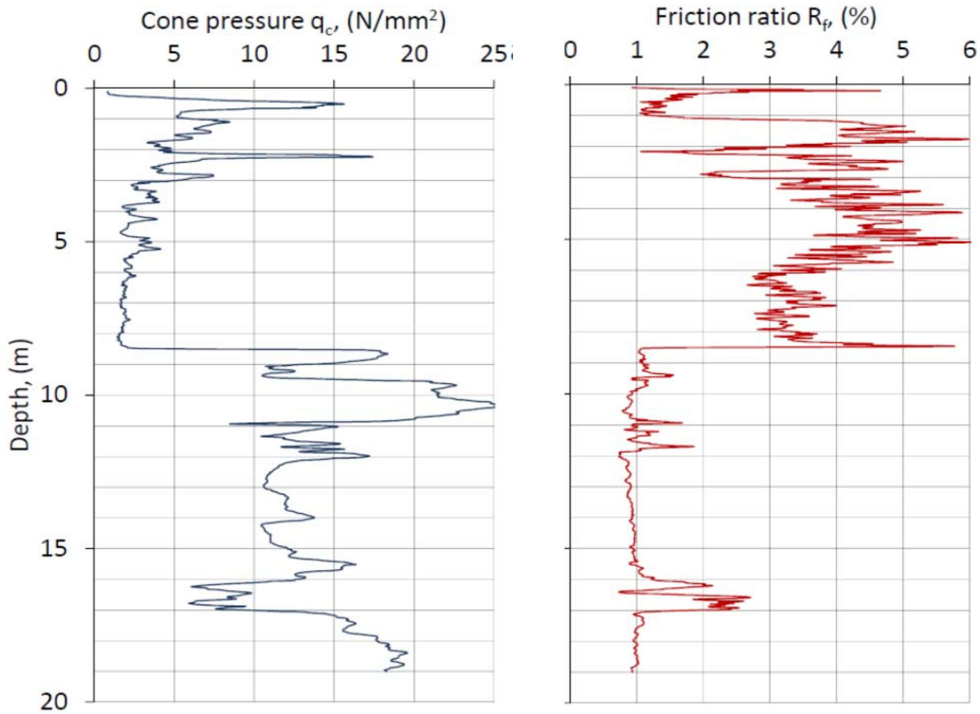

Fig. 2. Cone pressure and friction ration from CPT10

It can be seen that except for the first 1 m below the ground level (BGL) the fill is cohesive with q_c and R_f in the range 2÷3 MPa and 4÷5 % respectively indicating stiff clays, which, based on the borings, are of middle plasticity. Adopting N_{kt}=20 leads to the undrained shear strength c_u=100÷150 kPa. Parent soil below this layer can be classified as low to middle plasticity stiff to firm clay. In this case q_c=1.6÷2 MPa and N_{kt}=17 lead to the undrained shear strength c_u=90÷110 kPa. Cohesive soils are underlain by fine sands to a depth of at least 19 m i.e. 1 m below the sheet-pile wall tip. With the ground level at 65.40 m ASL and the bottom of the excavation at 57.60 m ASL the sand up to 3 m below the formation level is in a dense state with q_c=18÷25 MPa and only locally lower values. The sand below is relatively uniform to the sheet-pile tip with q_c=8÷12 MPa indicating a middle dense packing. Water level was found below the parent clay at a depth of 8.4 m BGL and stabilized at a depth of 5.1 m BGL

The excavation wall was first designed by an independent design office with the total embedment of 18 m and 2 levels of anchors. For the purpose of a bid the wall was redesigned by the author's company which is a routine practice in Poland. According to the primary design the embedment of the wall could not have been altered because of the seepage considerations. Finally a retaining wall with one anchor level was proposed. The internal face of the wall was set at 0.15 m from the concrete walls of the basement giving the total perimeter of the excavation 273 m and the area of the excavation 4337 m^2 slightly larger than that of the construction.

MEASUREMENTS DONE

First survey area lies in the vicinity of the SE excavation corner (see Fig. 1). The instrumented anchors had numbers 22, 24 and 26 with the anchors in between No 23 and No 25 uninstrumented. It was thought that such configuration would enable assessment of the influence of the successively locked anchors. The distances from the anchor No 22 and No 26 to the corner B were ca 11 m and 17 m respectively. The average distance between the anchors at the location including the uninstrumented anchors was 1.5 m. In the second survey area anchors No 86, No 88 and No 90 were instrumented with the anchors in between No 89 and No 91 uninstrumented. The distances from the anchors No 86 and No 90 to the corner A were ca 37.5 m and 28 m respectively. The average distance between the anchors at the location including the uninstrumented anchors was 2.4 m. Taking into account the depth of the excavation he=7.9 m at the first location and the distance between the most distant anchor and the excavation corner being hardly 2.2·he it cannot be probably assumed the wall worked in plane strain. In contrast, at the second location it is believed the wall is in plane strain with the he=7.3 m and the distance between the closest anchor and the excavation corner being almost 4·he. Albeit the distance 4·he could be insufficient for plane strain according to Wu et al. (2010), in their paper relatively stiff walls in soft clays were analyzed, which is surely not the case. For the measurements Geokon 4900 vibrating wire load cells were used. The accuracy of the device specified by the manufacturer is ≤1 % of the full range. The cells were 105 mm OD steel cylinders equipped with 3 strain gages each and a built-in thermistor.

All the anchors were drilled with outside diameter 133 mm rotary casing with internal auger. The auger was analogous to the CFA auger i.e. with internal hollow stem. The anchors were bored with water-flush applied trough the auger. All the anchors were double pressure-injected. Primary pressure grouting was applied trough the casing every 2 m during its removal and the pressure then applied was 1 N/mm2. After approximately 24 h all anchors were grouted by tube a manchette with the pressure 6÷7 N/mm2. Both during primary and secondary pressure grouting the grout used had the w/c=0.50 and was produced form CEMII 32.5R. In the Table 1 all basic data about the anchors are given.

The tendons of all anchors were 3 strands with 140 mm^2 cross section each, made of steel St 1670/1860. The yield and tensile capacity of the tendons were 701 kN and 781 kN respectively. The levels of the anchors, their geometric characteristics and soil conditions at both of the locations are shown in Fig. 3.

Table 1. Basic data about the survey anchors

Anch. No	Location	Drilling date	Proof test date	Tendon	Anchor design load, E_d (kN)	Lock-off load acc. to the design, P_0 (kN)	Proof load, P_p (kN)
22	Survey 1	03/01/2012	10/01/2012	3x140mm^2, St1670/ 1860	360.0	250.0	450.0
24	Survey 1	04/01/2012	10/01/2012		360.0	250.0	450.0
26	Survey 1	17/12/2011	02/01/2012		360.0	250.0	450.0
86	Survey 2	12/12/2011	19/12/2011		500.0	350.0	625.0
88	Survey 2	12/12/2011	19/12/2011		500.0	350.0	625.0
90	Survey 2	12/12/2011	19/12/2011		500.0	350.0	625.0

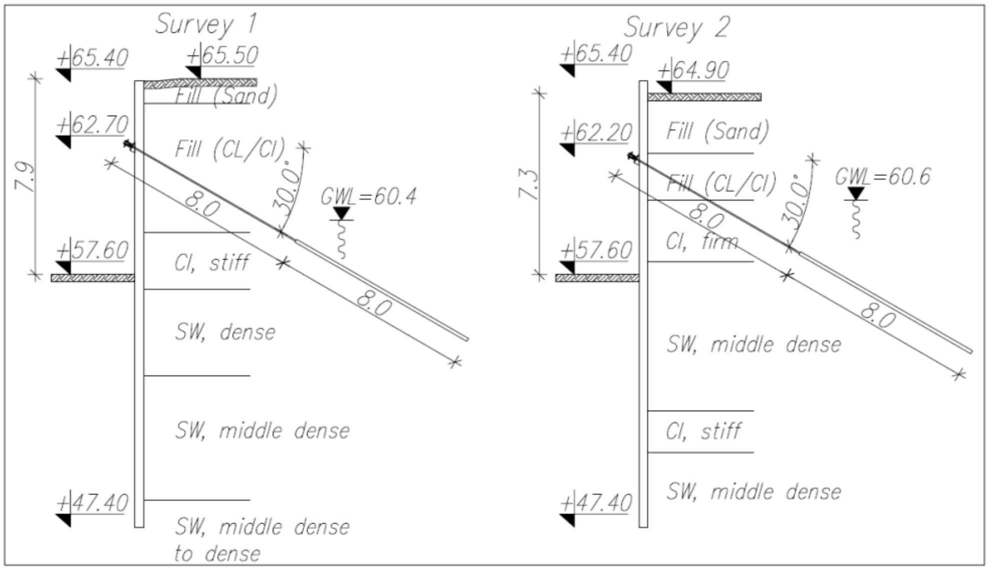

Fig. 3. Sheet pile wall cross section at survey locations

It can be seen all the anchors had total lengths of 16 m with 8 m bond length. The bond length of the anchors No 22, No 24 and No 26 (first survey location) was embedded partially within a stiff clay and partially within a dense sand. In the case of the anchors No 86, No 88 and No 90 (second survey location) the bond length was located mainly in a middle dense sand. Zero readings had been taken before the load application for the lock-off purpose and controlled after the anchors were cut-off. At the first location the anchors were locked on 10/01/2012 and at the second, on 20/12/2011. The key data about the excavation progress are given in the Table 2. Note that only the date is given, when the measurement was done and it is clear that the excavation was at the final level. However the moment when the excavation works had been completed is not known. When the author visited the site for the measurements the lean concrete had already been in place. It can be assumed the excavation works must have been completed between 10th and 20th February 2012.

Table 2. Basic data about measurement

Anchor No	Sheet pile installation	Excavate to working platform for anchors	Depth of the excavation, h_{e0} (m)	Excavate to formation level	Depth of the excavation, h_e (m)
22	02/12/2011	12/12/2011	3.7	not known, depth measured at full excavation on the 05/03/2012	7.9
24	03/12/2011	12/12/2011	3.7		7.9
26	03/12/2011	12/12/2011	3.7		7.9
86	23/11/2011	09/12/2011	3.7		7.3
88	23/11/2011	09/12/2011	3.7		7.3
90	24/11/2011	09/12/2011	3.7		7.3

RESULTS

Because of a risk of a theft or damage during the site activities the loggers were not left in place. The measurements of the anchor loads were therefore taken at some moments when the author visited the site. These moments were chosen with reference to important excavation phases. The results of the survey are shown in Table 3 for the anchors No 22 to No 26.

Table 3. Loads measured in the anchors No 22, No 24 and No 26

Date, dd/mm/yy	Time	Anchor force, E_m (kN)			Cell temperatue, T_m (^oC)			Comments
		No 22	No 24	No 26	No 22	No 24	No 26	
10/01/2012	14:40	0.0	0.0	0.0	7.1	5.5	5.6	Excavation 3.7 m
10/01/2012	14:50	262.0	0.0	0.0	6.0	5.2	5.2	Lock No22, excav. 3.7 m
10/01/2012	15:15	260.0	298.0	0.0	2.9	4.9	4.6	Lock No24, excav. 3.7 m
10/01/2012	15:55	257.0	292.0	196.0	3.7	4.1	-	Lock No26, excav. 3.7 m
11/01/2012	11:00	217.0	245.0	161.0	5.2	5.4	5.5	Last read., excav. 3.7 m
05/03/2012	14:00	255.0	290.0	197.0	9.4	7.9	6.0	Full excavation 7.9 m
10/05/2012	16:30	-7.0	-3.0	-2.0	32.4	32.5	32.3	Final zero

It should be mentioned that there is no measurement between 10/01/2012, 15:55 and 11/01/2012, 11:00. The reason is a battery fail, which appeared on the next day. Within this period neighbor uninstrumented anchors were locked and the loss of data is unfortunate.

First relation to be observed is that between the design and actual lock-off load. In practice the actual lock-off load is rarely measured and can be evaluated only approximately. In the authors' company for any anchor system the slip between the tendon and the wedge after the force release is measured in mm at the beginning of the contract. After the proof test, the tendon is tensioned to a theoretical value of the lock-off and then over tensioned by the value of the aforementioned slip. At such displacement the wedges are assembled and the force released. The theoretical value of the lock-off force for the anchors No 22, No 24 and No 26 is 250 kN. The loads measured on 11/01/2012, i.e. ca 19 h after the lock-off, are adopted as the representative values and they are 217 kN, 245 kN and 161 kN in the anchors No 22, No 24 and No 26 respectively. Though the choice of the forces at this particular moment is debatable for the reason of possible impact of the other anchors tensioning, this is the actual load in the anchors before the commencement of further earth works. It can be seen that the measured forces are generally lower than theoretically specified with the difference of as much as 89 kN in the anchor No 26. There is no clear trend between the difference and the sequence of the anchors tensioning. It should however be pointed out that the tensioning of the neighbor anchors had some impact on the load measured in previously locked anchor.

The next measurement was done 3 months later on 05/03/2012 when the excavation was at the formation level. An increase in the load of 38 kN, 45 kN and 36 kN was observed for the anchors No 22, No 24 and No 26 respectively and the values are to some extent related to the load after the lock-off i.e. on 11/01/2012. This increase expressed as a percentage of the start load is in average 19 %. Taking into account the fact that the lock-off load was specified to be 0.70

of the design anchor load, it can be said that the design assumption with respect to the increase was fulfilled with reasonable margin. It should however be noted that the maximum anchor loads measured after full excavation were 255 kN, 291 kN and 197 kN which accounts for 71 %, 81 % and 55 % of the design load in the anchors No 22, No 24 and No 26 respectively. During the site visiting the author observed that truck concrete mixers were parking behind the retaining wall at the survey location. An average surface load calculated from this observation seems to be reasonably close to the value of 10 kN/m^2 adopted in the design. The results of the survey are shown in Table 4 for the anchors No 86 to No 90. If the measured values are compared to the characteristic values instead of the design it does not change the relation between the measured and the calculated load, since the maximum design values were obtained in the exceptional load case, when the partial factors are set to 1.0.

Table 4. Loads measured in the anchors No 86, No 88 and No 90

Date, dd/mm/yy	Time	Anchor force, E_m (kN)			Cell temperatue, T_m (^oC)			Comments
		No 86	No 88	No 90	No 86	No 88	No 90	
20/12/2011	14:00	0.0	0.0	0.0	5.8	6.9	5.3	Excavation 3.7 m
20/12/2011	14:15	311.0	0.0	0.0	4.5	6.4	5.6	Lock No86, excav. 3.7 m
20/12/2011	14:25	310.0	284.0	0.0	4.2	5.4	4.9	Lock No88, excav. 3.7 m
20/12/2011	14:55	309.0	262.0	356.0	3.5	3.6	3.7	Lock No90, excav. 3.7 m
20/12/2011	18:20	305.0	246.0	329.0	-0.2	-0.2	-0.7	Lock No91, excav. 3.7 m
20/12/2011	19:00	305.0	245.0	328.0	-0.6	-0.5	-0.9	Lock No89, excav. 3.7 m
21/12/2011	19:00	302.0	239.0	323.0	-0.7	-0.4	-0.7	24h later, excav. 3.7 m
22/12/2011	11:20	297.0	238.0	321.0	-1.1	-0.6	-1.0	Excav. 3.7 m
22/12/2011	12:20	296.0	236.0	321.0	-0.3	-0.2	-0.5	Lock No87, excav. 3.7 m
23/12/2011	10:20	296.0	234.0	320.0	0.5	0.7	0.5	Last read., excav. 3.7 m
05/03/2012	17:00	338.0	269.0	384.0	9.5	8.3	7.8	Full excavation 7.3 m
10/05/2012	16:59	-4.0	-5.0	-6.0	32.3	33.0	30.9	Final zero

For the anchors No 86, No 88 and No 90 the theoretical lock-off force is 350 kN. The loads measured on 21/12/2011, i.e. ca 24 h after the lock-off, are adopted as the representative values and they are 302 kN, 239 kN and 323 kN for the anchors No 86, No 88 and No 90 respectively. It can be seen that the measured forces are lower than theoretically specified and the largest difference is 111 kN in the anchor No 88. No trend is observed between the difference and the sequence of force application. The tensioning of neighbor anchors had some impact on the load measured in previously locked anchor with the largest reduction of the load 27 kN in the anchor No 90 during the tensioning of the anchor No 91. After the last anchor had been locked reduction in the anchor load during the period of 22 h was negligible.

The measurement at the full excavation was done after almost 4 months on 05/03/2012. The forces increased by 36 kN, 30 kN and 61 kN for the anchors No 86, No 88 and No 90 respectively and the increase is related to the representative lock-off forces on 21/12/2011. The increase expressed as a percentage of the start load is in average 14 %. The maximum anchor loads measured after the full excavation were 338 kN, 269 kN and 384 kN which accounts for 68 %, 54 % and 77 % of the design load for the anchors No 86, No 88 and No 90 respectively. The survey location was observed to be operated by the concrete pumps positioned exactly behind the wall, which was not allowed in the design in such extreme version. Nonetheless it can be seen, that this practice of the general contractor didn't lead to any overstressing of the anchors.

DISCUSSION OF THE RESULTS

At the design stage the anchor forces were calculated based on the classical method i.e. earth/water pressure balance. For all cohesive soils fully drained conditions were assumed which usually leads to higher wall and anchor forces in stiff and firm soils than in undrained conditions. Water table was assumed at the retained side according to the results of the geotechnical investigation and at the excavated side 1m below the formation level. The latter is slightly non conservative in the classical method since the lowering of the water in the excavation leads to a reduction in the calculated wall and anchor forces. At the retained side active earth pressure was assumed with a rectangular distribution, which is generally non conservative with respect to the anchor forces when compared to the German recommendations (see Weissenbach 2003). At both sides of the wall the friction between the wall and the soil was adopted as 0.67ϕ' and both the active and passive earth pressure were calculated on the assumption of non planar slip surfaces (DIN 4085 2007). The calculation was carried out according to partial factor approach based on the Load Case 2 according to DIN 1054 (2005) with 10 kN/m^2 uniform surface load at the retained side. For the exceptional case the values of partial factors from Load Case 3 according to DIN 1054 (2005) were adopted with 10 kN/m^2 uniform surface load and additional strip load of 40 kN/m^2 in the area 2 m behind the wall. The values of the soil parameters assumed in the design and for the purpose of additional analysis are compiled in Table 5 below.

Table 5. Soil parameters assumed in the design and in further analysis

Soil	Bulk density γ / γ', (kN/m^3)	Effective angle of shearing resistance, ϕ', (deg)	Effective cohesion intercept, c', (deg)	Undrained shear stregth, c_u, (kN/m^2)
Fill	19.0 / 9.0	30	0	80,0
Firm clay	20.0 / 10.0	26	5	100,0
Stiff Clay	21.0 / 11.0	26	15	120,0
Sand dense	18.0 / 9.5	37	0	-
Sand middle dense	17.5 / 9.0	36	0	-

As it was shown, the measured anchor forces were in average only 68 % of the values calculated based on parameters assumed in the design. Other calculation was carried out to check different possible situation. For this purpose undrained conditions in all clay layers including the fill were assumed. Adopting in such a case water under pressure below the clay without changing other assumptions leads to forces 161 KN in the anchors No 22, No 24 and No 26 and 290 KN in the anchors No 86, No 88 and No 90. The measured values are 154 % and 114 % of the calculated values at Survey 1 and Survey 2 locations respectively. This result is extremely non conservative. Of course other assumptions about the drained vs.

undrained conditions and about the detailed earth pressure distribution can be made. However in the author's opinion this type of back-analysis makes only sense with larger statistical database and is a kind of "numerical engineering" rather than analysis when applied to a singular case. Therefore more data is needed from real case studies. At the present stage it can only be surmised that the reason for the anchor forces smaller than calculated in the design can be too low values of drained parameters or conditions being not fully drained. Based on the results obtained from the undrained case it can be said that this kind of analysis in stiff and firm clays leads to non conservative values of the anchor forces and should not be applied in the design.

With respect to the interpretation of the data it should be mentioned, that the load cell manufacturer states that the temperature increase lead to a reduction of the cell reading of -1.5 unit per ^{o}C. In the case analyzed the range of the temperature measured in the cells was -2 ^{o}C to +9 ^{o}C except for the final zero reading. The zero reading after the anchors cut-off with respect to the zero reading before the anchor lock-off was in average 34 units with the temperature change of +26 ^{o}C. This gives a correction factor -1.3 unit per ^{o}C, which is very close to the manufacturers specification. During the whole interpretation the author applied the value of a correction factor -1.3 unit per ^{o}C. It must be stated that this procedure could be insufficient for similar measurements if the temperature change were larger. If the ambient temperature rises, the temperature of the anchor tendon rises too and it comes to its loosening. This itself leads to an actual reduction of the anchor load. In the case presented the full range of the temperature fluctuations was however not very large and the results are believed to be accurate.

CONCLUSIONS

Anchor forces were measured in a full scale with vibration wire load cells. The measurement was carried out on a sheet pile wall supporting an excavation in mixed clay/sand soil. The forces measured were in average 68 % of the values calculated in the design with the assumption of fully drained conditions in clay. Calculation made with undrained clay led in turn to calculated forces significantly smaller than measured. Since this lies on the unsafe side, it is not recommended to assume undrained conditions in firm and stiff clay. The actual anchor forces were shown to depend more on the value of the lock-off load than e.g. surface load at the retained side. The observation showed also that uniform rule with respect to the application of the lock-off load can lead to significant differences in the load the anchors are actually locked.

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