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Experience from site investigations in glacial soils of the Alpine region

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ABSTRACT: Glacial soils are quite heterogeneous and their composition and properties vary substantially over short distances. As coarse grained soils may be present within fine-grained soils the application of standard equipment for the investigation of uniform fine-grained soils may be limited. The deposition environment and the subsequent stress history of the soil has a significant effect on soil properties. In glacial terrain within a mountain range the stress history is highly variable. Thus a reliable methodology of characterization of the subsoil is necessary. Experience from two case histories, small to medium size projects from Switzerland, is presented. Different types of in-situ investigations, like cone penetration tests (CPTU), Marchetti Dilatometer tests (DMT), pressure meter tests (PMT), standard penetration tests (SPT) have been used. Also laboratory tests were carried out and their results compared to the in-situ tests. A combination of different techniques proved to be useful for a reliable site characterization.

1 INTRODUCTION

Experience from site investigations in glacial soils in Switzerland is described and compared. Switzerland is very densely populated and poorer ground is used for construction of buildings, road works and infrastructure. In one case a 340 m long tunnel for separation of a grade crossing between a commuter railway and a major road had to be constructed in glacial sands and silts and postglacial sensitive clay, in Schönbühl, in the northern suburbs of Bern.

The second case is a building for a Health Center in Lugano, Switzerland, an addition to the General Hospital (Ospedale Civico) built in 1969.

In-situ investigations have been applied for other projects (Steiner et al., 1992; Steiner, 1994) where stability and deformation problems had to be solved. There in-situ investigations with DMT and CPTU proved extremely valuable in characterizing soil properties and subsequent analyses. This experience will be also used in the evaluation of site characterization methods.

2 CASE HISTORY SCHOENBUEHL

This site is located some 10 kilometers north of Bern in area where, during the ice-ages two glaciers had conflued. During their retreat the glaciers had left sand and silt deposits overlying ground moraine.

During the postglacial phase sensitive clays and peat were deposited in the remaining lake. During earlier centuries when building roads, railways and villages this unfavourable ground was avoided by following the margins of the swamp land. With the increasing population density property with good foundation conditions became scarce. In the 1950's an underpass was constructed under the main east-west railway, substituting two grade crossings. The access road had to be built on a thick, close to 10 meters, layer of sensitive clay. For acceleration of the consolidation of an embankment of 2 - 3 meters height on clay, sand drains were required. Under an additional temporary surcharge of 3 meters 1 to 1.8 meters of total settlement were monitored.

At the beginning of the 20th century a narrow gauge railway was constructed which over time muted to a commuter railway. During the construction of this railway embankments experienced large scale instabilities and settlements. In the 1960's the construction of the motorway N1 led to a temporary reduction of transit road traffic. However, a large shopping center was built in area of swamp land, on a large pile foundation, thus traffic increased again. It became necessary to separate a grade crossing of the commuter railway with the state road. Initially, for geotechnical reasons an underpass should be avoided and overpasses were considered. However, their appearance was judged to be detrimental to the town and bridges were rejected. Finally, the conclusion was reached that a 340 meter long cut-and-cover tunnel was the best solution both technically and environmentally. Subsoil investigations already existed, which were, however, mostly executed to insufficient depth. New ground investigations for geotechnics and hydrogeology were executed in 1990. The 340 meter long cut-and-cover tunnel has slurry walls reaching to a depth of approximately 17 meters from ground surface and might form a hydrogeologic barrier, resulting in secondary settlement effects. This site investigation included 11 drill holes in soil between 15 and 30 meters depth. Also 12 cone penetration tests with pore pressure measurements (CPTU) were carried out to depths from 12 to 30 meters depth. The cone penetration tests had to be terminated due to excessive tip resistance, once they had reached the ground moraine. The project was delayed several years. In 1994 five more borings were drilled also with in-situ tests. The main scope was hydrogeologic monitoring: filter tips were placed in sealed zones. These could be equipped with electric pore pressure transducers at a „docking station", thus giving a closed system for measuring pore pressure allowing for automatic registration .

2.1 *Main results from site investigation*

One third of the tunnel lies in sensitive postglacial clay and the remaining two-thirds were in ice-marginal sands and silts. The groundwater table was close to the ground surface, only in 1 to 3 meters depth. In deeper zones, 15 to 20 meters depth, the pressure was slightly higher. The combination of drill holes and cone penetration tests allowed a comparison of in-situ measurements with laboratory tests. As glacial deposits are also heterogeneous within the same stratigraphic unit the individual strata will vary quite erratically. With CPTU these variations of the layers in the range of decimeters was easily to determine. The contact between the sensitive clay and the ice-marginal deposits was rather steep. The sands were deposited by currents in a lake (delta deposits) leaving a rather irregular surface of the sand. In the immediate vicinty of the slurry walls the sensitive clay had a thickness of 14 meters, yet only 10 meters further to the north the sensitive clay reached to a depth of 19 meters. From the construction of the new town center, 1986-1988, to the north of the eastern end of the tunnel, it was known that the depth of the sensitive clay might reach to 25 meters depth. The piles, driven to the ground moraine with tip bearing, had introduced a disturbance into the sensitive clay reducing the undrained shear strength.

2.2 *Characterization of sands and silts.*

Cone penetration test CPTU-6 was carried near the center of the tunnel, with predominantly sandy to silty soil as was confirmed by a parallel boring. In Fig. 1 the strength properties of the subsoil based on the interpretation of normalized tip resistance NQ_T versus normalized pore pressure B_Q is presented, corresponding to the method proposed by Robertson (1990). Also shown are the interpretation of strength using corrected tip resistance Q_T versus friction ratio F_R according to Robertson and Campanella (1983). Much more strata were classified as cohesive with this earlier method. The computed undrained shear strength are extremely high (C_u = 500 - 600 kPa). Field logging of the cores and a sample classified in the laboratory clearly indicated a silt (ML). The classification with normalized tip resistance and pore pressure ratio indicated less cohesive sections with lower undrained shear strength.

This experience illustrates the problems associated with silty soils which behave as partly draining during cone penetration. In any case the large undrained shear strength obtained are judged to be non-reliable values. Silt in glacial area is mostly rock powder or rock flour, a granular material. The friction angles computed in Fig. 1 with a range from 32° to 43°, however, appear reasonable.

With the DMT the compressibility of the sands was estimated and compared favorably to values back calculated from earlier settlement observations in the area of the underpass constructed in the 1950's.

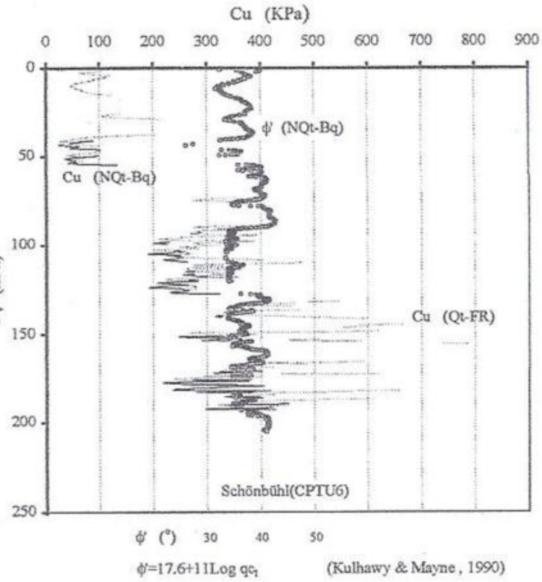


Fig. 1 Undrained shear strength and friction angle

2.3 *Characterization of postglacial clays*

Cone penetration test CPTU 10 was mostly located in postglacial sensitive clay. A parallel boring with DMT measurements was also carried out and in 1994 undisturbed samples were taken from an additional boring. The interpretation of the undrained shear strength for N_k = 14 with the Q_T versus F_R classification is shown on Fig. 2. Also shown are the undrained shear strength determined in the boring by the flat dilatometer (DMT).

The undrained shear strength appears rather constant with depth and shows a minimum in the center of the layer at 12 meters depth. In contrast two peaks with c_u = 40 kPa are present at 10 and 15 meters depth. For further interpretation one has to take

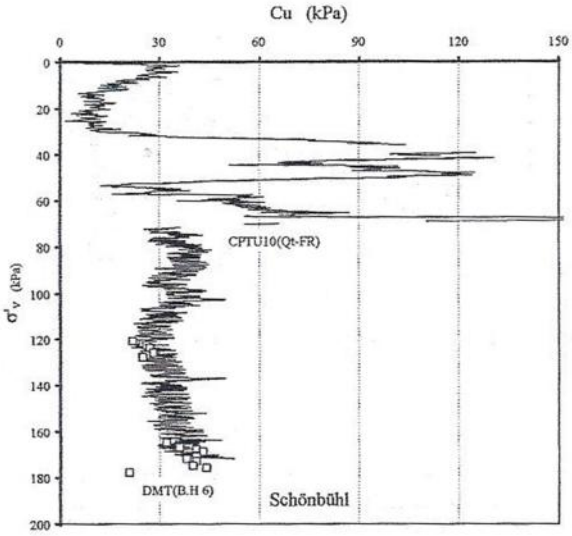


Fig.2 Comparison of undrained shear strength

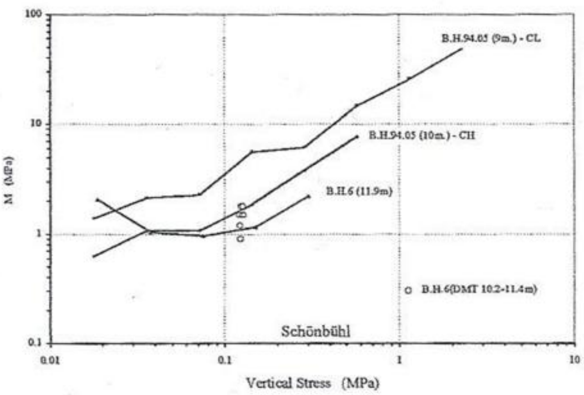


Fig.3 Comparison of moduli from DMT with oedometer tests

into consideration construction activity in the vicinity of the site. Piles had been driven several (3 to 4) years prior to carrying out the cone penetration tests.

The reduction of the shear strength is most likely the effect of disturbance caused by the pile driving, otherwise one would expect an increase in undrained shear strength with depth. The peaks may have been caused by zones of larger permeability resulting in more rapid consolidation.

The results of the undrained shear strength with a mean C_u = 30 kPa and a standard deviation of 5 kPa analysis were used for a probabilistic analysis of the stability of slurry trenches of varying width (Steiner and Rieder, 1997) which proved very useful.

Moduli determined from DMT were compared with moduli from oedometer tests (Fig. 3) and good agreement was found.

3. CASE HISTORY HEALTH CENTER

During 1996 site investigations were carried out for an addition to the General Hospital at Lugano, southern Switzerland. The Health Center has a footprint of 50 by 50 meters. From the construction of the General Hospital the results of the site investigation were available. The site is located on a valley slope and the existing buildings are partly founded on bedrock (mica-schist) and the remaining sections are founded on large diameter bored piles in order to achieve homogeneous foundation behaviour.

Four core borings with continuos sampling and a core diameter d = 145 mm were executed. Localized in-situ tests were carried out, namely 24 SPT tests, 13 DMT (Flat Dilatometer tests), 5 PMT (pressure meter tests) in these borings and were supplemented by a single cone penetration test (CPT-U) in an area where a truck had access. The CPTU had to be executed in two phases, since from 4 to 8 meters depth dense gravely ground moraine was present which gave too much resistance to the penetrometer. Also the DMT device could only pushed ahead a limited distance from the bottom of the borehole permitting only two readings. This was most likely caused by two factors: (1) the dense nature of the overconsolidated soils and (2) the limited thrust of the light-weight drill rig which had to be used due to the limited accessibility.

The short period available for the site investigation did not permit to take undisturbed samples and carry out a detailed laboratory investigation. However, data from the site investigation (50 mm lab test from 65 mm push-in tube sample) from 1969 were available and comparisons were drawn as well as laboratory results carried out on 70 mm samples carved from the 145 mm diameter core in 1996.

3.1 *Stratigraphy*

The stratigraphy can be summarized as:

(1) Overburden with a maximum thickness of 4 meters, fill of various origin, which will be mostly removed for the construction.

(2) Glacial and glaciolacustrine deposits (Würm) consisting of gravel, often rather coarse embedded in a matrix of silty or sandy clay extremely variable with boundaries changing erratically. Included are also sandy silts and clayey silts often with varves. This layer is the weakest link of the subsoil and was investigated in detail.

(3) Bedrock, Mica schist, 10 to 15 meters below the future foundation level.

The hydrogeologic conditions can be characterized that no continuos water table is present. Water percolates on impervious layers like the surface of bedrock and interbedded clay layers.

3.2 *Interpretation of in-situ tests.*

The interpretation will focus on the properties of the

Tab.1 Laboratory soil tests (Part 1)

HEALTH CENTER - LUGANO (1996) GENERAL HOSPITAL - LUGANO(1969)

Boring	Depth	Specific Gravity	Unit Weight	Water Cont.	Void Ratio	Degree of Saturation	Atterberg Limits			Activity	Elevation
		γ_s	γ	W_N	e	S	W_L	W_P	I_P		
nr.	m	kN/m³	kN/m³	%		%	%	%	%		m.s.m.
G	2.80	26.30	20.8	16	0.47	90.2					365.24
C	15.25		21.0	18.4							352.56
H	17.70	27.30	20.0	25	0.71	96.6	27	17.8	9.2	0.61	352.48
B	12.35		20.8	21.5							352.45
C	17.50		20.8	19.3							350.31
C	18.90	27.60	20.0	21.1	0.67	86.8					348.91
31(1996)	12.50	27.60	19.7	18	0.60	80.5	26.8	26.4	0.4	0.02	345.38
C	23.55		21.5	9.9							344.26
33(1996)	10.60	27.62	19.3	20	0.66	80.5	27	25.4	1.6	0.18	341.88
13	9.40	27.70		13.7							339.85
16	10.60	27.20	20.2	17.5	0.58	81.8					339.59
13	9.80	27.80	19.9	19.3	0.67	80.5	39.3	30	9.3	1.86	339.45
13	10.20	27.70		13			22.3	17.6	4.7	1.57	339.05
13	10.55	27.30	20.5	18.2	0.57	86.6					338.70
13	10.70	27.10		17.8			21.6	16.4	5.2	1.73	338.55
16	12.00	27.90	21.2	9.6	0.44	60.6	38	14	24	2.40	338.19
16	13.05		19.4	16.7							337.14
16	13.80		19.4	20.4							336.39
F	5.70	26.90	20.6	21.5	0.59	98.6	28	16.8	11.2	0.49	333.74
E	11.85	27.50	18.9	16.1	0.69	64.2					332.65
E	12.10	27.40	18.7	14.9	0.68	59.7					332.40
E	12.60	27.80	19.7	25.4	0.77	91.8					331.90
E	13.25	27.70	20.4	22.3	0.66	93.5					331.25
F	8.70		21.1	21.3							330.74
E	13.80	27.50	19.8	28.4	0.78	99.7					330.70
E	14.65	27.70	19.6	26	0.78	92.3					329.85
F	9.70	27.20	21.3	17.7	0.50	95.7					329.74
20	17.4		21.2	15.2							328.99

Tab.2 Laboratory soil tests (Part 2)

Boring	Sample	Depth	Particle Size Distribution				USCS	Eff. Press.	Constr. Modulus	Direct Shear		Elevation
			Clay	Silt	Sand	Gravel		σ'v	M(σ'v)	C	φ'	
nr.	nr.	m	%	%	%	%		kPa	Mpa	kPa	[°]	m.s.m.
G	151	2.80	2	29	47	22	SM-ML	57	2.5	5	35	365.24
C	140	15.25						310				352.56
H	159	17.70	15	62	23		CL(CL-ML)	359				352.48
B	139	12.35						251				352.45
C	141	17.50						355				350.31
C	142	18.90	9	43	22	26	SC-CL	384		-	34	348.91
31(1996)	IGC	12.50	22	61	15	2	ML	254	12			345.38
C	146	23.55						478				344.26
33(1996)	IGC	10.60	9	34	42	15	ML	214	11	43	41	341.88
13	123	9.40	11	51	25	13	CL-ML	191				339.85
16	119	10.60	8	56	33	3	CL-ML	215				339.59
13	136	9.80	5	73	17	5	ML	199	9			339.45
13	137	10.20	3	41	15	41	GM-ML(CL-ML)	207				339.05
13	124	10.55	6	59	35		ML(SM-ML)	214				338.70
13	135	10.70	3	47	40	10	SM-ML(CL-ML)	217				338.55
16	120	12.00	10	65	19	6	CL-ML(CL)	244	7			338.19
16	121	13.05						265				337.14
16	122	13.80						280				336.39
F	143	5.70	23	68	9		CL(CL-ML)	116	6	10	27	333.74
E	125	11.85	7	54	34	5	ML(SM-ML)	241				332.65
E	126	12.10	6	39	54	1	ML(SM-ML)	246				332.40
E	138	12.60	1	74	25		ML	256	14			331.90
E	127	13.25	6	67	27		ML	269				331.25
F	144	8.70						177				330.74
E	128	13.80	3	88	9		ML	280				330.70
E	129	14.65	6	57	37		ML(SM-ML)	297		-	35	329.85
F	145	9.70	11	65	13	11	CL-ML	197	14			329.74
20	130	17.4						353				328.99

layer of glacial and glaciolacustrine deposits and the comparison of in-situ measurements of soil compressibility and shear strength with laboratory results (Tab. 1 and 2).

Compressibility (Oedometer moduli M) from lab samples are shown on Fig. 4 and moduli obtained from different procedures are shown on Fig. 5.

With regard to the compressibility of these soils the following conclusions can be drawn:

(1) no distinction can be made for different types of sampling procedures whether the crude 1969 core samples or the larger 1996 „undisturbed" core samples.

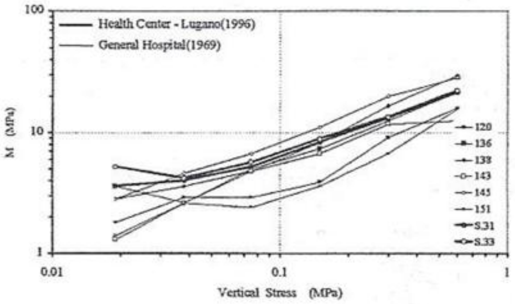
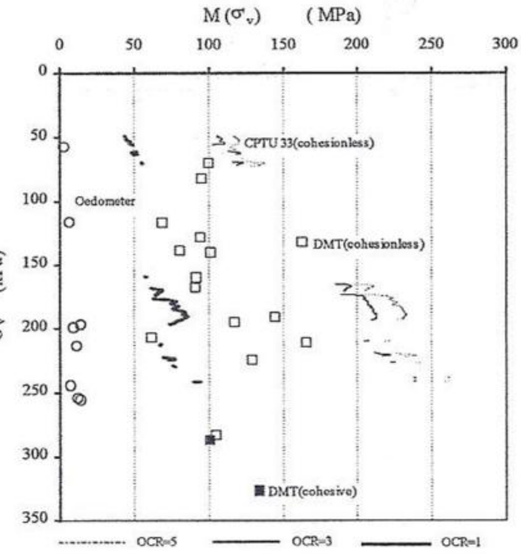


Fig.4 Comparison of moduli from Oedometers



$D_r^2=(q_c/p_a)\,/\,(305\ OCR^{0.18}(\sigma'_v/p_a)^{0.5})$ (Kulhawy & Mayne, 1990)

$M_{nc}=q_c\,10^{\,(1.09-0.0075\,D_r)}$ (Kulhawy & Mayne, 1990)

$M_{oc}=q_c\,10^{\,(1.78-0.0122\,D_r)}$ (Kulhawy & Mayne, 1990)

Fig.5 Comparison of moduli

(2) The modulus of compressibility M from oedometer tests at overburden stress are below the true values. The samples from overconsolidated sandy clay with gravel inclusions probably always suffer extensive sample disturbance, such that „undisturbed" samples are of little value.

(3) The maximum overconsolidation ratio estimated from the 1996 laboratory test is OCR = 5 and from dilatometer test it is OCR = 5.5. The deformation properties obtained by DMT suffer in quality from the difficulty of penetration of the blade. It may also be possible that the drilling introduced a major disturbance ahead of the core bit. The moduli estimated from DMT are only slightly above those estimated from CPTU with the original formula by Kulhawy & Mayne (1990) assuming non-cohesive soil and OCR = 1.

(4) In Fig. 6 modified formulae have been applied for PMT, CPTU and SPT in order to compare

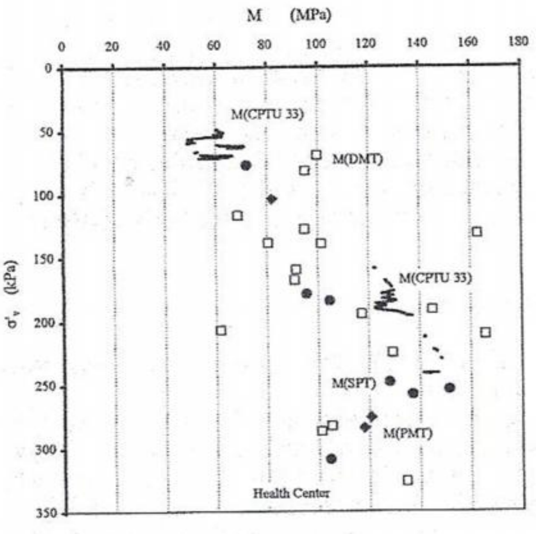


Fig. 6 Comparison of modified relations for moduli

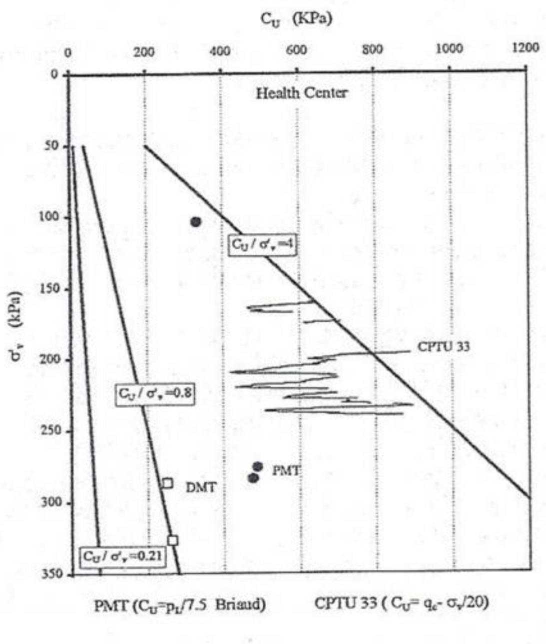


Fig.7 Comparison of undrained shear strength and OCR evaluation

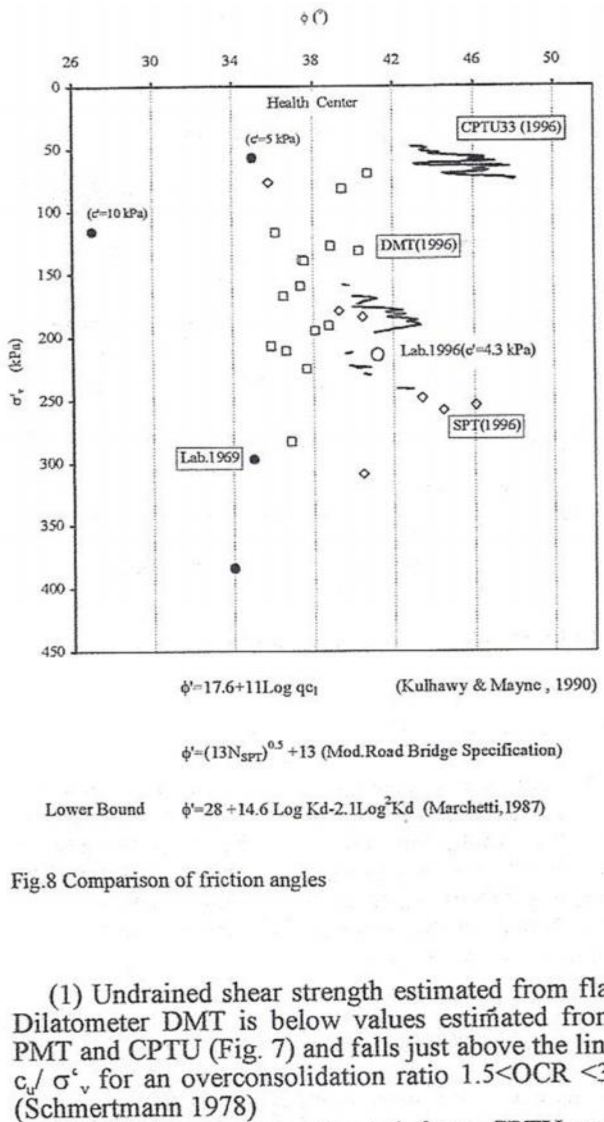


Fig.8 Comparison of friction angles

moduli. Assuming the modulus M from DMT as reference, the following relations were deduced:

M (PMT) $= 33\ p_L$

M (CPTU) $= q_c\ 10^{(1.78 - 0.016\ Dr)}$, (Dr for OCR = 5)

M (SPT) $= 1.8$ N

3.3 *Shear strength of soil*

The results of shear strength data obtained from different methods are shown on Figs 7 and 8.

(1) Undrained shear strength estimated from flat Dilatometer DMT is below values estimated from PMT and CPTU (Fig. 7) and falls just above the line $c_u/\ \sigma'_v$ for an overconsolidation ratio $1.5<OCR<3$. (Schmertmann 1978)

(2) Friction angles estimated from CPTU and DMT are substantially above laboratory values determined on undisturbed samples from the earlier investigation for the General Hospital in 1969. However, the strength determined in the laboratory on a sample carved from a 145 mm core is substantially higher corresponding to the mean of the in-situ values (Fig. 8).

3.4 *Classification of soils*

The comparison of classified soils on the soil chart (Fig.9) shows that different techniques are necessary to select the proper classification procedure. Here the classification with CPTU (Qt-FR correlation) give obviously better agreement whit the DMT classification for these overconsolidated partially saturated, than the Qt-Bq . This classification is also in agreement with the laboratory tests (Tabs 1 and 2).

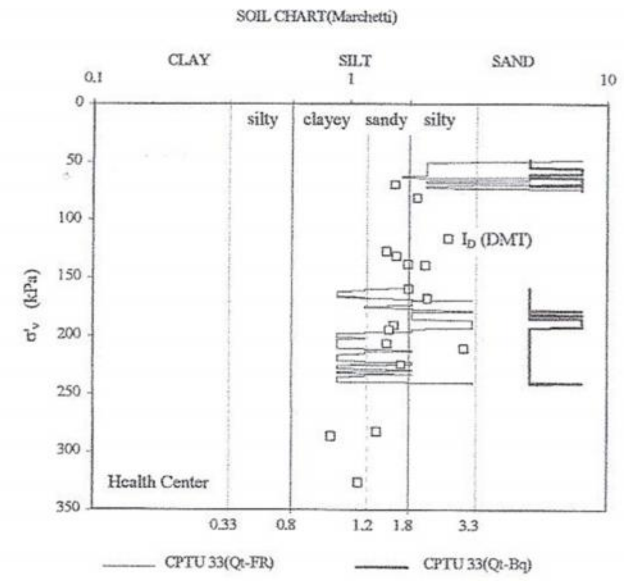


Fig.9 Comparison of classification with DMT and CPTU

3. 5. *Conclusions for construction*

The new site investigation has indicated that the foundation might be constructed on a slab in an excavation supported with tiebacks, thus the choice was left to the design engineer and contractor. The winning bidder chose large diameter (0.9 - 1.2 m) piles based on structural consideration, resulting in a more economic solution.

4.CONCLUSIONS

Glacial soils are quite heterogeneous and consist of coarse and fine-grained, cohesive and non-cohesive, layers. In coarse grained soils in-situ tests (CPT-U, DMT) are difficult to carry out, due to the presence of gravel and stones which may damage instruments.

In fine-grained soils CPTU, DMT provide reliable data with regard to deformability and strength for normally to slightly overconsolidated soils. For strongly overconsolidated soils the in-situ measured values with DMT appear to be below the true values, probably due to disturbance of the ground.

In non-cohesive soils DMT and CPTU are essentially the only tests that provide a profile and realistic information on deformation and strength characteristics of these soils. It is practically impossible to extract undisturbed samples.

In fine-grained non cohesive soils, mostly silts, the interpretation of strength properties is difficult and ambiguous. The correlation based on NQ_T vs. Bq appears to give more reliable results. During cone penetration partial drainage may occur. The techniques for interpretation in partially draining, silty soils needs improvement.

From the experience presented the following is evident: (1) Site characterization in heterogeneous glacial soils cannot be based on a single type of in-situ investigation. CPT has to be calibrated on DMT or PMT measurements. (2) In overconsolidated soils sample disturbance may be substantial.

The in-situ methods provide profiles of soil properties and the variability is shown. These data may be used as input to probabilistic methods of design.

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