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Thrust Evolution on the Shoring of Two Large Excavations in Geneva

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SYNOPSIS This study presents the results of thrust measures on the shoring of two large excavations, located in downtown Geneva, in fine saturated soils of low capacity and great thickness. Loads were measured by regular extensometer readings, completed in one excavation by three daily readings on VSL load cells. Althoug the obtained measurements seem puzzling at first sight, their analysis enable to draw some useful conclusions for the design of deep retaining structures.

INTRODUCTION

The evaluation of "in situ" measurements on thoroughly instrumented field cases, the geotechnical situation of which is well defined, forms one of the most fruitful line of study for better prevision of actual field performances, better design and for the improvement of the existing state of our knowledge.

This paper will focus on the thrust evolution on the shoring, the problems of deformations of the excavation and around it having already been treated. (Dysli et al. 1979, 1982).

The studied excavations are located in downtown Geneva, close to the lake, in the same geological environment, characterized by fine saturated soils of low bearing capacity, high compressibility and great depth, which the latest glaciation (Würm), during the sequences of its retreat, deposited on a molassic bedrock.

These excavations are both laterally supported by slurry walls, embedded beetween 6 and 7 meters. The main design concepts of these two deep retaining structures are quite different and will not be treated here, having already been presented. (Fontana, 1981).

INSTRUMENTATION

Loads acting in the H-Beams struts were measured on two ways :

- with a Huggenberger extensometer, 25 cm long, with a precision reading of one micron (10^{-3} mm).
- with D150 VSL load cells; manometric reading of pressures with a maximal error of 5 %.

At the time of readings, H-Beams temperature were measured by a contact thermometer with a precision of 0.1°C.

CONFEDERATION CENTRE

The Confederation Centre complex, still under construction, is with the Grand Casino, among the most important buildings erected in downtown Geneva during these last 10 years.
At Confederation Centre, contiguous buildings bordering the excavation, the asymmetry of earth pressure due to the location of the construction at the foot of the hill of the Old City of Geneva imposed here very particular excavation sequences, among others the subdivision of the site area in 9 independant excavations, performed successively.

Measurements on the shoring of the largest deep retaining structure - excavation # 4 - are studied in this paper. Fig. 1 shows this subdivision, the site plan, and the excavation chronology.

Below the compact sands, which form the upper part of the hill of the Old City, there are successively and down to great depth silt and silty clay of the wurmian retreat. The ground water is located around 2 meters depth.

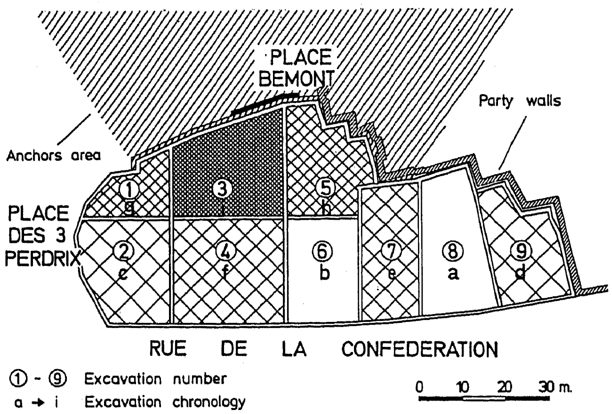

Fig. 1 Confederation Centre
Principle of subdivion and excavation chronology.

Fig. 2 summarises the geotechnical characteristics of the site.

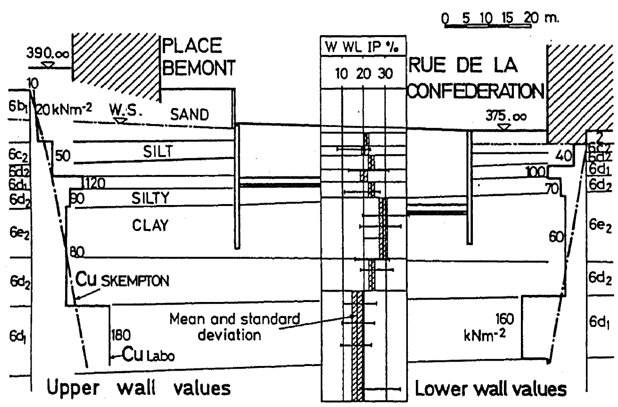

Fig. 2. Confederation Centre
Geotechnical characteristics.

Excavation # 4, 20 m deep, and with an earthwork volume of 13.000 cubic meters, is classicaly buttressed by 6 levels of struts. The struts, consisting in H-Beams, were prestressed at 500 kN each. Measurements showed however that this load decreased rapidly, to increase afterwards with the different sequences of excavation.
Fig. 3 shows the shoring plan, the location of the measurements points by extensometer and VSL load cells. Earth and water pressures acting on the slurry wall were evaluated separately. Earth pressures were calculated for each excavation step assuming some wall friction.

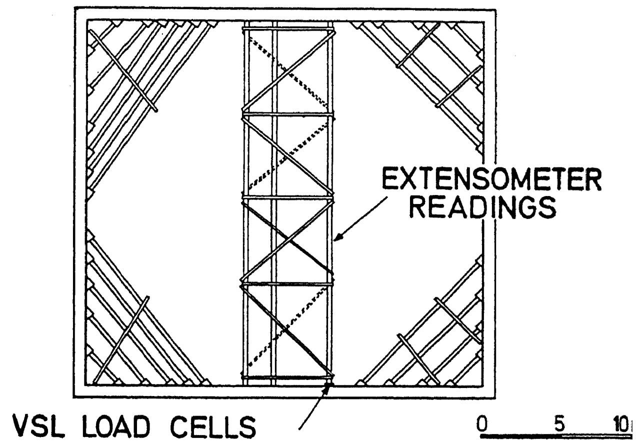

Fig. 3 Confederation Centre - Excavation # 4
Shoring plan and location of extensometers and load cells readings.

Due to the possible lateral displacement of the retaining structure (Numerous authors, from Hansen (1953), to Eisenstein and Medeiro (1983)) the theoretical lateral earth pressure distribution of triangular shape was transformed into a trapezoidal one, about half-way beetween the triangular and rectangular distribution of same magnitude.

Among many others, Dibiagio and Roti (1972), Josseaume and Senne (1979) have measured these trapezoidal type lateral-earth-pressure distribution. For the final step of excavation, earth-pressures were also calculated according to the Swiss norm SIA 191 "Ground anchors" (1977) although none were used on this excavation.

Water pressures were assumed hydrostatic in sands and silts, hydrodynamic in silty clays, this to take into account the seepage effect from the slurry wall, the drop in pore pressure due to excavation, and of course the hydraulic flow.

Determining the efforts acting on the shoring according to this method is at once and the same time complete and rudimentary. Complete for each stage of excavation is verified independently, rudimentary for the inwardmovements of the slurry wall due to each excavation step are neglected.

By this method, the theoretical maximum load in any strut level in reached once the excavation required for the lower level is carried out.

These different excavation stage are shown in fig. 4.

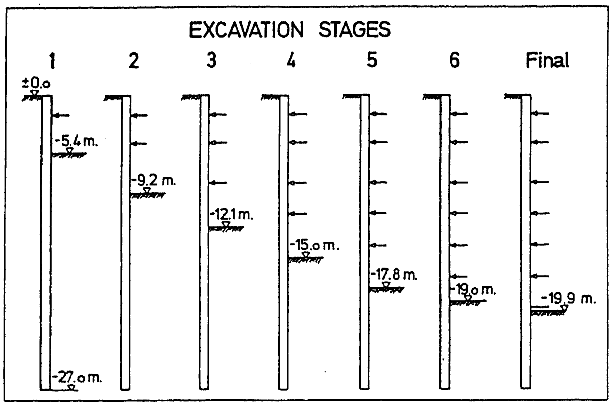

Fig. 4 Confederation Centre Excavation stages.

On this job, extensometer measurements were taken twice a month while readings on VSL load cell took place 3 times a day. At the end of the shoring period, a comparison was made beetween the loads measured at the same time by extensometer or by manometric readings (VSL load cell). The mean values are in good agreement as shown in table I.

STRUT LEVEL	1	2	3	4	5	6
N mesures	12	12	11	17	23	2
Mean load value:						
load cells kN	183	689	834	1009	1218	956
extensometer	203	666*	750	957	1235	732
Ratio load cells / extensometer	0.9o	1.03	1.11	1.05	0.99	1.31

Table I. Confederation Centre. Excavation#4. Mean strut loads values as measured by load cells or extensometer readings.

This concordance enabled to draw the evolution of the total loads acting on the struts vs. the different excavation stages. This evolution is represented on fig. 5.

This diagram shows the importance of the efforts due to daily temperature variations, this in spite of the thermical insulation - a layer of plastical foam (ethafoam) covering the H-Beams.

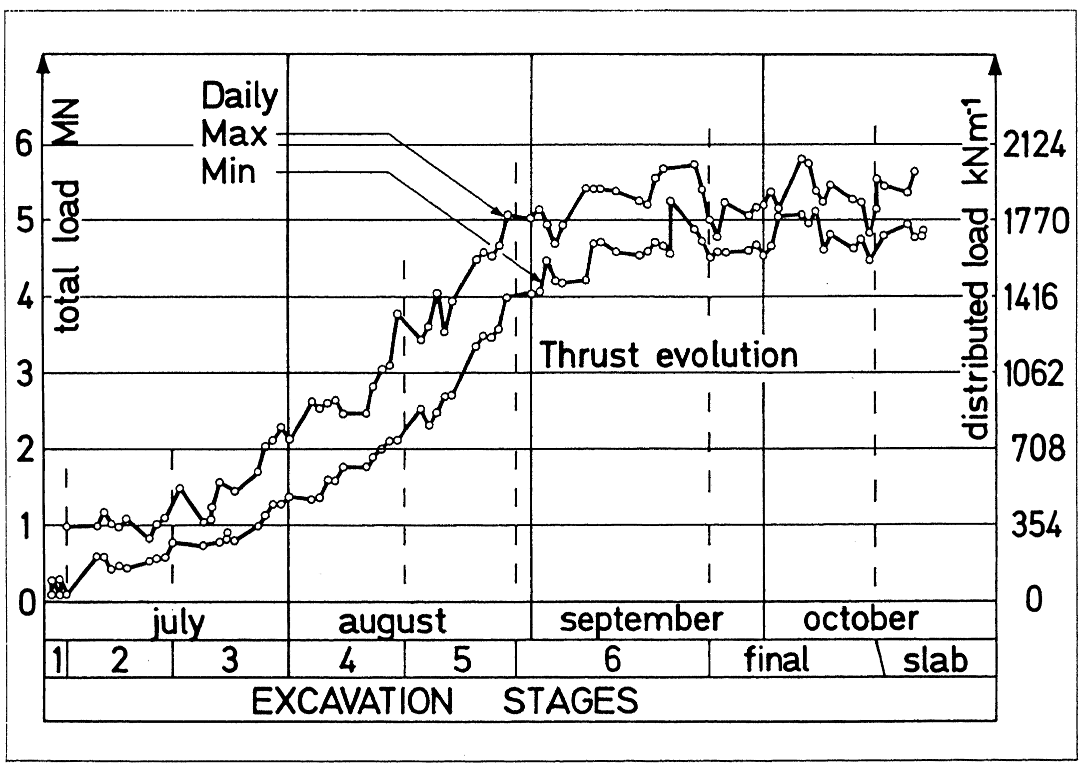

Fig. 5. Confederation Centre. Excavation#4. Thrust evolution.

These efforts are in fact comprised beetween 7 and 50 % of the total load, maximum values being measured during summer-time and the early stages of excavation. Strut loads remained always well below the design values, as shown in table II.

STRUT LEVEL		1	2	3	4	5	6
MAXIMUM MEASURED DURING EXCAVATION	kN	510	1000	1080	1420	1610	1170
CALCULATED	kN	690	920	1500	2270	3030	2030
DESIGN VALUE	kN	1130	1980	1980	2540	2540	2540
RATIO MAXIMUM MEASURED / DESIGN VALUE	%	45	51	55	56	63	46
MAXIMUM MEASURED AFTER EXCAVATION	kN	-	1430	1460	1530	-	-
RATIO MAXIMUM MEASURED / DESIGN VALUE	%	-	72	74	60	-	-

Table II. Confederation Centre. Design and maximum measured loads.

For 3 struts, maximum values were measured during basements construction, once the lower level removed. Maxima values, during excavation stages, were strongly delayed in time vs. design model. This shifing is illustrated in fig. 6.

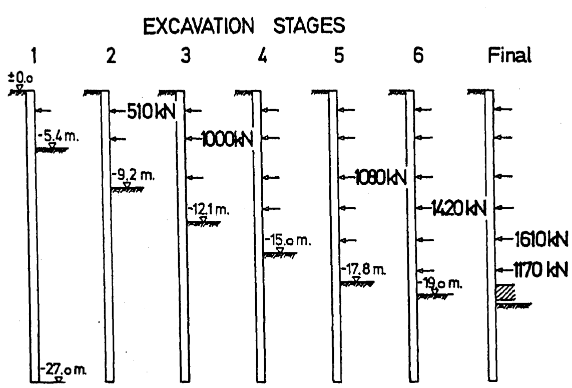

Fig. 6. Confederation Centre. Excavation#4. Excavation stages and maximum loads values.

Daily temperature variations - maximum measured 12°C - produce stresses in the order of 40 to 60 % (0 - 100 % variations) of those obtained with the hypothesis of fixed undeformable supports. Fig. 7 shows these load variations with temperature increase, as measured with the extensometer on the second strut level during two sunny days of July.

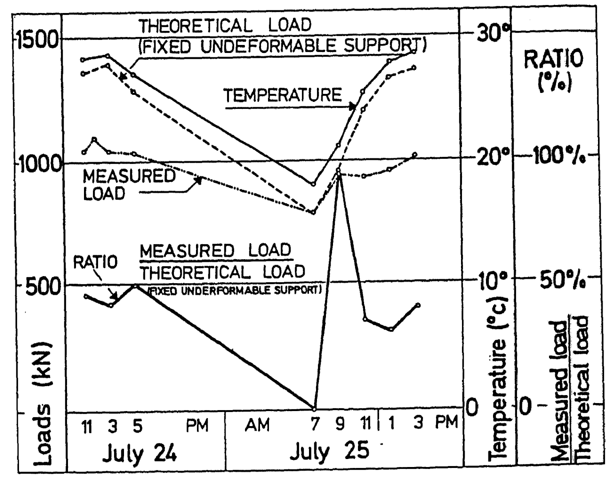

Fig. 7. Confederation Centre. Strut level Load variations with temperature increase. Extensometer readings.

In that case, the support provided by the complex "earth-slurry wall" acted like a spring having an average stiffness K of 43 MNm^{-3}. Other measures of that type gave values of K ranging beetween 10 and 50 MNm^{-3}. Hilmer and Vogt (1980) give such orders of magnitude.

GRAND CASINO

This building occupies an area of 5800 squa meters on the lake shore. Under a layer of fill, there are boglimes, then lacustrine muds above a very thick layer of compressib silty-clay and clay deposits of low shear strength. The ground water surface is hori zontal and located at -2 m and the earth pressures are practically symmetrical. The closest building is located at more tha 10 m from the excavation.

Fig. 8 recapitulates the geotechnical conditions of the site.

The depth of excavation reaches 13.8 meters the total volume of earthwork 77.000 cubic meters.

Once the slurry walls carried out, a first excavation enabled to cast a reinforced con crete rim on the ground and to span across the rectangular aperture by a single centra horizontal shore; this permitted the earthworks in the central part of the excavation and the concreting to be carried out in stages.

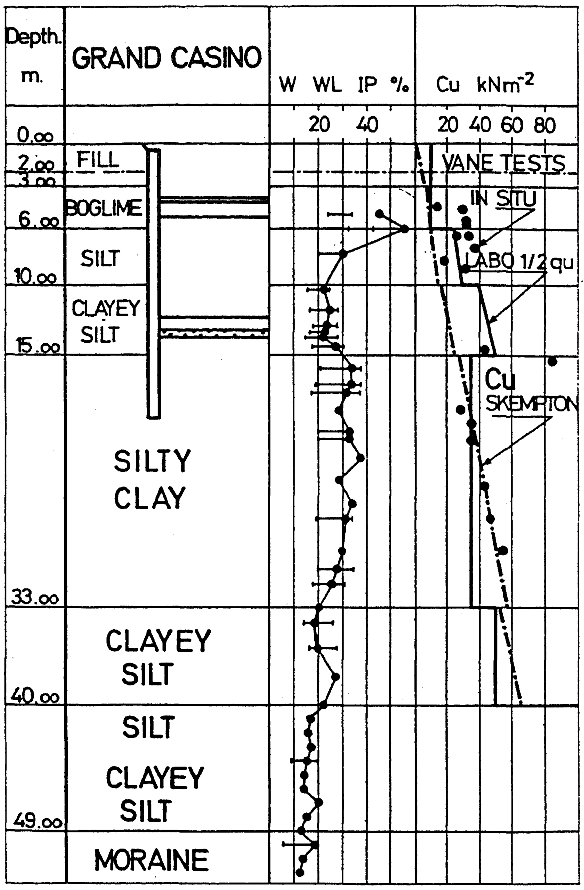

Fig. 8. Grand Casino. Geotechnical characteristics.

Berms were afterwards excavated in daily stages of 4 meters width. An under-base slab in porous concrete was cast beetween the base of the walls, for which it subsequently acted as support and then the base slab itself was cast, again in stages.

Fig. 9 gives an outline of the excavation, the bracing placed, the location of extensometer readings and of the different stages of excavation.

The volume of earthwork, and the need for an as best as possible appraisal of the stresses acting on this deep retaining structure led to many studies.

The element of structure formed by the reinforced concrete rim and the central shore was computed by the finite element method, while the total pressures acting on the slurry wall were analysed by several ways. In addition to the usual method defined before, global methods were considered, such as those proposed by Tschebotarioff (Neutral Ratio Design) and Peck (1969).

Total pressures acting on the reinforced concrete rim computed according to these methods are summarised in table III.

Considering the big scatter of these results, it was difficult to fix a design value. This choice was to be eased by the back calculations of existing "in situ" measures on the shoring of a deep retaining structure, braced on three levels, which was located in the immediate vicinity of the Grand Casino project.

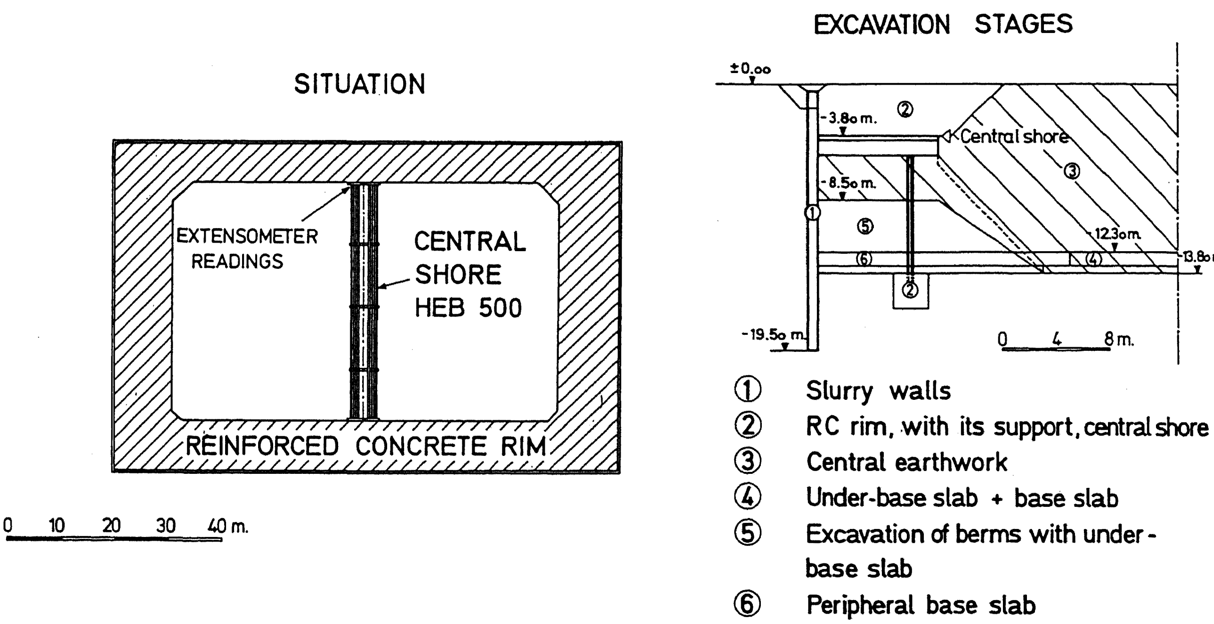

Fig. 9. Grand Casino. Excavation stages.

Total pressures acting on the reinf. conc. rim (kNm^{-1}) after	Embedment level under base slab	Embedment level base slab
1° Usual computations	520	480
2° Tschebotarioff	390	360
3° Fluid $\rho = 2.00$	670	610
4° Peck Mexico (1969) Cu labo	1110	1050
Cu "in situ"	970	920
Design value	550	

Table III. Grand Casino. Total pressures on the reinforced concrete rim after various methods.

Measured loads being lower - as for Confederation Centre - than those computed by the usual method described before, the design value was fixed at 550 kNm^{-1} on the rim, i.e. 15.8 MN for the central shore.

Regular extensometer readings on the H-Beams forming the central shore gave the following results :

- a very fast increase of forces (the design value was attained when only half of the excavation volume was performed) which led to the need for the shore reinforcement.

- the reinforcing H-Beams (2 HEB 500), jacked each at 1050 kN, presented after wedging a residual average force of 860 kN. This total added force of 1720 kN induced a total decrease in the first 6 H-Beams of 340 kN only. 4 days later, the added force due to prestressing had almost disappeared.

- a momentary force peak for a week (160 % of the design value, when 75 % of the excavations was carried out) due probably to internal secondary passive thrusts.

- an average final force value very close to the computed value.

- loads variations due to fast (1 to 4 days) temperature modifications also comprised beetween 40 and 60 % of those computed with the hypothesis of fixed undeformable supports.

- the apparent lack of relationship beetween forces and the monthly temperature averages

Fig. 10, 11, 12 et 13 recapitulate these points.

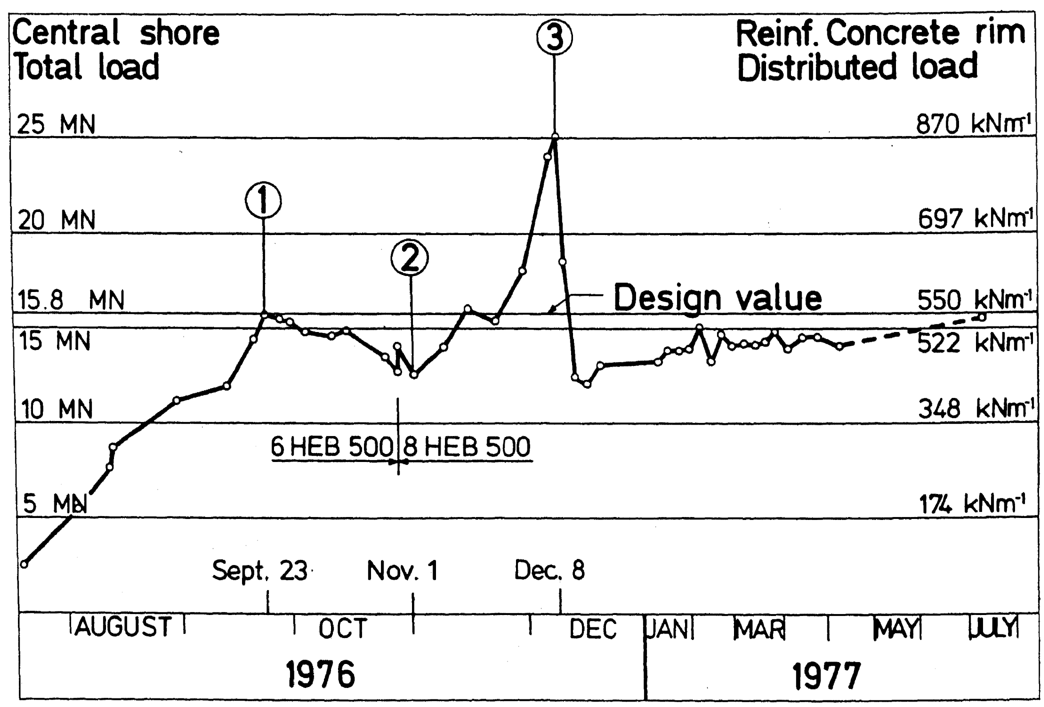

Fig. 10. Grand Casino. Thrust evolution on the central shore.

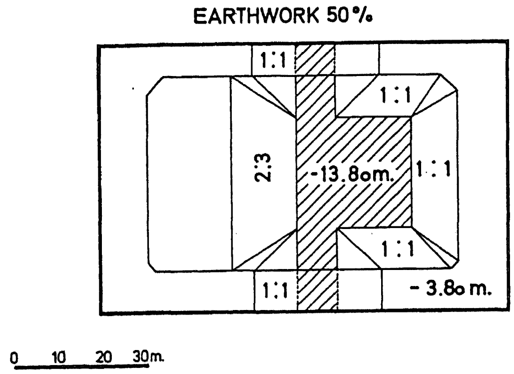

Fig. 11. Grand Casino. Point 1
Excavation on September 9, 1976.

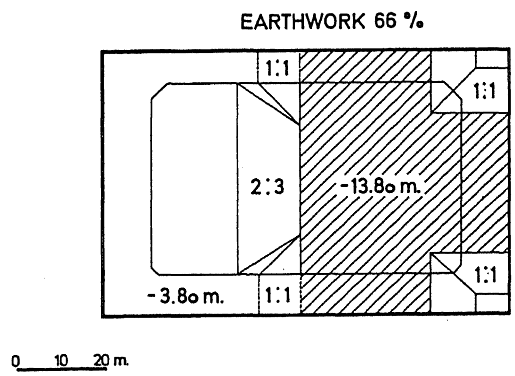

Fig. 12. Grand Casino. Point 2
Excavation on November 1, 1976.

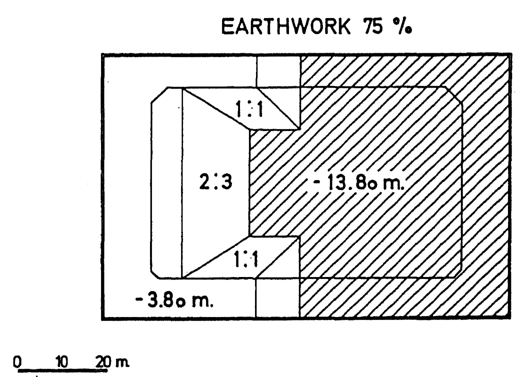

Fig. 13. Grand Casino. Point 3
Excavation on December 8, 1976.

It is worth noting that any earthwork in the central shore vicinity induced in this latter a load increase.

CONCLUSIONS

Altough the measurements performed on these two excavations seem quite contradictory, the following points can be brought out :

- Design models of an excavation braced at several levels give conservative values. This type of shoring produces as a consequence a higher overall safety factor, hence lower deformations. Reciprocally these facts disappear for an excavation braced on one level only.

- Excavation speed, in saturated fine grained soils, plays a part having for result to shift the total pressures in the time.

- Tridimensionnel effect appears :
  . by arching effects on the angles, reducing forces and deformations (Confederation Centre).
  . by internal secondary forces in the case of a single continuous shoring (Grand Casino).

- Daily temperature variations may be reduced by adequate constructive measures and taken into account in the design.

In Geneva (climate of the temperate type) seasonal temperature variations do not seem to play a role, according to the measures taken up-to-now.

In geotechnical engineering, external loads are often hard to assess with precision. "In situ" measures make it possible :

- to reduce this uncertainty.

- to adapt the project to actual conditions.

- to improve the present state of our knowledge.

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