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SYNOPSIS In the paper is presented a history of the long-term process of deformations and damages to a five-storied apartment building in Gliwice, one located over a thick lens of a young, very weak peat deposit. The evolution of deformations is defined by changes in time of representative foundation movement components (the angular distortion, deflection ratio, etc). Relationships given and analysed in the paper are based on the results of settlement monitoring preformed since 1970, and failure escalation descriptions make use of crack documentations included in the expert opinions. The case history also comprises not quite efficient attempts of object protection. The description is completed by data concerning the building structure and soil conditions. In conclusion an idea for ground stabilization is recommended by the authors.

INTRODUCTION

Among the various causes of building failure and disaster, faulty foundations take a very important place. There are geotechnical situations, where the application of shallow foundations without appropriate ground improvement is absolutely inadmissible, and this restriction cannot be softened by constructional treatment such as the over-stiffening of structures, especially their underground parts, additional reinforcements, or expansion joints. Ignoring that fact accounts for the prevailing errors in foundation engineering. The sources of this can be various, e.g. insufficient recognition of soil conditions or its lack, incorrect prediction of the subsoil bearing capacity or settlement, or the neglect of an influence that soft layers more deeply situated have.

The history presented in this paper is an extreme example of the consequences of this last error cause. A not so very high and moderately heavy apartment building has suffered very great deformations and damages, in spite of continuous strip foundations and rigid floors, both made of reinforced concrete.Results of far-reaching reconstruction undertaken when the object condition was close to disaster proved insufficient. Although one managed to prevent a violent destruction of one building part, the center of damages went over to another area. Cracks of walls, floors and stairs, as well as deformations of door - ways and window openings have been developing there through a slow, long-term process. At the same time, the differential settlement of the structure is increasing due to large strains in the thick peat lens which were neglected during building design. In the authors' opinion, there is today no alternative for a rational stabilization of the peat layer. Probability of a disaster, e.g. a wall collapse, is at present, very high.

The paper provides details of the above case study. At the beginning, data are quoted concerning the building geometry and structure, as well as the subsoil stratification and geotechnical properties of peat. The next section contains an exhaustive description of the case history. At the end, an evaluation of the present state is included. A strategy for ending the deformation and damage process, and for the building reconstruction, proposed by the authors to the owner are then briefly discussed.

GEOMETRICAL AND STRUCTURAL CHARACTERISTICS OF THE BUILDING

The described apartment building is situated at Chopin Street in the center of Gliwice, an Upper Silesian town with a population of 200,000 people. The plan of the object shown in Fig. 1 is the L-shaped, 47,2m long and 12m or 15.8m wide. This is a five-storied building with cellars. Its height amounts to 18m (from the ground level to the roof ridge), and the average depth from the ground level to the foundation concrete bottom - 3m.
The load-bearing structure consists of the longitudinal brickworks 0.51m, and 0.38m thick, and the typical Akerman's rib-and-slab floors entirely restrained in the walls.The walls of the building basement are rested on a system of continuous strip footings. The widths of carrying longitudinal footings are rather large. They amount to 2.15m in the case of the external walls, and 3.15m for the central one.The foundation rests on a lean concrete layer of the thickness varying between 0.1m and 1.2m. The design did not provided for any building division by expansion joints. During its erection an engineering supervisor decided to divide the building between segments No.15 and No.17 into two parts. This division did not occur, however, in the foundation.

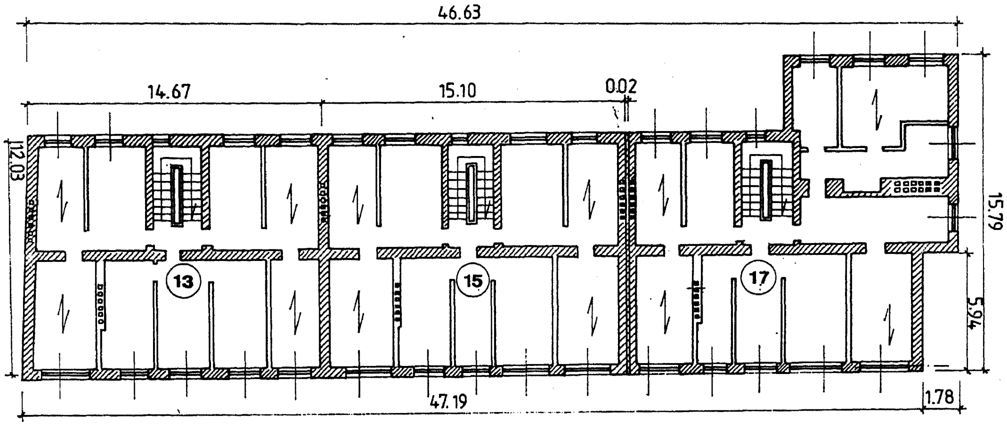

Fig.1. Plan of a typical storey of the building

GROUND PROFILE

The ground in the building site is composed of holocene lacustrine deposits. Geotechnical investigations carried out in six bore-holes have allowed for separation of the following soil layers:

- a continuous fill layer formed of medium sand, clayley sand, silty clay and industrial waste, occurring from the ground surface to the depth of 3.0m to 4.8m,
- a lens of organic soils of the maximal thickness (next to the expansion joint between segments No.15 and No.17) reaching 5.4m,
- a layer of sands of different granulation, surrounding and underlaying the organic lens,
- a layer of stiff sandy clay.

The ground profile described above is visualized by a block-diagram which presents spatial variability of soil layers beneath the structure (Fig.2).

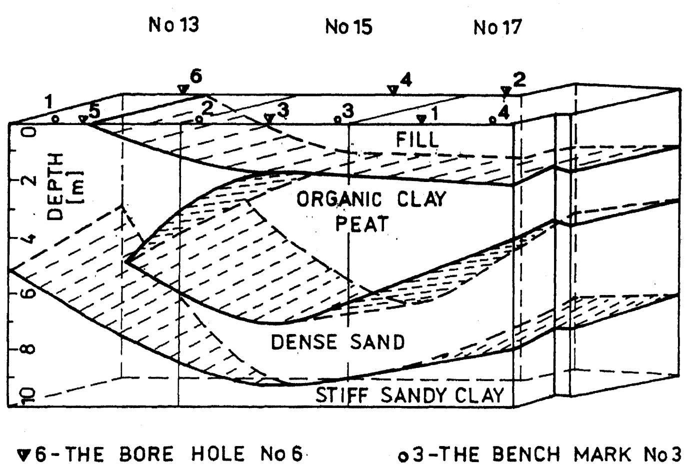

Fig.2. Block-diagram of the building subsoil

As can be seen, the geometry of the weak organic layer is extremely unfavourable from the foundation engineering point view. Its maximal thickness falls beneath a central part of the structure (in the area of expansion joint). Moreover, in this part, the top surface of organic lens is situated on the minimal depth. Towards both gable ends of the building this surface lowers and the thickness of lens rapidly decreases. Under an external part of the segment No.13 organic soils do not occur at all. It is quite evident that a such layering favours differential settlements which are far larger beneath the central area of the building than under its ends.

ENGINEERING PROPERTIES OF ORGANIC SOILS

The organic soil lens is non-homogeneous. Soil, in the top part 1.7m thick, has been identified to be organic clay. Some of its engineering properties have been evaluated on the basis of the autors' laboratory tests, and are as follows: averange organic matter content - 10.4%, analogical moisture content -52,2%, and constrained modulus - 1260 kPa. Organic soil occuring in the sublayer 4.3m thick is a very soft and weak peat characterized by the following average engineering properties: organic matter content - 93%, moisture content - 370%, and constrained modulus - 540 kPa. Fig.3 shows variations of the above features with depth. As can be seen, peat appears to be quite homogeneous. The organic matter content is particularly high, which is a distinct sign of an unusually low bearing c
apacity for peat.

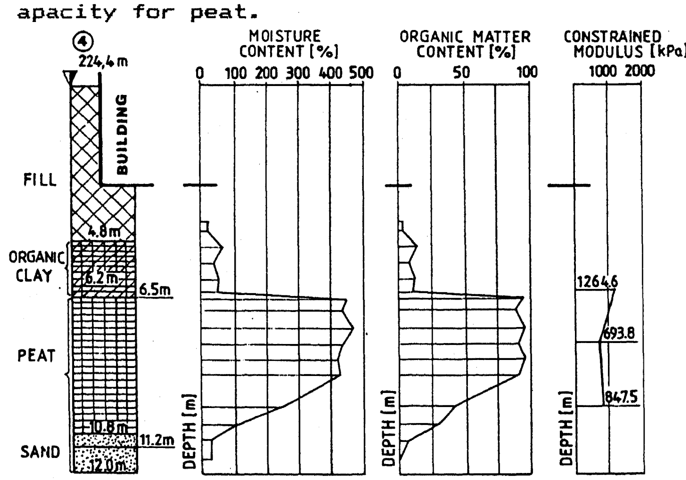

Fig.3. Variations of organic soil engineering properties with depth, a) organic matter content, b) moisture content, c) constrained modulus

There is no doubt but that the building failure and destruction process is a consequence of applying shallow foundations over the organic soil lens, as describing above. Its particularly unfavourable geometry (the shape and situation) as well as an immense deformability and strain ageing of peat are responsible for the scale of failure.

HISTORY OF DEFORMATIONS, DAMAGES AND RECONSTRUCTIONS OF THE BUILDING

The building was erected in the years 1962-1965. Large subsoil surface displacements appeared as early as the building construction stage and were developing quickly for the first three years. The magnitude of total subsoil settlement which occured during that period was not known. In the expert opinion elaborated by Bela and Śliwa (1968) one could find, however, the measured differences in levels of landings in the neighbouring staircases. They amounted to about 210 millimetres, when comparing the segments No.13 and No.15 and only 40 mm comparing the segments No.15 and No.17. Starting from the above data one can roughly estimate two of the foundation movement components introduced by Burland and Wroth (1974) to describe differential settlements and check if they do not induce the ultimate or serviceability limit states of a structure. In 1968 first of these components, the s.c. maximal deflection ratio $(\delta\rho/l)_{max}$ amounted approximately to 1/70 and so it exceeded more than four times the value $(\delta\rho/l)_{adm}$ = 1/300 recommended by Skempton and Mac Donald (1956) to be admissible one. The other component, s.c. maximal deflection ratio $(\Delta/l)_{max}$, occuring in the expansion joint zone, amounted approximately to 1/285 and was larger as much as seven times than the admissible value $(\Delta/l)_{adm}$ = 1/2000 recommended a.o. by the Standard Eurocode 7 (1989).

In these circumstances, the ultimate limit state in structural elements of the building was inevitable. Indeed, severe damages to the structure appeared almost at the beginning of the construction stage and increased together with the growth of differential settlement. In 1968 the structure condition, in the segment No.13 proved to be catastrophic, particulary in the zone comprising the staircase and apartments adjoining to the segment No.15. All interwindow pillars suffered cracks through walls, running obliquely from one window to another. Their widths reached 25 mm. At the same time, the basement walls and the beams crowning floors of lower storeys were tearing apart and the walls of the top floor storey suffered crushing. All these failures indicated that the total collapse of the structure happened in a zone of the staircase of segment No.13. Some cracks of walls also occured in another part of the building. However, these were local and of little importance.

According to prescription including in the above cited Bela and Śliwa's expert opinion inhabitants of the building were timely evacuated. Moreover, some conclusions of this opinion constituted the basis for several variant design solutions concerning the structure protection against further differential settlements and repairs of building damages. There were also some ideas for a limited ground stabilization (sunk foudation wells under the transverse wall in the failure area and sheet pile walls around the building).

Finally, the general renovation performed in 1976-1978 years comprised the following reconstructions and repairs (Fig.4):

- the additional division of the building by the expansion joint between segments No.13 and No.15, including the foundation,
- excluding from use all apartments in the segment No.13 adjoining the new expansion joint,
- over-stiffening to the structure in the zone of these apartments by bricking up all windows and doors,
- prestressing all longitudinal and transverse walls of the building with horizontal anchoring rods of the diameter of 25 mm, performed on the levels of all floors and the roof,
- levelling floors, spraying cracks with cement grout, repairing doors and windows.

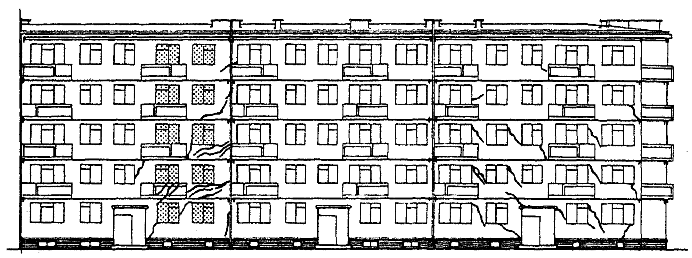

Fig.4. State of the front wall cracks till the general renovation and applied protections

Lack of any attempt of eliminating causes of the building failure is unbelieveable. Unfortunately, after finishing the general repair and renewed settling inhabitants the differential settlement and damage process continued developing, but its center went over to the area of the expansion joint between the segments No.15 and No.17. It is very clearly seen on the diagrams of the time-settlement relations, drawn up on the base of systematic monitoring settlements of several bench-marks since 1970 (Fig.5). The settlement of the bench-mark No.3 is largest and exhibits the utmost increase since 1970. When analysing the structure condition in 1984 Bela and Sękowski (1984, 1987) paid attention to the deformation process continuation, as well as to the crack development in the segment No.17, and compressing the expansion joint. In their opinion, however, settlement increments were of the decreasing tendency. On this basis,they predicted that the ultimate settlement will be at most 110 mm higher than that measured in 1984.

Unfortunately, this rather optimistic forecast has not come true. As can be seen in Fig.6, after the period of stabilizing tendency till 1985, the settlement rate began increasing and this trend continues to date.

PRESENT CONDITION OF THE BUILDING

The actual state of damages is assumed to be a function of the differential settlements which came after the reconstruction in seventies. Then, one can evaluate the foundation movement components saying nothing of the settlement till 1970. The reliable maximal value characterizing the present differential settlement of the front wall of building, evaluated according to the definitions given by Burland and Wroth (1974), are the following: the relative settlement $\delta\rho_{max}$ = 190mm, the relative deflection Δ_{max} = 157mm, the deflection ratio $(\Delta/l)_{max}$ = 1/242, the angular distortion β_{max} =1/70, and the angular strain α_{max}= 1/72.

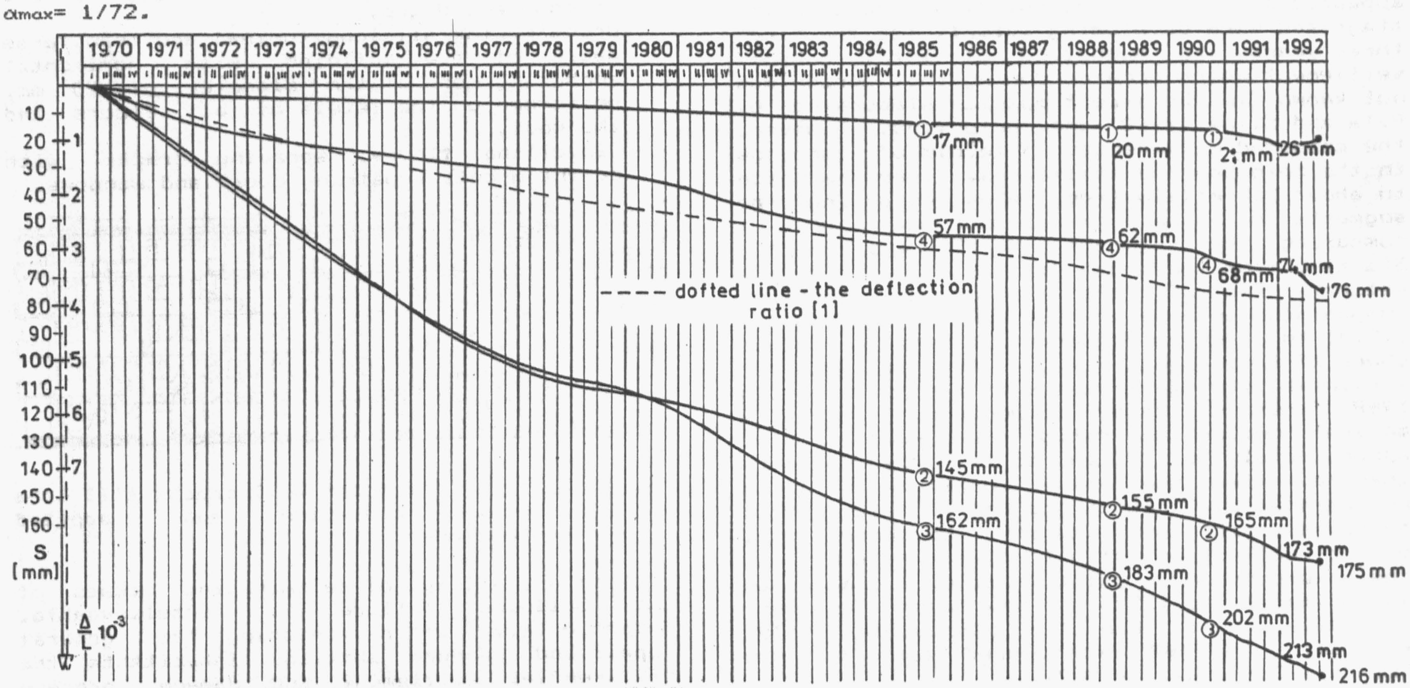

Fig.5. Results of monitoring of settlement process of the front wall

In the light of the Standard Eurocode 7 (1989) the maximal deflection ratio exceeds presently the admissible value over eight times.

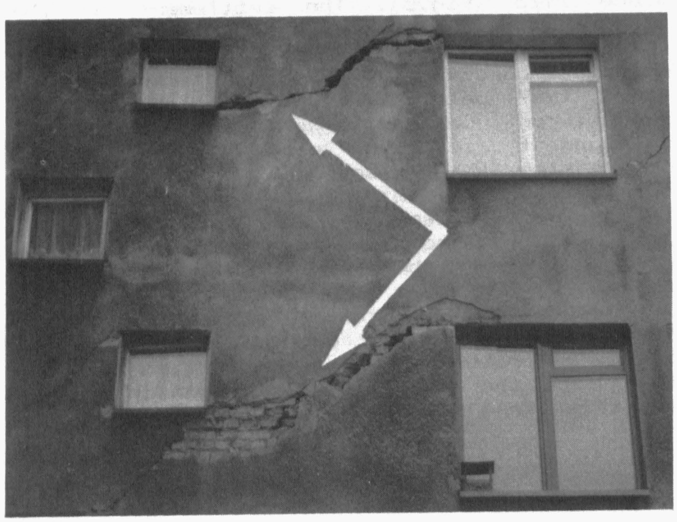

Fig.6. Crack in the back wall

This is reflected in the current state of the damages to the segment No.17. A network of oblique cracks comprises the majority of interwindow pillars of the front (Fig.7) and back walls, and also a part of the transverse ones. These are wide, continuous fissures (Fig.6) running through brickworks. The direct cause of cracks in the longitudinal walls of the segment No.17 are their angular disfortions. They are induced by the passive pressure of the walls of the segment No.15 transmitted through the cmpressed expansion joint. This is the response to the differential settlement of the No.17 one.

The present state of building structure is a source of serious threat for people and their belongings. The analysis of diagram in Fig.5 (dotted line), presenting changes in time of the maximal deflection ratio, points to progressive characteristic of this dependency. This is the effect of volume strains in peat following the process. of biodegradation of organic matter. The process is far from ending, and even a small increase of deflection can cause a disaster. An immediate intervention comprising the structure strengthening and subsoil stabilization is necessary.

CONCLUSION

In conclusion a strategy of the building saving is briefly presented. The carrying structure of the building is very weakened and it is necessary, at least, to reinforce areas of its cracking. At the same time an efficient subsoil stabilization is required.

Last year the authors undertook an attempt of saving the building in the above range (Gryczmański and Sękowski, 1991). Its general strategy is outlined in Fig.7. This is composed of reinforcing weakened wall areas with flat

steel rods situated across cracks, and of stiffening soft subsoil with the help of micropiles.
First of the proposed protection means has been designed by Gryczmański (1992). It is worth enlarging the information contained in Fig.7 by some details. The flat steel roads 60 mm wide, 10 mm thick and of various length are to be placed in special grooves hewn out on both sides of the given wall. The rods are to be inserted into grooves so that their width are prependicular to the facades.

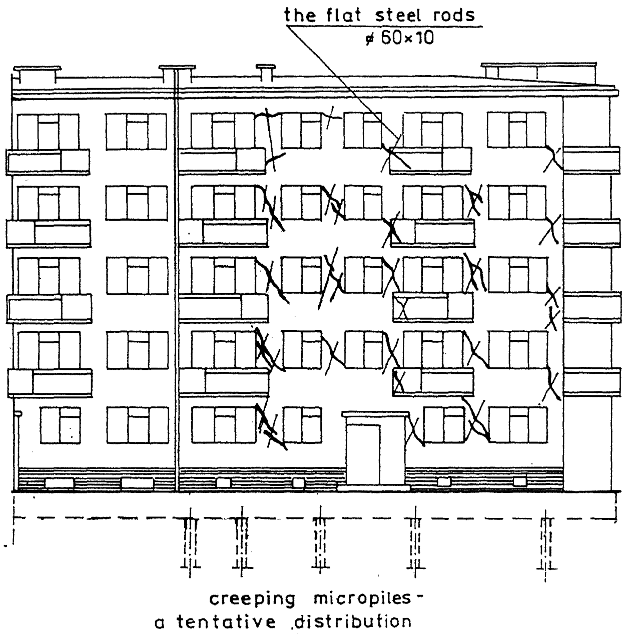

Fig.7. Distribution of the present cracks in the front wall and the applied protection means

The micropiles are being just designed in detail. Therefore, Fig.7 only shows their tentative distribution and lengths. At any rate, these are the s.c. creeping micropiles which are conceived to be interacting with fill and organic soils. They are assumed to carry only a part of the subsoil loads following from their relative stiffness, as compared with that for surrounding soil. Their application will cause a significant general stiffening of subsoil and a favourable stress distribution (a relaxation of effective stress in peat). Selecting the suitable number, distribution and lengths of micropiles one can reduce further increase of settlement to a small magnitude not dangerous for the reinforced structure of the segment No.17. The detailed solution will be presented in an other paper after some time of building use, when further settlement monitoring results will be available, evaluating the efficiency of subsoil stabilization.

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