

PROCEEDINGS
OF THE NINTH
DANUBE-EUROPEAN CONFERENCE ON

SOIL MECHANICS AND FOUNDATION ENGINEERING

BUDAPEST, OCTOBER 2-5, 1990

EDITED BY
G. PETRASOVITS



AKADÉMIAI KIADÓ, BUDAPEST

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Ninth Danube-European Conference
on Soil Mechanics and Foundation Engineering

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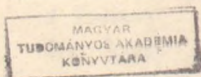
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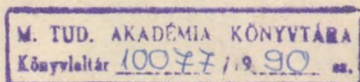
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PREFACE

The Ninth Danube-European Conference on Soil Mechanics and Foundation Engineering is to be held in Budapest from 2nd to 5th of October 1990.

The idea of this Regional Conference was initiated more than twenty years ago and the first Danube-European Conference was jointly organized by the Austrian and Hungarian National Committees of ISSMFE. Since this time practically all of the Danube Countries have been hosts to these Regional Conferences and the good cooperation between the member Societies stimulated keeping this good idea alive.

The main topic of this Ninth Danube-European Conference on Soil Mechanics and Foundation Engineering:

« Deep Geotechnical Works in Urban Areas »

was chosen by the representatives of National Committees ISSMFE of Danubian Countries.

The Proceedings contains 70 papers contributed by authors from 18 countries. These papers were selected from more than 120 abstracts submitted to the Organizing Committee.

The distribution of papers among the countries is the following:

Austria	3
Belgium	1
Bulgaria	3
Canada	1
Czechoslovakia	4
Germany (FRG)	6
Germany (GDR)	2
Hungary	20
India	1
Japan	1
Netherlands	1
Poland	5
Romania	6
United Kingdom	1
Soviet Union	11
USA	1
Yugoslavia	3
Total	70

Special thanks are due at this stage to the members of Scientific Advisory Board, especially to Prof Dr Smoltczyk, the Vice-President of ISSMFE for Europe.

Finally, I wish to express my gratitude to all who helped in the preparation of this Proceedings during the short time at our disposal to my colleagues, Mr Gyula Páti, Mr Zoltán Czap, Dr János Lőrincz, Mr Miklós Juhász, to Mrs Judit Gödl, to the Staff of the Geotechnical Department BTU and to Akadémiai Kiadó, for making possible the publication of this Proceedings.

Budapest, July 1990.

Prof G. Petrasovits
Chairman
Organizing Committee

PLENARY SESSION

GROUND WATER IMPACT ON FOUNDATIONS IN URBAN AREAS

PROF.DR.PETAR ANAGNOSTI
ENERGOPROJEKT YUGOSLAVIA

INTRODUCTION

The Danube river is one of the largest rivers in Europe having 2860 km in its length and 570 000 sq.km of the water shed area flowing through many countries(See Fig.1). The important settlements and industrial zones are located on the river banks including several capitals (Vienna, Budapest, Belgrade).

The large part of the Danube river bed has been formed in large planes or at the rims of the planes where the recent alluvial deposits are overlaying ancient sea or lake deposits consisting of sandy-silty soils. Such conditions are causing rather strong relationship between the water levels in the river and the ground water surface in the banks.

The absolute elevations of the Danube river water levels may vary generally from 4 to 6 m during the years of average flows in that parts which are outside the water level control by the power plants or other regulating structures. It has been recorded that the intensive water regulating works in the water shed and along the river course during last 50 years caused the trend to decrease the water level changes in the Danube river throughout the year. This tendency is causing the gradual changes in natural outflow - inflow discharges of the ground waters towards the river and from the river. The constant increase in the ground water surface is the generally recorded feature particularly during the lowest water levels in the river course. This is causing gradual increase of the permanent water surface in the banks and decrease of the inflow-outflow discharges between the river water and ground water. Such a change is particularly important in urban areas where the forming up of a steady ground water table at higher elevations creates unfavorable conditions

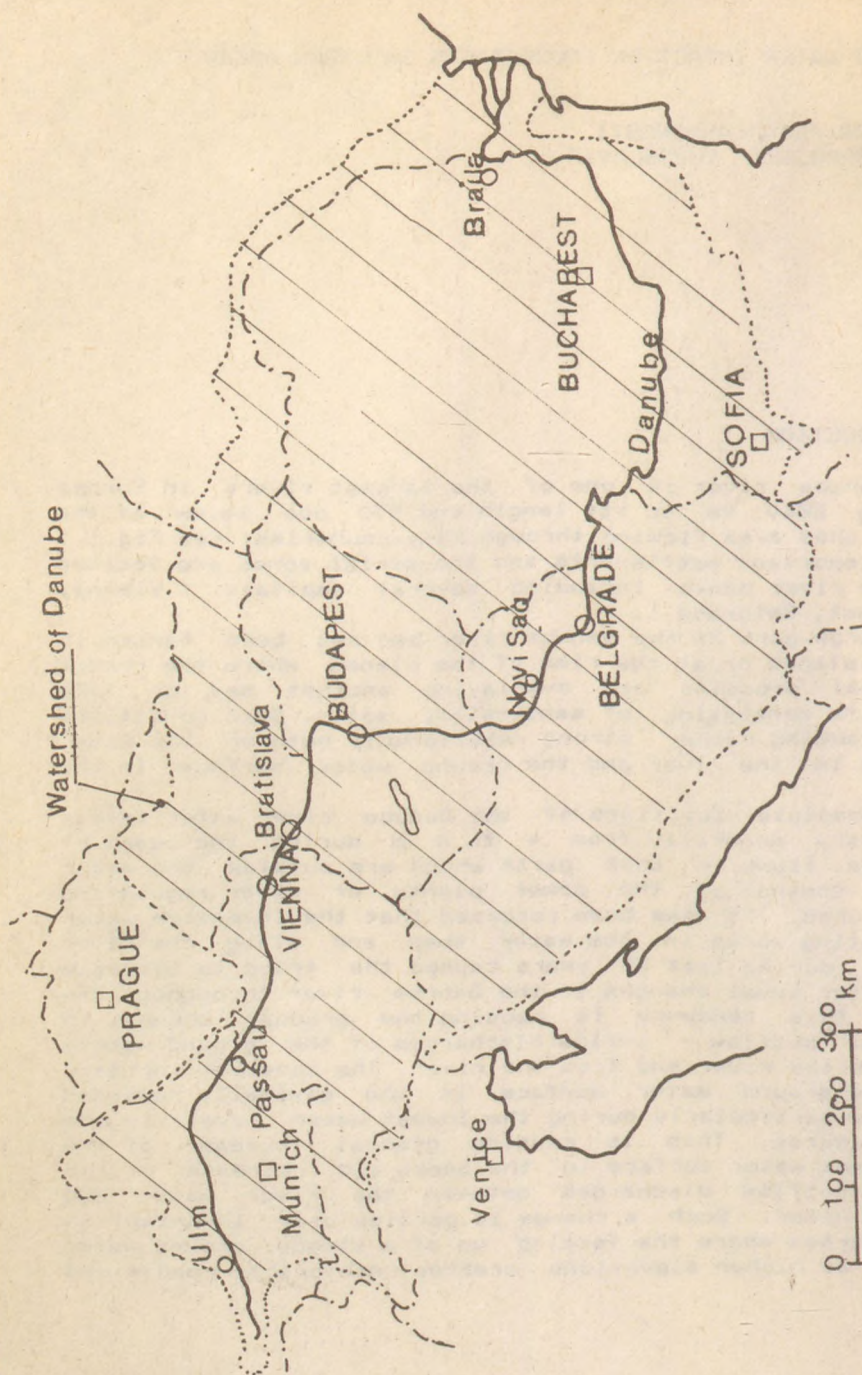


FIG.1

for subsurface construction works, increases the risks of pollution and causes various forms of deterioration of the existing buildings.

ROLE OF GROUND WATER IN FOUNDATION ENGINEERING

In the geotechnical engineering the appreciation of the ground water influences is the necessary prerequisite for the town planning activities, for carrying out the subsurface construction works, designing the foundations and other buried structures. Nowadays the particular attention is to be paid to the pollution problems and to the safety measures for various waste materials deposits. It has to be recognized that the correct knowledge of the ground water conditions, particularly in the urban areas is a complex task that requires the extensive scope of studies to establish the particular features common in ground water hydrology, such as artesian aquifers, zones of recharge, impervious boundaries, gradients of ground water flow and its direction etc.

The quality of the ground waters in urban areas is a particularly sensitive feature due to the existence of many sources of pollution and its changeable character in time and in space. The appearance of the polluted water at the ground surface, in open air collectors, in the deeper parts of the existing buildings etc. may jeopardize the sanitary conditions and create otherwise unacceptable states for the populated areas.

In the mean time the building activities in urban areas are more and more oriented to the use of underground space not only for the building foundations, but as usable spaces for storing the goods, for parking spaces, shelters, traffic connection lines etc.

The building activities associated with long term dewatering for water supply, for temporary protection of foundation excavations, for permanent drainage of critical areas etc. may cause the continuous lowering of the ground water surface and displacements of the overlaying strata due to the increased effective pressures.

The various impacts caused by the ground water on the subsurface parts of the structures and subsurface structures, construction works etc. may usefully be subdivided into two categories :

a. The presence of the ground water (without considering the effects due to the ground water flow) creates the following typical interactions:

- uplift forces (pressures) which has to be evaluated in relation to their structural and hydraulic impacts on the submerged parts of the structures.
- changes in soil properties due to submergence or due to direct contact of the soil with ground water may be very

different and even opposite in its character. Thus the dense clayey soils may swell, but loose and weakly cemented soils may collapse, the uniformly graded sandy soils may become liable to liquefaction during vibrations of the ground, the soils containing dissolvable salts may be leached out and become micro,porous or cavernous etc.

- chemical impact on the structural members of the foundations made of steel, concrete, wood etc. may be caused by the presence of the specific matters dissolved in the ground water. This impact may lead to gradual deterioration of the cited materials.

b. The ground water flow may present the additional effects such as :

-migration of the fine particles along the ground water flow and their washing out through the exit areas or their agglomeration at the faces of the drainage collectors. This kind of the migration may result in significant changes of the soil structure and its strength.

-creation of the seepage forces that may cause " boiling effect " when directed in upward direction or additional effective stresses in direction of the flow.

-spreading out the pollution matters from its source.

The time factor in the interactions described here is an important component of the ground water impact, and the short term simulations (in the field or in the laboratory) may not be sufficiently representative for proper representation of the long term phenomena.

The climate may create significant seasonal influences on the states of the ground water, particularly in areas of large differences in temperatures and precipitations. The presence or absence of the vegetation, paved or nonpaved surfaces, regulated surface run-off susceptibility of the soil for water absorption etc. may have the important influence on the states of the ground water throughout the year.

The urban areas are subjected to very different building activities and the final structures are also rather different. Some of them are long continuous structures like subways, sewers, water supply lines etc. Many of them are sensitive to the ground water changes.

The long term drainage works, such as for the water supply requirements, for temporary or permanent dewatering, may cause continuous settlements of the ground surface and have the impact on shallow building foundations.

Quite opposite effects may be caused by withdrawal of large industrial plants from the residential areas (City of London case) with removal of dewatering facilities. This is causing the gradual increase of the permanent ground water table with changes in the moisture content of the soils overlaying the aquifer. The changes of the moisture content

due to full submergence, saturation, or by suction may lead to soil volume changes or to additional distortional strains required for additional mobilization of the shear strength.

METHODS TO CONTROL GROUND WATER EFFECTS

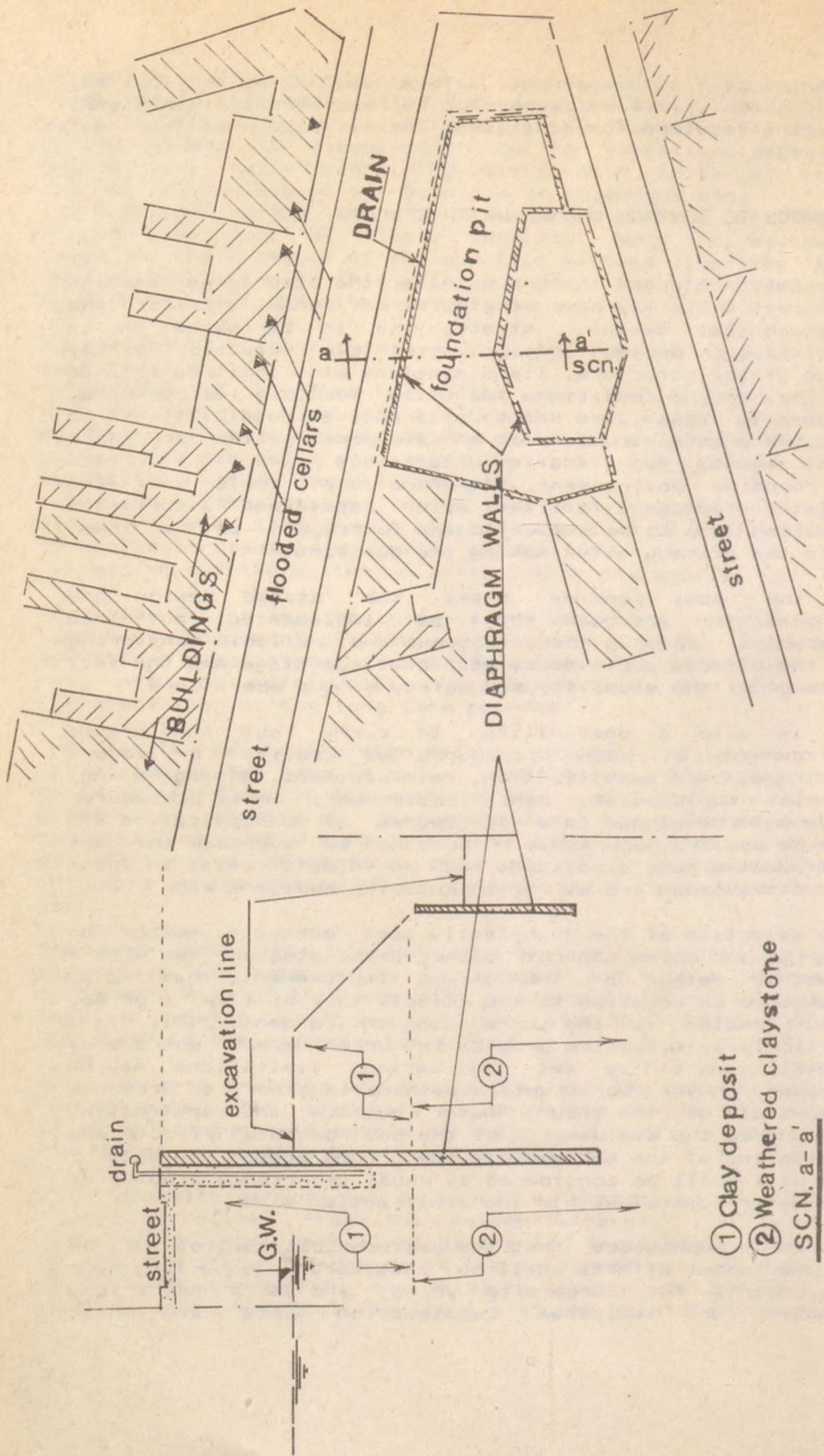
The interaction of the ground water the structures and the adjacent soils may have very different forms, therefore the geotechnical design strategy is to be based on an optimization process through which the selection of the type of the structure, its dimensions etc. shall be related to the ground conditions and their possible improvements. Generally there are many alternative possibilities to achieve a safe structure or a safe construction procedure that would not create undesirable impacts on the surrounding environment. Therefore the costs and time related factors for the each considered alternative solution has to be evaluated and introduced as an input into the procedure for making the decision.

In the most complex cases the staged design and construction approach shall be implemented, but this procedure requires the continuous and reliable monitoring of the effects achieved by each previous stage and the full control of the stability and safety of the operations.

It is also a possibility to carry out the ground improvement by such procedures as gravity or vacuum drainage, impermeabilization, reinforcement, strengthening, partial replacement, densification etc. These procedures have been developed to a high degree of sophistication and can be applied separately or combined to overcome the most difficult ground conditions such as in soft clays, in loose saturated sands and silts, in artesian aquifers etc.

The selection of the technically most adequate method for the ground water control either associated or not with a specific method of the ground improvement, has to be evaluated in relation to the effects that will be (or may be) created in the surrounding environment. This is a particularly sensitive problem in urban areas where many restrictions can be met and various limitations may be imposed on the construction methods in order to preserve the state of the ground water outside the construction site. Thus the evaluations of the environmental effects and monitoring of the behavior of the ground around the site of the work shall be considered as equal importance problem as the water control within the construction site.

It can be emphasized that the geotechnical control of the ground water effects shall be considered as a long term assignments for large size areas, and as a short term problem for individual construction sites and their



- ① Clay deposit
 - ② Weathered claystone
- SCN. a-a'

FIG.2

immediate environments.

The most common methods applicable in different ground conditions and ground water states are :

Ground impermeabilization - can be achieved by pressure grouting, soil replacement methods such as cast in situ soil cement or concrete cut-off walls, by placing interlocked piles or stabilized soil columns, by artificial soil freezing etc.

Ground strengthening - can be attained by densification, improved consolidation, by soil reinforcement methods, by thermal treatment, soil replacement methods etc.

Ground drainage - can be made effective by gravity drains or vacuum drains, through various methods such as inserted drains, sand column drains, well point drains etc. The by-side effects can be adequately controlled by the monitoring of the ground surface behavior and the ground water surface, by piezometric heads etc.

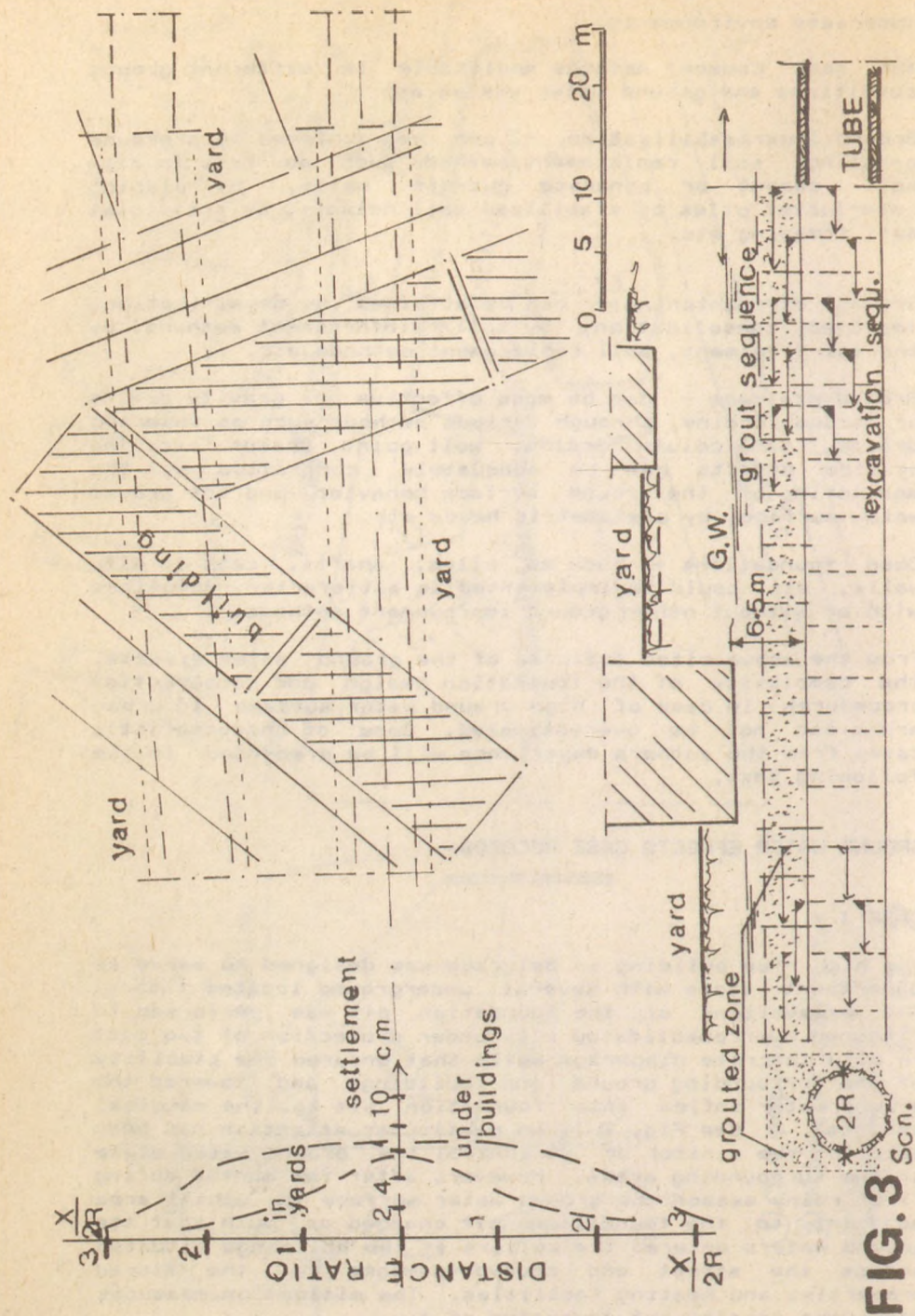
Deep foundations - such as piles, shafts, cast in situ walls, etc. could be implemented as satisfactory solutions with or without other ground improvement methods.

From the above cited features of the ground water effects, the complexity of the foundation design and construction procedures in case of high ground water surface in urban areas can not be overestimated. Some of characteristic cases from the authors experience will be presented in the following text.

GROUND WATER EFFECTS CASE RECORDS

CASE 1

The high rise building in Belgrade was designed to serve as department store with several underground located floors. The excavations of the foundation pit was performed in fissured overconsolidated clay under protection of two cast in situ concrete diaphragm walls that ensured the stability of the surrounding ground and buildings and lowered the ground water inflow into foundation pit to the marginal amounts. (See Fig. 2). No particular attention had been paid to the monitor or to control the ground water state in the surrounding areas. However, after few months during first rainy season the ground water surface in uphill area adjacent to the foundation pit changed so much that the ground waters entered the cellars of the buildings situated across the street and caused damages to the stored properties and heating facilities. The mitigation measures that were carried out consisted of :



- forming up of the chain of vertical drainage sumps located just in front of the concrete cut-off wall around the foundation pit, and
- constructing the drainage trench all along the cast in situ wall aiming to intercept all existing ground waters in front of the impervious wall barrier.

These measures lowered the ground waters to the previous elevation and ensured permanent dry conditions in endangered spaces (cellars).

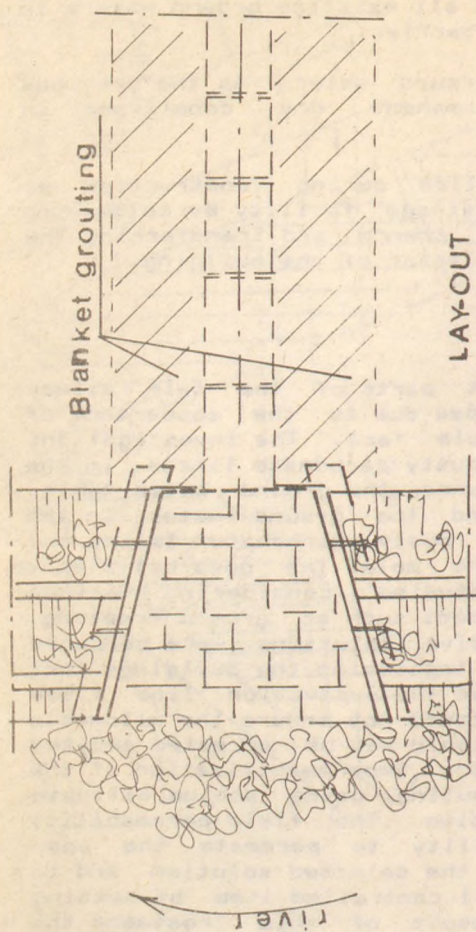
The temporary solution applied during construction was converted into a permanent drainage facility by collecting the waters from the drainage trench and transferring the seepage to the main sewer collector of the building.

CASE 2

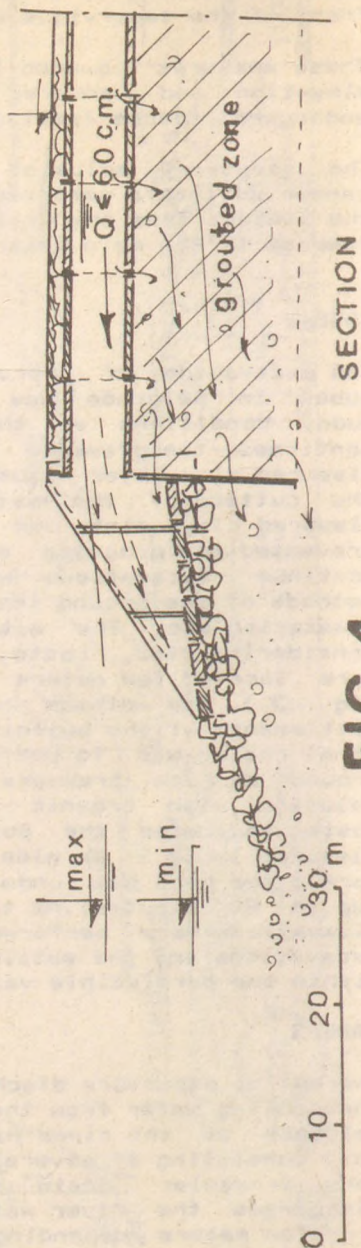
The excavation of the outlet parts of the twin railway tubes in Belgrade was stopped due to the appearance of muddy conditions at the mole face. The investigations confirmed the presence of dusty carbonate lances in the fissured clay layer situated under the ground water table. The cutters of the mole and the ground water in the fissured clay disturbed the dusty carbonate lances and prevented the progress of the mole. The possibilities to continue excavations was studied considering various methods of the ground improvement such as ground freezing, dewatering etc. The alternative solutions were compared considering time, costs, and impacts on the buildings that were located few meters above the excavation line (See Fig. 3). The methods which could not ensure the allowable settlements of the buildings could not be accepted and the final choice was to perform the impermeabilization of the ground by low pressure grouting using sodium silicate solution with organic reactive .The field permeability tests indicated the possibility to permeate the open fissures 1 to 3 mm wide by the selected solution and to form the hard gel under well controlled time of setting (up to 20 minutes). As the result of this treatment the excavations were performed in sequences of pregrouting and excavations and the settlements of the buildings were well within the permissible values of few cm.

CASE 3

The outlet structure discharging up to 60 cu.m. per sec. of the cooling water from the thermal plant situated near by Belgrade at the river bank ,was founded on the alluvial soil consisting of several layers of silty-sandy gravels of very irregular grain size composition. During the discharges the river water levels changed the elevations for few meters depending on the river water flow thus creating the seepage gradients oriented from the river banks towards the river. The static gradients were enlarged



LAYOUT



SECTION

FIG.4

by the subpressures developed in the plunge pool area during large discharges and low river levels, and the gradual washing out of soil fines has taken place during several years before the settlements of the discharge pipe was recognized at the ground surface. The settlements were associated with the cracking and openings of the concrete pipe joints and additional water leakage from the conduit to the foundation soil. (See Fig. 4)

The remedial works consisted in blanket pressure grouting using thick cement based mix with addition of sand. The grouting of the area around and beneath the conduit stabilized and strengthened the foundation strata and made them sufficiently impervious thus preventing further deterioration by the seepage and leakage waters.

CASE 4

The multistory buildings in a suburb of Belgrade town were constructed on raft foundations over 10 m thick deposit of loessial silty-sandy soil cemented by carbonate and clayey matters. The deposit was generally known as the foundation soil liable to the structural collapse phenomena after its saturation, but laboratory tests carried out on tube driven samples failed to reconfirm this particular nature of the soil on the samples taken from the areas beneath the building foundation areas. However, after one year from the end of construction the progressive settlements were recorded at its one side (See Fig. 5). The additional investigations carried out after this evidence indicated significant changes in the moisture content of the foundation soil underlaying the cited side of the building. The laboratory tests carried out on hand cut and trimmed samples confirmed the collapsible nature of the soil under the recorded changes in the moisture content. It has been found that the unexpected changes in the moisture content of the foundation soil were caused by the gradual seepage that occurred from the sewer mains and water supply lines passing near by the settled part of the building. Water from this source has caused the gradual increase of the moisture content by suction and slow but consistent accumulation of the moisture under the building foundations. The repeated cone penetration tests in the zones of increased moisture content also confirmed the significant decrease of the in situ soil resistance. The parts of the foundation soil situated underneath the zone of increased moisture content which remained unaffected by the water seepage did not change their resistance to cone penetration. The remedial measures that were implemented after these findings consisted in repair of the conduits from which the leakage had occurred, and securing the repaired conduits against uncontrolled leakage of the water. The measures ensured the building against further development of uncontrolled settlements.

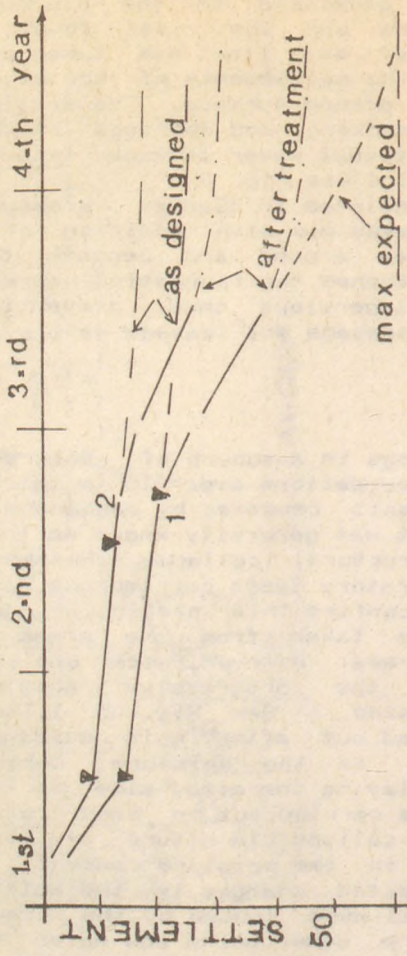
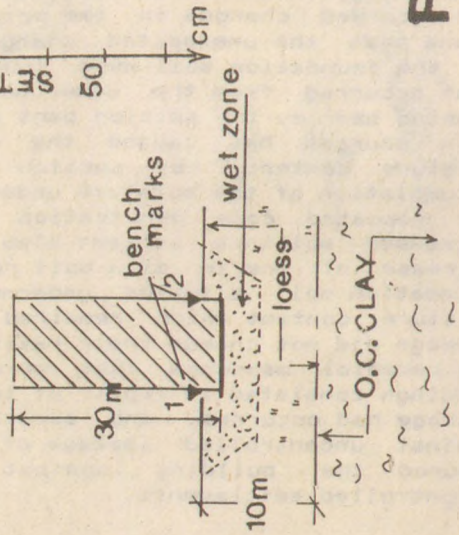
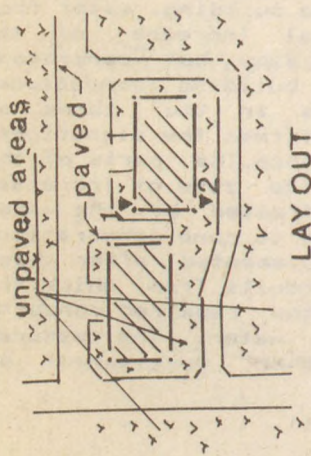


FIG. 5

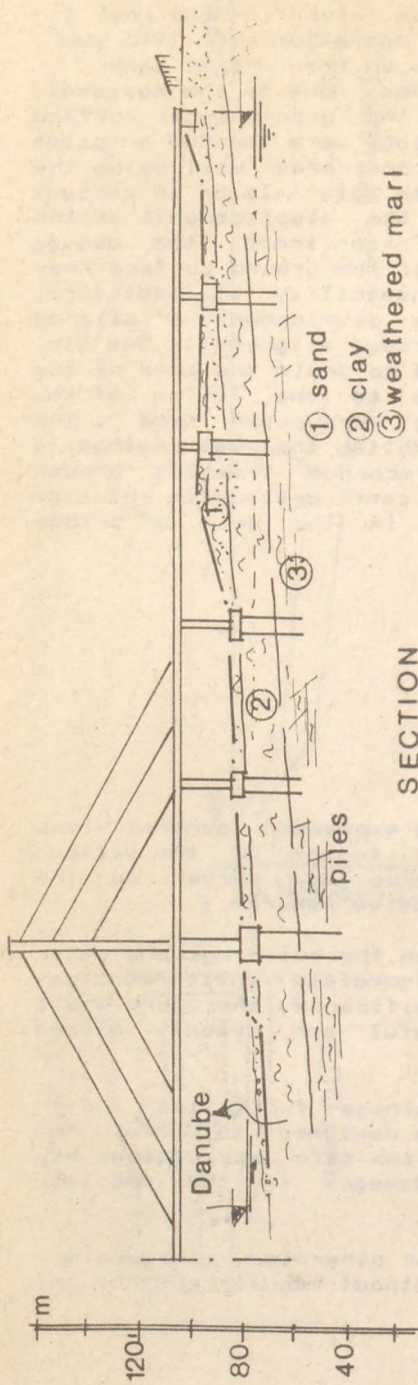
CASE 5

The new highway bridge over Danube river in the Novi Sad town was constructed and put into operation in 1981 year. The area in vicinity of the bridge on the right bank was known as liable to land slides due to the seasonal changes in the river water levels and ground water surface in the bank. The piers of the bridge were founded on piles lowered to the depth that were considered well below the possible slip surfaces in overburden materials or in ancient sliding zones. The monitoring of the displacements of the piers during last 10 years confirmed the design assumptions, but the displacements of the ground surface near by the bridge indicated the possibility of additional loadings on the piers due to the development of sliding phenomena in the vicinity of the bridge alignment. (See Fig. 6). The drainage galleries designed to drain the area of the piers could not be completed due to the inflow of the running saturated sand into the excavation face. The studies were carried out to establish the best method to secure the piers against the recorded treating ground displacements and to provide more confirmations in relation to the mechanism of the slides in the area of bridge abutments.

CONCLUSIONS

The presented case records and the expressed general views on the subject of ground water impact on the various structures and construction works may serve as the background for the following conclusive remarks :

- The impact of the ground water on the buildings and other structures their foundations and immediate environment may take very different forms and significance, therefore shall be a subject of specific, careful and broadly minded considerations.
- The usually performed drainage facilities and-or impermeabilization works which are designed to secure the foundation pit in dry conditions and safe excavations may become a cause of significant damages in the immediate environment.
- The long term drainage works or otherwise achieved dry conditions in foundation soils without regular control may



SECTION

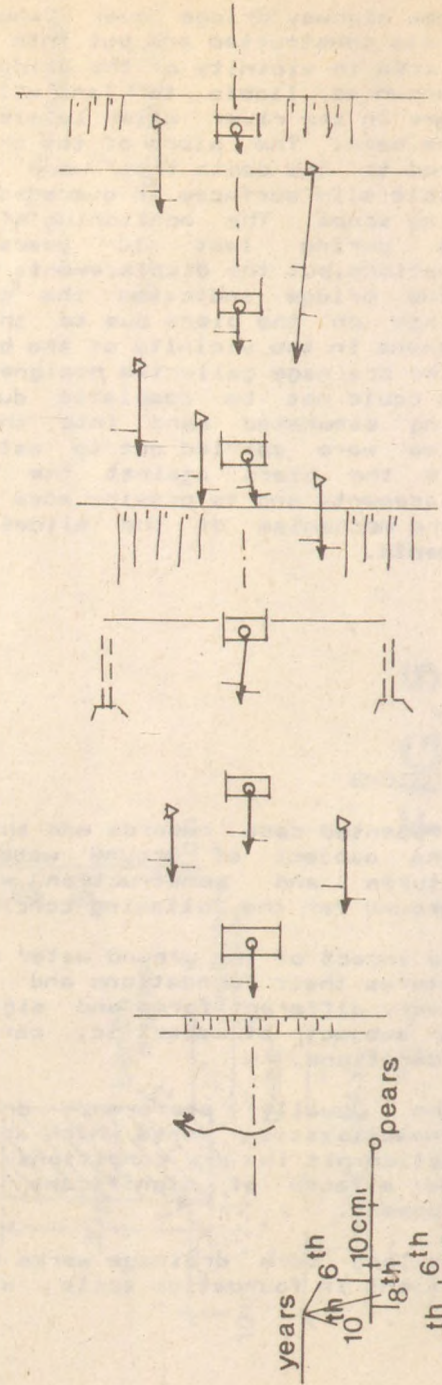


FIG.6

LAY, OUT

DISPLACEMENTS

undergo the gradual changes due to the changes in ground water states under influences outside the secured areas. Such changes may lead to the unfavorable effects on the soil properties and unacceptable behavior of building foundations previously designed for initial states of the soil.

- The active role of seasonal changes in the ground water states in conjunction with other climate dependent influences like freezing temperatures, suction effects, precipitations and water penetration into the ground, gradual washing out of fines or dissolving of salts in the ground shall be a subject of careful attention.

- The changes in the large rivers flows that are causing differences in the ground water states in the river banks shall not be oversighted from various aspects particularly in relation to pollution effects.

The cited comprehensive role of the ground waters in designing, constructing and maintaining the foundations shall not be considered as complete coverage of the subject, but as a attempt of the author to present his experience with the problems and to indicate the possible solutions. The main lesson that can be learned, is that the oversight of the ground water impact may lead to expensive mitigation measures and conditions that may jeopardize safety of the existing structures and facilities..

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GEOTECHNICS AND ENVIRONMENTAL PROTECTION

Prof Petrasovits, Géza

Geotechnical Department of Budapest Technical University

1 INTRODUCTION

The scope of Geotechnics has traditionally covered a participation in the design of engineering structures of various uses with the prime goal to provide a solution for the "safe" foundation of these structures (buildings, underground structures, retaining walls, dams and earthworks etc.). Whereby - with due allowance for rational probabilistic change in the environment as well as for rare incidental events (floods, earthquakes etc.) - the proper use and functioning of the structure is secured for its entire life-time.

The methods and solutions developed in recent decades do not always prove adequate to meet the above requirements mainly because of the emergence of the environmental factors such as chemically active agents precipitating from the air or water, or leached out of the soil, which all tend to corrode the building material of our structures and thereby introduce an element of hazard into the biological and technical conditions of human society which were formerly believed to be naturally safe.

Industrial development and ensuing rapid civilization have brought about in the majority of European countries the by-product of an enormous mass of waste materials, the disposal of which poses a formidable task among others for the civil engineers as well, if we are to safeguard and preserve normal conditions of human life. In many countries the situation has already reached an alarmingly intolerable level, calling for immediate action to remedy the situation.

The Tokyo '77 International Conference on Soil Mechanics and Foundation Engineering devoted a special session to the role of Geotechnics in the protection of the environment. Since then, the topic of environment has been prominently on the agenda of all the 4-yearly international conferences. It appears that in this region of Europe where we live the problem of the environment now receives the attention it deserves. The disposed and safe storage of harmful wastes, especially of radioactive wastes, is not merely a technical matter; it also requires, particularly in densely populated areas near the planned disposal site, a proper information of the public in order to win their cooperation. Quite recently, in Hungary, plans for establishing a burial place for radioactive wastes aroused heated public debate, demonstrating the fact that on such a vital issue to reach a decision requires for more than mere technical reasoning backed by geotechnical, geological and hydrological considerations.

The paper gives a brief review of the geotechnical aspects of environment protection, also dealing with the methods of disposal and storage of harmful wastes, further most with the determination of soil parameters underlying the design of disposal works such as permeability and anisotropy in permeability.

2 RELATIONSHIP BETWEEN ENVIRONMENTAL PROTECTION AND GEOTECHNICS

The effects that endanger the environment can be grouped in two categories Fig. 1. As far as Hungary is concerned the "Natural" group is only of minor importance except perhaps agents due to "Fluid flow" but even these are only remotely related to Geotechnics.

For this reason we shall deal only with the man-made environmental hazards.

Nearly all activities produce refuse. With the industrial society, wastes have increased greatly in quantity and multiplied beyond all possible forecasts in type. While, until recently, wastes were, compatible with the recycling capability of the planet, this is no longer so today. In the past, the amounts of wastes have been small and therefore their presence has gone unnoticed. Today a huge, varied and non-digestible

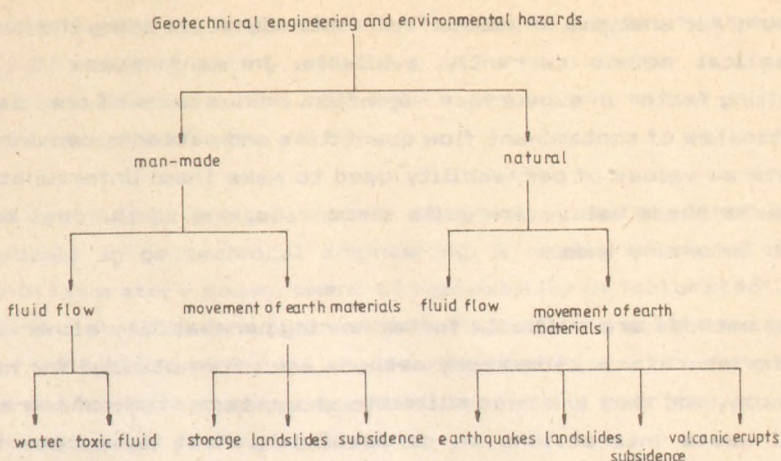


Figure 1

Association of Geotechnical Engineering with Environmental Hazards
(Morgenstern, 1985)

mass of wastes has little place to go apart from being stored on ground surface.

The definition of the hazardous waste in this paper will cover those wastes that are:

- ignitable (paint solvents)
- corrosive (acids),
- reactive (explosives),
- toxic (giving health problems) and
- radioactive (industrial or mining).

3 MEASURING METHODS FOR DETERMINATION OF THE PERMEABILITY OF COHESIVE SOILS

Practice in measurement of permeability is variable, with resulting variation in measured values of permeability.

The permeability of a clay soil is a major parameter necessary for the design of clay barriers. A number of hazardous waste disposal regulations stipulate maximum values of permeability, typically in the range of 10^{-6} cm/sec to 10^{-8} cm/sec, above which a clay material is unacceptable

for use in isolation of toxic waste materials. Permeability is also necessary for analysis of subsurface waste migration using the numerous mathematical models currently available. In many cases it is the controlling factor in subsurface migration. Evaluations of clay barriers and estimates of contaminant flow quantities and patterns can only be as accurate as values of permeability used to make them. Unfortunately, in many cases these values are quite inaccurate, due to the test methods used to determine them.

Several methods are available for measuring permeability either in-situ or in the laboratory. Laboratory methods are often utilized for reasons of economy, and they are best suited to short-term study of the effects of soil-waste interactions on permeability. Most measurements are completed on samples assumed to be saturated rather than partially saturated.

Within geotechnical engineering the permeability of a soil sample is determined in the laboratory in either

- a fixed wall apparatus,
- a triaxial cell or
- a consolidometer.

As different aggressive chemicals are used, standard equipment can not be use.

The opinions on which test to use are diverse. Some researchers claim that a fixed wall cell will give erroneous results due to leakage along the sides, while others say that the decreasing diameter in a triaxial cell is inconsistent with the boundary conditions present in the field. Undoubtedly, the consolidometer with its boundary conditions (no lateral deformation, one-dimensional flow) has an effective stress distribution resembling what is present under full scale conditions. The possible draw-back of a fixed wall side effect could be limited by using an annular seal.

Beside of this value of the permeability is effected by the testing procedure as well. The laboratory investigations are shown, that substantial influences have on the hydraulic conductivity values:

- the hydraulic gradient,
- methods of sample saturation and verification of degree of saturation,
- quality of permeant water,
- sample consolidation during test and
- time (thixotropic) effects.

Permeability of soils is an extremely variable parameter, possibly the most variable in geotechnical engineering. A review of the state of practice in laboratory measurement of permeability of "saturated" soils indicated that few standard test methods are available and the extensive degree of variation in the test procedures used in practice can result in measurements of saturated permeability varying over two or three orders of magnitude for a given soil.

Waste permeants can affect soils in a variety of ways resulting in either net increases or decreases in hydraulic conductivity as compared to water as a permeant. Therefore, in addition to measuring permeability it is important to analyze the soil and permeant fluid chemistry both before and after permeation. This will provide an indication of the interactions occurring between the waste permeants and soils, and will also aid in the use of the data in the field. Even with careful laboratory testing procedures, there is still an inherent variation between laboratory and field conditions, and this can have large effects on permeability.

Much better conclusion can be made, if we are informed about the permeability anisotropy of the given soil. In the next we introduce a two-way permeability apparatus and a calculation method for determination of the permeability coefficient.

The degree of permeability anisotropy - the ratio of maximum to minimum permeability - has been to carry out laboratory permeability tests on two samples of the soil. Difficulties could arise in this case if the two samples were not exactly the same in terms of permeability characteristics and if the direction of the maximum permeability, for example along the stratification, was not visibly evident.

By Moore was introduced a two-way permeameter that facilitates measurement of the degree of permeability anisotropy on a single soil

sample. The test does not require knowledge of the direction of maximum permeability. A cross-section of the two-way permeameter is shown in Fig. 2. The soil sample is a cube with a side of 150 mm. The porous stones have a length equal to that of the sample. When compacted soil is used, a firm contact between the soil and the sides of the permeameter is achieved. No difficulty is encountered with seepage around the boundary between the soil and the sides of the permeameter, however, some precautions need to be taken to prevent the passage of water around the sides of the sides of the sample:

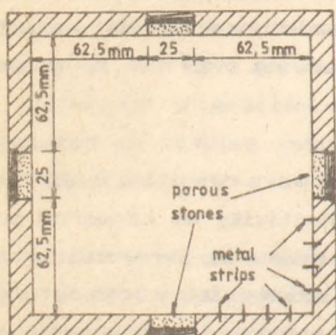


Figure 2

Cross-section of a two-way permeameter for a cubic soil sample

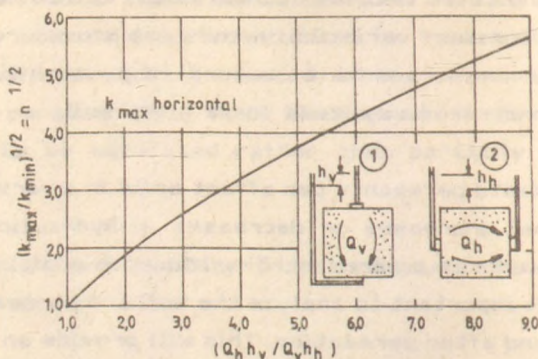


Figure 3

Seepage in a two-way permeameter

- using a permeameter with one moveable side to reduce the air gap around the soil sample,
- filling the air gap around the sample with an impervious material such as wax, or
- using thin metal strips that penetrate the soil sample over its whole length and extend around the perimeter of the sample. Fig. 2 illustrates the metal strips over one quadrant.

The test is carried out in two stages. For the first stage, as shown in Fig. 3, steady state seepage is permitted in the vertical direction only, the rate of seepage flow and total head loss across the sample being designated Q_v and h_v , respectively. For the second stage, Q_v is stopped and flow is developed in the horizontal direction by imposing a total head difference h_h across the sample. The resulting steady state flow in the

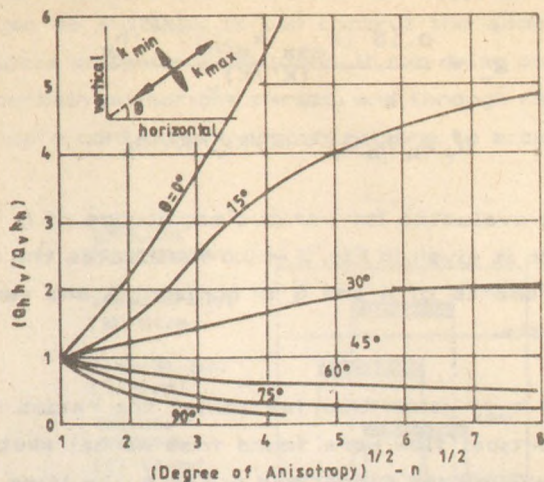


Figure 4

Effect on seepage parameters of various orientations of $k_{\max}$

horizontal direction is termed Q_h .

For the special case where the direction of maximum permeability $k_{\max}$ is parallel to one of the sides of the permeameter, an exact solution for the flow through the soil is available from Pavlovsky and is described in Harr. With reference to the testing orientation of the permeameter, it may be supposed that $k_{\max}$ is in the horizontal direction and $k_{\min}$ in the vertical direction.

For steady state horizontal flow the Pavlovsky solution for Q_h is:

$$Q_h = \frac{0.15 \cdot (k_{\max} \cdot k_{\min})^{1/2} \cdot h_h}{(K'/K)_h} \quad (1)$$

where

0.15 - permeameter length in meters,

K - the complete elliptic integral of the first kind with modulus m ,
and

K' - the complete elliptic integral of the first kind with modulus m ,
equal to $(1 - m^2)^{1/2}$.

The modulus m is given by a Jacobian elliptic function:

$$m = \lambda \cdot \sin(\Lambda/6, \lambda)$$

For steady state vertical flow the *Pavlovsky* solution for Q_v is:

$$Q_v = \frac{0.15 \cdot (k_{\max} \cdot k_{\min})^{1/2} \cdot h_v}{(K'/K)_v} \quad (2)$$

$$h_v \cdot Q_h / h_h \cdot Q_v = (K'/K)_v / (K'/K)_h \quad (3)$$

Equation 3 can be evaluated for various magnitudes of $\underline{n}$. The result of such an evaluation is given in Fig. 3 which facilitates the determination of $\underline{n}$ from measurements of $\underline{h}$ and $\underline{Q}$ in horizontal and vertical steady state seepage tests.

For directions of $k_{\max}$ other than horizontal the ratios of $\underline{h}$ to $\underline{Q}$ for horizontal and vertical flow were found from manual sketching of flow nets. The results have been summarized in Fig. 4. The lines marked $\theta = 0^\circ$ and $\theta = 90^\circ$ are based on the *Pavlovsky* solution described above.

4 SYNTHETIC AND CLAY-BASED BARRIERS

The common engineered pollution control barriers may be classified according to whether they are walls installed deep in the ground, or whether they are liners installed at shallow depth or directly on the surface of the ground. Although grouting and other procedures have application in the containment of waste at depth, the most common procedure is to install a slurry cut-off wall. Experience with the use of slurry walls to control seepage beneath water-retaining structures indicates clearly that their performance is controlled by details of installation rather than matters of design, provided that the slurry is compatible with the waste and hence long-term performance can only be guided by the sharing of experience.

Synthetic liners are impermeable and have been designed by geotechnical engineers to prevent large losses of stored fluids, to reduce pore pressures in an embankment and to inhibit erosion. Much of this design has proceeded empirically with the assumption that the liner is of such low permeability that little analysis is warranted. These procedures are not totally transferable to the design of pollution control barriers. As observed by *Folkes (1982)*, rational approaches to the design of pollution

control liners require an understanding of the role of the liner. The role of a liner can be multiple. It can control the amount of seepage and thereby mitigate pollution by dilution. It can delay pollution by containment for a certain prescribed period, and through retardation effects, it can attenuate contaminant concentrations to acceptable levels.

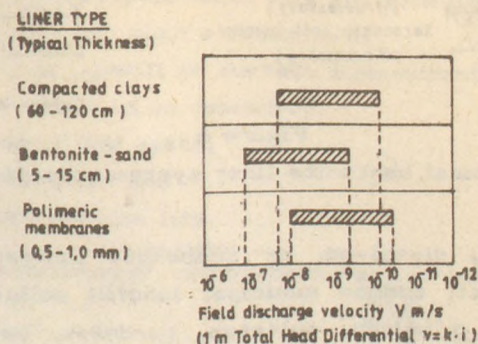


Figure 5

Linear discharge velocities

The fluid does pass through the synthetic liner by diffusion under vapour pressure differentials, osmosis due to chemical gradients, and absorption due to polymer solubility. The amount are negligible for most practical purposes. However leakage in-situ does arise from pinholes, punctures, tears, poor seams and otherwise degraded or damaged sections. This results in concentrated streams of contamination and is not really the equivalent of permeability. For a more meaningful comparison of liner materials, liner thickness should also be considered. Ranges of discharge velocity for various liner materials are shown in Fig. 5 (Folkes, 1982).

In several cases to present the leakage was applied a double membrane liner with a leachate collection system between the two liners that acts as a leak collector.

Wastes are primarily divided into liquid and solid wastes, and secondarily into municipal, industrial, special wastes, and radioactive wastes. Municipal liquid waste is primarily domestic sewage. Municipal solid waste is that refuse from residential and commercial sources that is collected in garbage cans and trash bins. The solid wastes produce leachates which

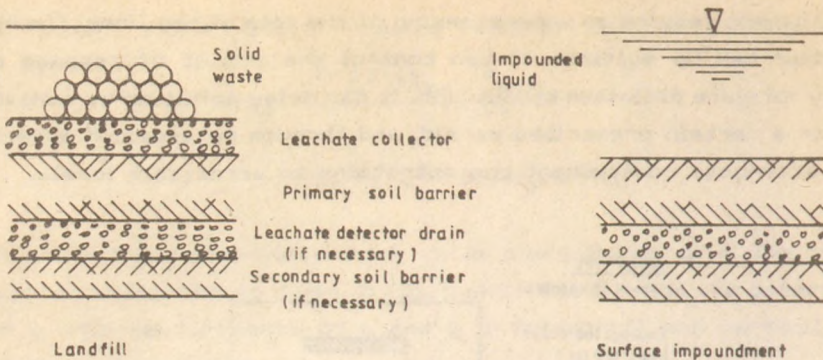


Figure 6
Typical bentonite liner system concepts

contain extracted, dissolved, or suspended inorganic and organic materials. The most, common municipal landfill pollutants are total dissolved solids, chlorides, sulfates, hardness, methane, oils and greases, plastic and other organics, in addition to bacteria and algae (EPA, 1980).

Industrial wastes include the following four physical classes:

- aqueous - inorganic,
- aqueous - organic,
- organic or
- sludges.

Leachates are generated in solid waste disposal sites when water is allowed to infiltrate the site. Inorganic and organic materials are then extracted and dissolved from the waste as the water percolates through the decomposing refuse. The leachate mounds on the bottom of the landfill and starts seeping through the bottom as soon as enough hydrostatic head has built up. The composition of the leachate depends on the type and quantity of waste. Dispersion of the leachate plume is three dimensional in that a plume not only extends laterally, but often sinks. Given enough time, the plume can disperse over many square kilometers, contaminating the ground water supply of entire communities.

If at all possible, liquid and solid waste disposal sites are constructed in geologically favorable areas. Such sites are usually associated with fine grained soils that restrict ground water flow. The historical high ground water table should be at least 1.5 m (1 to 10 m, depending on regulation) below the bottom of the facility.

A liner system must be designed which fits the site and specific waste conditions. The criteria for such a design are:

- type of facility (i. e. landfill or surface impoundment),
- general nature of waste to be deposited,
- approximate volume of the waste,
- approximate site location and
- anticipated facility service life.

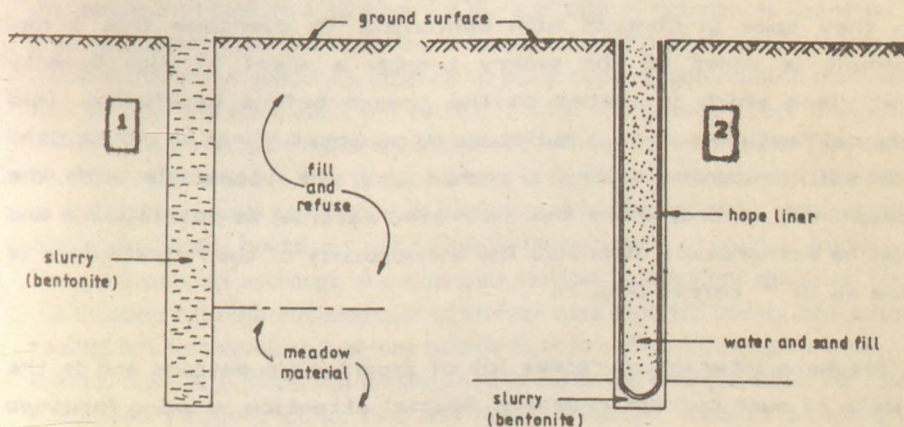


Figure 7

Excavated trench before placement of the liner
and cross-section of the trench with liner

Each liner design is unique, but trends are apparent and typical liner systems which satisfy many conditions have been developed. Fig. 6 exemplifies two such systems. The landfill design concept utilizes a gravel or sand drainage blanket (leachate collector) which removes the leachate from the upper surface. A leachate collector system may be installed here if only one lift of soil bentonite is placed underneath. This blanket serves three purposes:

- to protect the liner,
- to collect and route leachate and
- to retain the moisture which prevents the bentonite from drying out.

In recent years slurry trenches have become the principle type of barrier to mitigate pollution of ground water, especially when a barrier is used as a remedial measure. An impervious layer of lightly packed bentonite molecules forms in a relatively short period of time on soil slurry interface and acts as a water-tight membrane, stopping flow of slurry out of the trench and preventing ground water from entering the trench. During the excavation the filter cake formed is usually three millimeters thick. This thin barrier is an effective barrier to water movement since the permeability of the filter cake can be as low as 10^{-9} cm/sec.

Among different hazardous waste sites it is possible to find some chemicals which are not compatible with bentonite, increasing permeability when they come in contact with bentonite. To overcome this a new component is added to the slurry trench: a sheet of High Density Polyethylene which is placed in the trench before backfilling. This additional feature serves a multitude of purposes. First it can be used where soil bentonite slurry trenches are not compatible with the contaminants; second, where the excavated material is not suitable and cannot be economically disposed. The permeability of the Polyethylene is as low as 10^{-12} cm/sec (Fig. 7).

Soil-leachate interaction raises lot of important questions and is the subjects of much current research. Special attention is being focussed on the effects of hydrocarbons and other organic materials on the hydraulic conductivity of compacted clay.

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Geotechnics - Research and Reality

by U. Smolczyk, Stuttgart

An international gathering like this 9th Danube-European Congress is organized with the intention to provide a platform for the exchange of experiences and views among geotechnical engineers having different backgrounds of professional knowledge due to different systems of education, different habits in applying geotechnical methods, different national and commercial procedures. It is the very positive effect of mutual benevolence that we often do not become aware of such factual differences. They should not be over-emphasized but they should be kept in mind when we look into the difficulties to make congresses a real source of professional inspiration and a step towards overcoming traditional boundaries. The growing insignificance of political frontiers within Europe fosters the exchange of engineering services between our countries. Until now the situation was different: engineering services were exported mainly from a European country into less developed regions outside Europe - often following the paths of early colonial links. A geotechnical investigation in Africa for example would not follow any established standards of its own but would rather be executed according to the know-how of the involved consultant who often is - or at least for a long time has been - a European geotechnical expert from that country which the region had been subject to during the times of colonialism.

This type of transfer of know-how will certainly continue until appropriate engineering and investigation facilities are available in those regions. But in the meantime a new and quickly growing exchange of geotechnical engineering services amongst industrialized European (and certainly not only European) countries can be observed. This, in turn, makes us realize that the terminology used in our profession is universal. A pile is a pile is a pile from the Atlantic Ocean coast to the Ural mountains and from Finland down to Sicily. However, the methods of forecasting the bearing capacity of a pile and the procedures of execution and the performance tests - that is, all the details of practice that make a pile a reliable supporting member of a structure - in fact vary from region to region, from the mountains to the coastal areas, from the cities to the countryside etc. Or let us take another example: the definitions of shear parameters are internationally acknowledged and are found in any handbook of any country. Notwith-

standing the apparent similarities in the testing practice in different countries, we would be surprised about the deviations in results if the same kind of soil were tested by ten different institutes even in the same country. This at least has been the experience so far with the so-called ring-analyses. Small countries with but one dominating soil testing institution will not have such frustrating experiences, but they surely would, if they tried to compare among each other.

The same scatter holds when geotechnical design procedures are compared. During the work done for ten years to draft a European Code of Practice for geotechnical design, simple examples as indicated in fig.1 and fig.2 were calculated according to existing national practice. The input data were given for each design situation as complete as possible and the same routine algorithms were applied in the various calculations. In spite of this, the results in terms of dimensioning the foundation elements showed a wide scatter. The reason for this is the national peculiarities or special requirements to those routine design procedures. To demonstrate this, let us take the bearing capacity calculation for a rectangular footing. Apart from the French who do it by use of the pressiometer, all nations would agree that the well-known Terzaghi equation with its three terms to be summed be an established and reliable analytical procedure. However, one will find a significant dissent on how to introduce all those additional factors needed to consider the influences of load inclination, eccentricity, shape of the base, inclination of the base, sloping of the ground, embedment depth. There is little harmony to be achieved on this in Europe and certainly less worldwide. And this is the reality in spite of decades of international research.

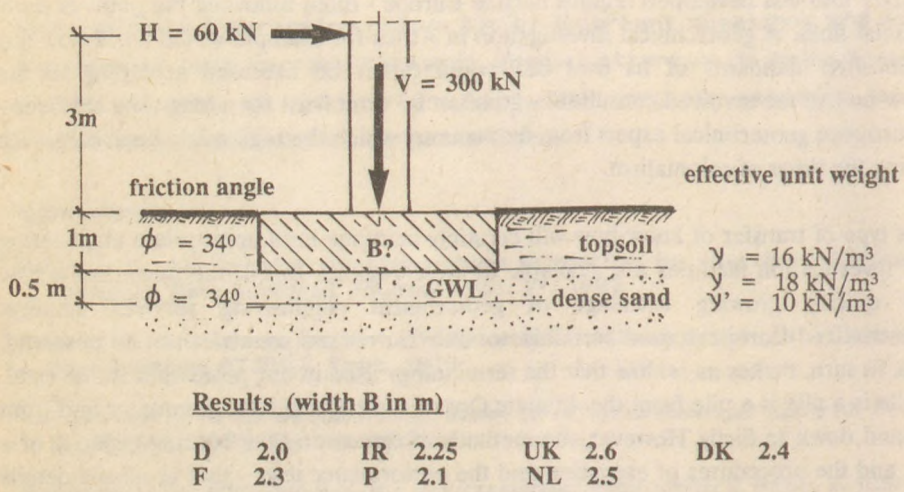
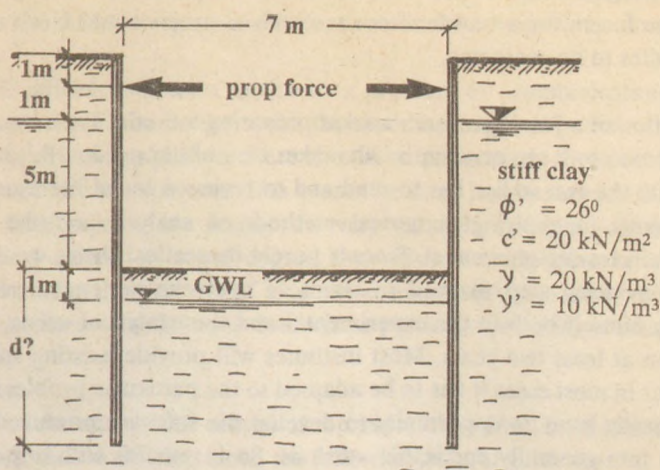


Fig. 1 Comparative analysis of a square footing according to various existing national codes in Europe



Results

	F	D	DK	P	UK	NL	IR
length (m)	16.1	12.0	13.8	16.2	14.4	14.0	12.0
bending moment (kNm/m)	279	210	699	502	550	670	121
sheetpile weight (kg/m)	2504	1866	3276	2520	3427		1866
prop force (kN/m)	82	62	239	191	226	207	85

Fig. 2 Comparative analysis of a retaining wall according to various existing national codes in Europe

Therefore I choose the somewhat provocative title "research and reality". There is a considerable gap between these two fields of geotechnical activity, and I feel this gap is steadily widening instead of closing. We observe especially with international congresses that practitioners lose their professional interest to attend them - many of them have told me that the information which can be taken home from such events is not worth the expense. This is a serious complaint and must induce organizers to think about how to improve the attractiveness of their congresses.

This is difficult because modern geotechnics is a very difficult matter. The scientific progress of geotechnics is achieved worldwide by ambitious hard-working young engineers in universities and big institutions whilst the technological progress depends mainly on the creativity of free enterprises subject to the pressure of competition and is mainly done by engineers who have already gained professional experience. There are certainly favourable examples where these two activities are joined in a mutual

exchange of knowledge but this I believe is rather the exception. Conventions like this should provide the forum for a transfer from research to practice and vice versa but I foresee many hurdles to be overcome.

Imagine the situation of a young research worker preparing a thesis. The research topic may have been chosen by a senior scientist who takes care of her or him. But to attain a state of workability the researcher has to read and to review a lot of literature, has to achieve special skills in applying numerical methods of analysis and the facilities provided by modern testing equipment. Since a purely theoretical thesis would not be accepted as a convincing step forward in terms of geotechnical engineering, he is obliged to qualify himself both in the experimental and the analytical arena. This will normally take him at least two years. Most institutes will provide existing software to their engineers but in most cases it has to be adapted to the particular problem. Scientific institutes generally have no opportunity to develop the software produced by their research workers into generally applicable versions. So it remains with any following user to introduce appropriate changes to the program according to his own findings. Sometimes such action would change the program to such an extent that it seems preferable to write a new program to avoid numerical mistakes.

After a couple of years, our candidate feels that his program is reliable and that its results yield a sufficient agreement with the test data from the experiments, which he hopefully also has finished. So he writes his thesis with the conclusion that his findings may help to solve a problem of geotechnical practice in a better way than before. If his professor has the same attitude, he will help him to publish the essence of the thesis in a journal or to send it to a geotechnical congress. But, then, what happens thereafter? Some research findings will get public recognition and will become referred to in future. But that is where the story mostly ends. Modern geotechnical research is generally on a high level of sophistication and it is even for an experienced reader very cumbersome to decide on the reliability of the results presented to him in a journal or at a congress. He can compare the input data and the applied material equations to his own experience but he could evaluate the correctness of the calculations and the appropriateness of the solutions only by an independent computation. By just listening or reading the evidence this will generally not be possible.

When he takes part in a discussion, our young woman or man will - for the same reasons - find it difficult to meet a partner who can argue with him or against him in a truly profound manner which helps him to avoid mistakes and dead-ends.

And the practitioner in such a session sits and listens and cannot understand the meaning for his own work or he returns home with some unreflected new ideas in his mind and applies them like using a black box.

We definitely ought to improve the effectiveness and efficiency of geotechnical research

bridging the gap between research and practice. Research costs us a lot of money and therefore we should feel responsible not to waste it. How can this be done?

The ISSMFE officials will term this to be a problem of communication. But how can persons communicate who do not speak the same language? When a research report or a thesis is finished and published, there should be someone to check on its applicability to current geotechnical problems. This cannot be done just by state-of-the-art reports of congresses. Such reports are mainly the very valuable and time-consuming work of individuals and reflect to some degree the reporter's subjective opinion. The only instrument to have the process of evaluation and translation done, I believe, is the technical committee established to deal with a particular problem of practical (or scientific) interest.

A technical committee should consist of representatives from both science and practice and their work should develop along the following general guidelines:

- (i) review the existing knowledge which may be suited to answer the questions under consideration, and evaluate it;
- (ii) define the needs for further research and address the institutions who may be able fill these gaps;
- (iii) communicate the findings at regular time intervals to those who review the state of the knowledge at international congresses (if this is not done by the chairman of the committee himself).

ETC 1 Stabilization of Landslides in Europe
(Chairman: Togrol, Turkey)

ETC2 Earthquake Engineering Problems in Europe
(Chairman: Viggiani, Italy)

ETC3 Piles
(Chairman: Franke, Germany)

ETC4 Pressuremeter Test
(Chairman: Gambin, France)

ETC5 Laboratory Testing
(Chairman: De Quelerij, Netherlands)

ETC6 Reinforced Soil
(Chairman: McGown, U.K.)

ETC7 Numerical Methods in Geotechnical Engineering
(Chairman: Sagaseta, Spain)

ETC8 Geotechnics of Landfill Design and Remedial Works
(Chairman: Jessberger, Germany)

Fig. 3 European Technical Committees of ISSMFE

It should be well understood that a technical committee is not able to solve a geotechnical problem by its own efforts. Its terms of reference should clearly state that its task is evaluation and interpretation. It is expected to digest the application rules of modern geotechnique in an operational manner.

In the meantime, quite a number of technical committees has been established internationally in addition to what was done earlier solely by national committees. Fig.3 shows the European Technical Committees of ISSMFE. It is felt that for economical reasons it is easier to run such an international group within Europe than a committee composed of participants from all over the world. This extension into the intercontinental scene may be considered as soon as the work of the European (or other regional) committees has reached a state of some maturity. A typical good example was the European Committee of Sounding which after many years of activity in Europe is now transformed into an international TC.

Next question: what happens with the report of such a technical committee after being published? Is there a mechanism that the conclusions and recommendations which have taken so much time and energy to be collected and drafted, will be brought to the knowledge of those who design the structures, execute the works on the sites, or have to control the performance of structures? In some countries we have such mechanisms but it is rather the exception limited to highly industrialized regions. The geotechnical member societies have so far made little effort in this direction. But a change in attitude became visible when the members of the ISSMFE Board at their last meeting discussed how the transfer of geotechnical knowledge could be realized in favour of less developed regions. A European proposal was welcomed to initiate training courses in geotechnics by using already existing training or research centres in developing regions. But it was also emphasized that this is not only a need for such regions but it should also be realized in our own countries to diminish the gap between university education and site practice.

In this view, I would draw your attention to a course which is held twice a year in Germany to train foremen of drilling contractors who are specialised in site investigations. The course is finished by a qualifying test which documents the official approval by the German member society. We found that it is not sufficient to publish standards on the quality of boreholes, drilling equipment and sampling and leave it to the profession to follow these guidelines or not. Of course, you will ask "Where is the supervising engineer who has to see to it that the principles of established codes are considered?" Yes, he is present and he is responsible. But taking care is much easier if the team at the drilling machine knows the practical essentials. A well-trained foreman checks the quality of his equipment before he starts its transport to the site.

The same holds with laboratory personnel. The equipment and know-how which is available in a soil mechanics laboratory depends on when it was established. This date

may be long ago, and the soil test engineer may have had his education during the fifties. Most institutes are so much involved in daily routine tasks that there is no time and concern to review ones state of knowledge or to exchange a test device. So I believe we as the responsible member societies of ISSMFE should start courses to promote professional knowledge as soon as we shall have obtained soil testing standards harmonized at least within Europe.

As to the developing countries, ISSMFE will take the responsibility in pioniering training courses for both the technical personnel and the site engineers. Model libraries have been provided to give a first input to promising institutions. But we are at the very first beginning of our duties. One of the future main tasks will be the collation of a manual of basic geotechnical knowledge. This should be published in all main languages and accomplished by slides, transparencies and films. Simple tests to demonstrate basic phenomena of soil mechanics should also be made available.

Let us close the gap between research and reality by improving education. This will help to avoid fundamental mistakes and make civil engineers and architects aware of the intricacies of the ground on which they want to build.

SECTION 1
EXCAVATIONS

Baugruben neben und unter bestehenden Gebäuden
Construction pits adjacent or beneath existing buildings

H. Brandl, Prof., Dr.techn., Dipl.-Ing.

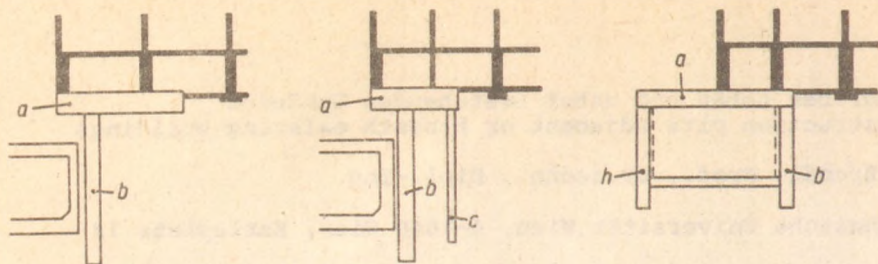
Technische Universität Wien, A-1040 Wien, Karlsplatz 13

ZUSAMMENFASSUNG: U-Bahn-Bauten und tiefe Baugruben in städtischen Ballungszentren erfordern Unterfahrungen und Unterfangungen von bestehenden Gebäuden, was höchste Anforderungen an den Spezialtiefbau stellt. Aus der Vielzahl technischer Möglichkeiten werden vor allem Pfähle, Schlitzwände und Injektionen herausgegriffen und Varianten für den Tunnelbau in offener Bauweise aufgezeigt. Eine Besonderheit stellt die Verbundwirkung zwischen Kleinbohrpfählen (Wurzelpfähle) und Boden dar, worauf theoretisch näher eingegangen wird. Die diversen Baumethoden werden mit Beispielen aus der Praxis veranschaulicht.

SYNOPSIS: Underground construction and deep excavations for construction pits in urban areas require undercrossing and underpinning of existing buildings - thus also requiring high standards from ground engineering. From the numerous technical alternatives mainly piles, diaphragm walls and grouting are taken as examples, focusing on the cut-and-cover method for underground tunnelling. Special systems are composite bodies consisting of small diameter injection piles (root piles) and the surrounding soil. A semi-empirical design method is derived from a theoretical analogon to reinforced concrete, and it is demonstrated by a calculating example. Several case histories from construction sites illustrate the application of special ground engineering methods.

1. EINLEITUNG

Die dichte Besiedelung städtischer Ballungszentren erfordert in zunehmendem Maße die Herstellung von Untertagebauten, wie U-Bahnen, Tiefgaragen oder Unterkellerungen im allgemeinen Sinne. Dabei ergeben sich zwangsläufig mehr oder weniger tiefe Baugruben unmittelbar neben oder unterhalb bestehender Bauwerke. Solche Tiefbauarbeiten gehören erfahrungsgemäß zu den diffizilsten Aufgaben des Bauwesens und erfordern eine enge Zusammenarbeit zwischen dem Grundbauer und dem Statiker bzw. konstruktiven Ingenieur.



- | | |
|------------------------------------|--|
| a) permanent girder grille or slab | f) underpinning r.c.beam |
| b) underpinning + pit wall | g) auxiliary piles |
| c) tension piles | h) frame leg (single piles) or wall (pit / tunnel) |
| d) temporary girder grille | i) diaphragm wall |
| e) auxiliary columns | |

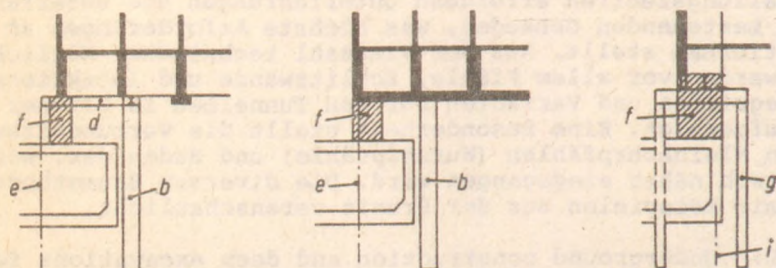


Abb. 1:

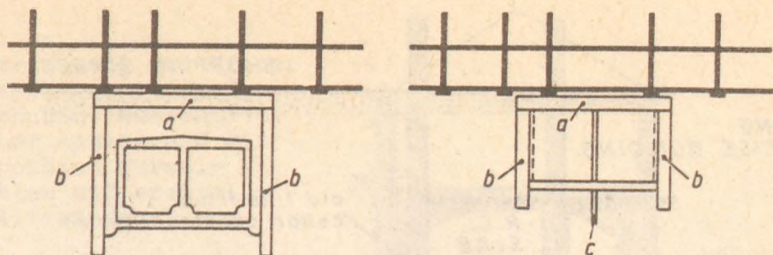
Einige Konstruktionsprinzipien (schematisch) für Teilunterfahrungen von Gebäuden in "offener" Bauweise

Tiefe Baugruben erfordern unter beengten Platzverhältnissen Baugrubenwände, welche im allgemeinen Fall sowohl horizontale als auch vertikale Kräfte zu übernehmen haben. Neben klassischen Bauwerksunterfangungen (schrittweise) kommen folgende Alternativen infrage:

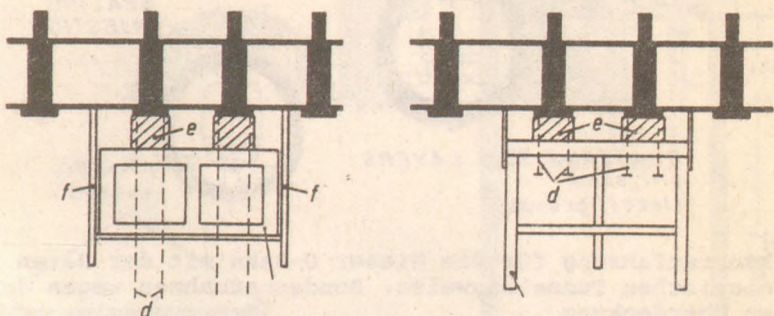
- Pfahlwände und Schlitzwände,
- Injektionen,
- Hochdruckbodenvermörtelung (jet grouting),
- Bodenvernagelungen (meist mit Spritzbetonabdeckung).

Bei der Unterfahrung bestehender Bauwerke kommen noch Schächte, Stollen, Tunnel, Rohrschirmdecken und andere Spezialmaßnahmen dazu. Einige wesentliche Konstruktionsprinzipien für "offene" Bauweise sind in den Abb. 1 und 2 zusammengefaßt (Baldauf, H. u. H. Timm, 1988). Die "geschlossene" Bauweise (bergmännische Untertunnelung) vermeidet zwar die Problematik der tiefen Baugruben, erfordert jedoch ebenfalls umfangreiche Maßnahmen, um eine unzulässige Beeinflussung bestehender Gebäude zu vermeiden - insbesondere bei sehr geringer Überdeckung (Abb. 3).

Im folgenden kann nur eine kleine Auswahl aus der Vielzahl der möglichen Baumaßnahmen behandelt werden. Der Schwerpunkt liegt auf Baugrubenwänden in Verbindung mit Bauwerksunterfangungen.



- a) girder grille or (prestressed) r.c. slab
 b) permanent underpinning wall
 c) row of columns (auxiliary)



- d) temporary underpinning (piles)
 e) permanent underpinning (r.c. beams)
 f) temporary pit wall

Abb. 2:

Einige Konstruktionsprinzipien (schematisch) für Vollunterfahrungen von Gebäuden in "offener" Bauweise (Baldauf & Timm, 1988)

2. SCHLITZWÄNDE

Schlitzwände bieten sich unter folgenden Voraussetzungen als Baugrubenwände an:

- hoher Grundwasserspiegel, wobei eine temporäre Grundwasserabsenkung während der Bauzeit vermieden werden soll;
- Übernahme großer Kräfte, vor allem seitlicher Erddrücke (starke Bewehrung), scheibenartige Versteifungen möglich);
- ausreichende Platzverhältnisse für das Schlitzwandgerät;
- keine größeren Hohlräume im Untergrund;
- sehr große Unterfangungstiefe.

Abb. 4 und 5 zeigen die Teilunterfangung des denkmalgeschützten Wiener Messeplatzes mit Schlitzwänden zur Herstellung eines mehrgeschoßigen U-Bahn-Bauwerks. Um die Tunneldecke unterhalb des Geländes betonieren zu können, war zunächst eine temporäre Abfangung durch Kleinbohrpfähle (Wurzelpfähle $\varnothing$ 24 cm) erforderlich. Knickhalterungen dienten zur Sicherung der Pfahlgruppen während des Teilaushubes bis auf

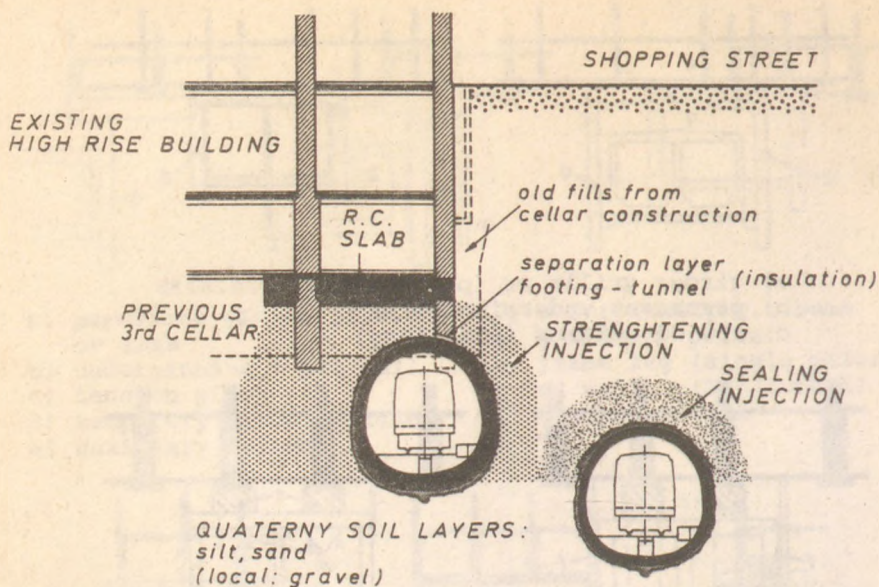


Abb. 3:
Bauwerksunterfahrung für die Wiener U-Bahn mit der Neuen Österreichischen Tunnelbauweise. Sondermaßnahmen wegen der geringen Überdeckung

ca. 4 m unter Pfahlkopfniveau (Abb. 4). Zwischen Tunneldecke und neuem Bauwerksfundament wurden hydraulische Pressen angeordnet, um eine Vorbelastung aufbringen und Setzungs-differenzen ausgleichen zu können. Nach dem Durchtrennen der Wurzelpfähle erfolgte die Lastumlagerung auf die Tunneldecke; die Fugen zwischen Decke und Unterfangungsfundament wurden verpreßt und die hydraulischen Pressen wieder ausgebaut. Der Bauzustand nach dem Entfernen der Wurzelpfähle ist in Abb. 5 dargestellt.

Die während der Herstellung der Wurzelpfähle und der anfänglichen Unterfangungsarbeiten aufgetretenen Setzungen betrugen maximal 5 mm und wurden durch die Hebungen während des großflächigen, tiefen Erdaushubes (10 - 12 mm) mehr als kompensiert. Dabei bestätigte sich neuerlich, daß die Sohlhebungen einer tiefen Baugrube im tertiären Schluff (sandig-tonig) mittels des Entlastungsmoduls aus dem Kompressionsversuch an ungestörten Proben größenordnungsmäßig gut abgeschätzt werden können. Die Bauausführung spielt natürlich eine wesentliche Rolle: während sich zeitliche Bauabläufe mit verfeinerten FE-Verfahren wenigstens einigermaßen simulieren lassen, ist dies für die Parameter Sorgfalt und Witterung nicht der Fall. Verformungsprognosen sind daher stets mit einer gewissen Unsicherheit behaftet.

Der Einsatz von Schlitzwänden wird dort problematisch, wo nur eine kleine lichte Arbeitshöhe gegeben ist. Trotz Kleingerät für den Schlitzausbau kann dies zu aufwendigen Erschwernissen führen (Abb. 6) oder überhaupt andere Lösungen erfordern.

Abb. 4:
Unterfangung des Wiener
Messepalastes für den
U-Bahnbau; Ausschnitt.
Früher Bauzustand mit
temporären Wurzelpfählen
und ersten Schlitzwänden

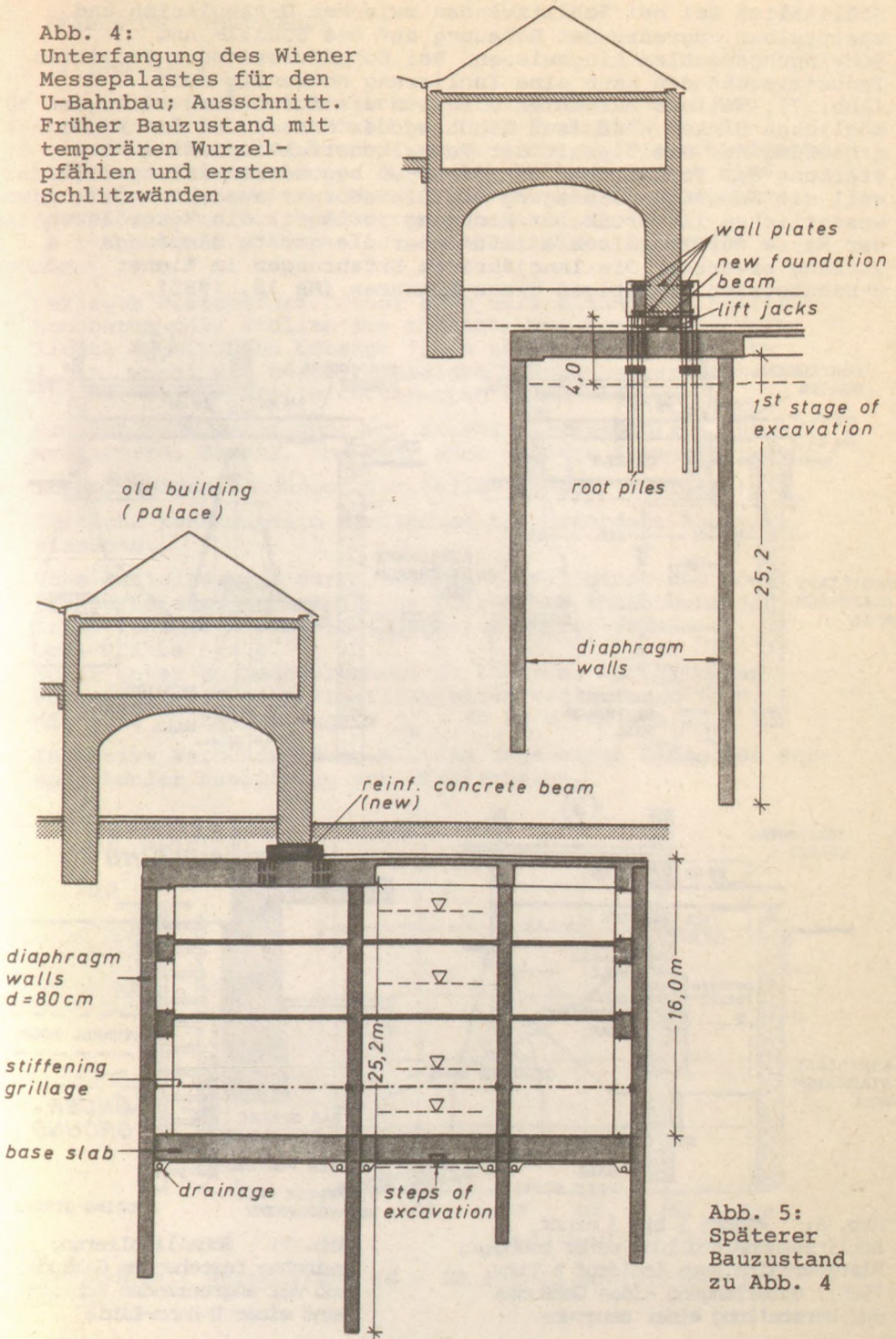


Abb. 5:
Späterer
Bauzustand
zu Abb. 4

Schließlich sei bei Schlitzwänden zwischen U-Bahnlinien und unmittelbar angrenzender Bebauung auf das Schall- und Schwingungsproblem hingewiesen. Bei Wohnhäusern oder sensiblen Industriegebäuden kann eine Isolierung notwendig sein (Abb. 7). Weitere Maßnahmen umfassen die Ausbildung einer möglichst dicken Wand (zur Erhöhung der Masse) und die Vergrößerung der Steifigkeit der Tunnelkonstruktion. Eine Verstärkung der Tunnelsohle ist deswegen besonders wirkungsvoll, weil die Schwingungsanregung vom Gleiskörper ausgeht und im wesentlichen in vertikaler Richtung verläuft; die Vergrößerung der Masse der Tunnelsohle läßt daher die größte Dämpfungswirkung erwarten. Die langjährigen Erfahrungen im Wiener U-Bahnbetrieb bestätigen diese Annahmen (MA 38, 1985).

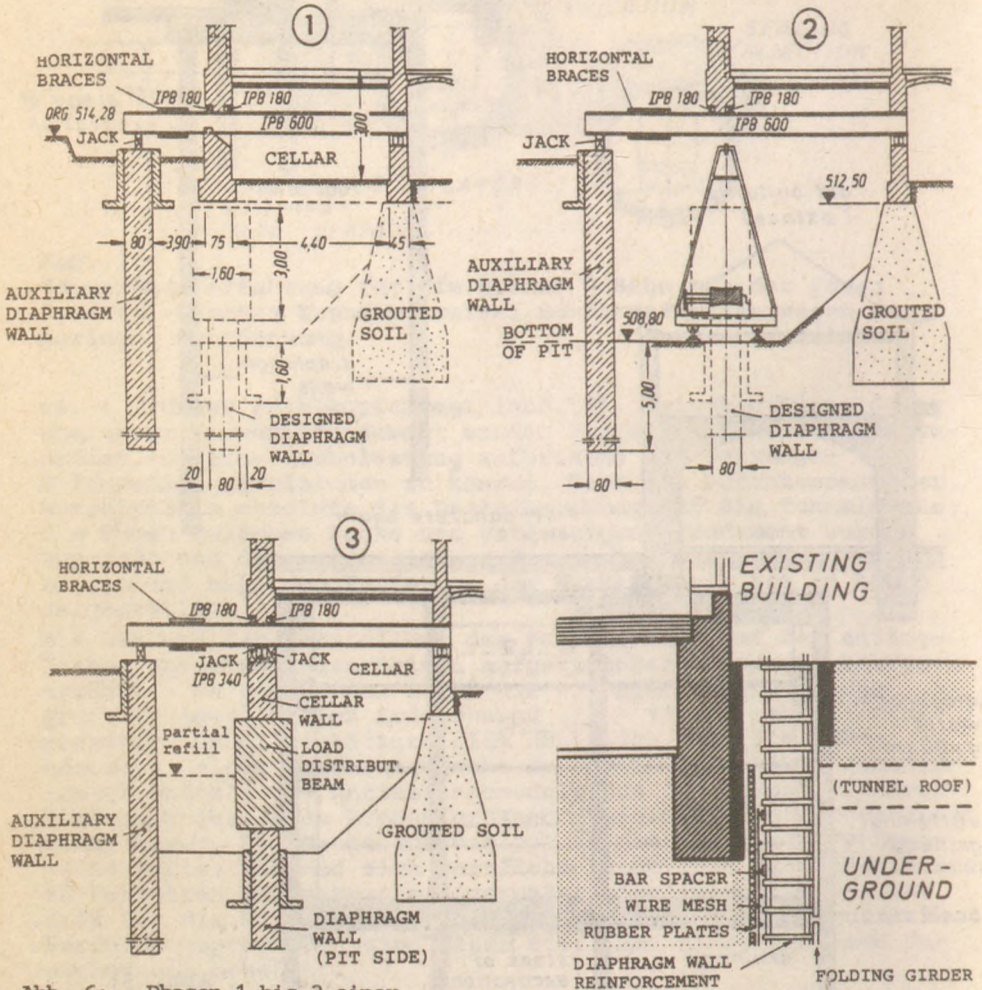


Abb. 6: Phasen 1 bis 3 einer Schlitzwanderherstellung unter beengten Platzverhältnissen (Baldauf & Timm, 1988): Unterfangung eines Gebäudes zur Herstellung einer Baugrube

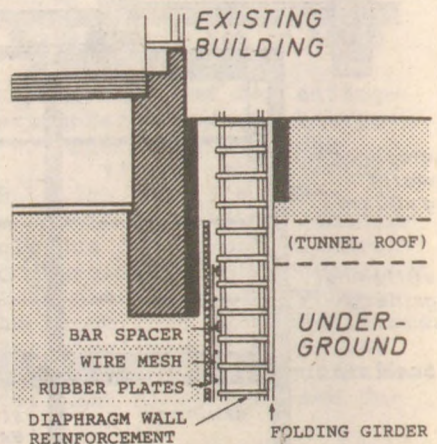


Abb. 7: Schallisolierung zwischen bestehendem Gebäude und der angrenzenden Schlitzwand einer U-Bahn-Linie

2. PFÄHLE UND PFAHLWÄNDE

2.1 Allgemeines

Für Baugrubenwände und Bauwerksunterfangungen kommen von Minipfählen ($d = 10 - 30$ cm) bis zu Großbohrpfählen ($d = 1,2 - 1,8$ m) sämtliche Durchmesserbereiche und verschiedenste Pfahlssysteme zur Anwendung. In vielen Fällen erfolgt eine Kombination unterschiedlicher Verfahren, wie die folgenden Beispiele zeigen. Von besonderem Interesse sind die *Kleinbohrpfähle*, und zwar aus folgenden Gründen:

- Geringer Platzbedarf, daher auch Herstellung von Kellern, Schächten oder Stollen aus möglich. Die derzeit minimale lichte Arbeitshöhe beträgt 1,8 m und die Mindestbreite 1,5 m, wobei mit neuesten Kleinbohrrauben sogar Öffnungen von nur 0,75 m Breite durchfahren werden können.
- Problemlose Überwindung von Bohrhindernissen (z.B. Mauerwerksreste, Beton), und zwar ohne größere Lärmbelästigung.
- In beliebigen Neigungen herstellbar (sogar steigend).
- Einfache konstruktive Einbindung in vorhandene Bauwerkselemente.
- Hohe Mantelreibung durch Druckluftverdichtung des Kontraktorbetons (Wurzelpfähle) oder durch entsprechenden Injektionsdruck auf die Suspension (Micro-Pfähle, Gewi-Pfähle usw.). Sogar unter Zugbeanspruchung in lockeren Auffüllungen wurden relativ hohe Tragfähigkeiten registriert (z.B. mittlere Mantelreibung um $p_s = 70$ kN/m² in Abb.8).
- Intensive Verbundwirkung mit dem umgebenden Boden bei entsprechender Ausbildung von Pfahlgruppen.

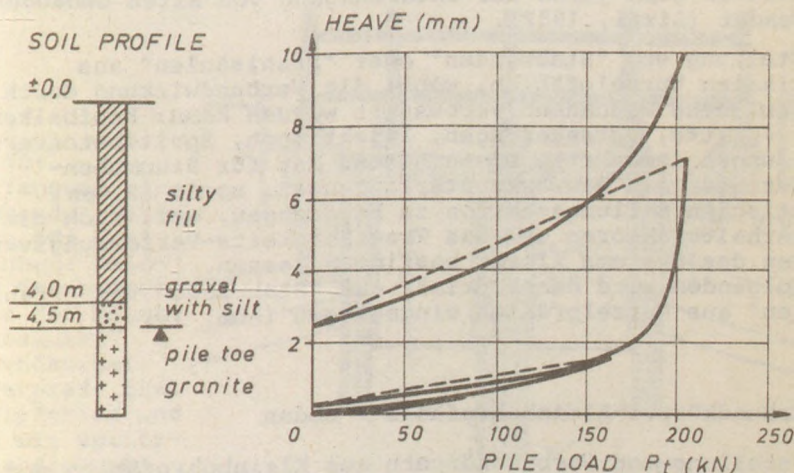


Abb. 8:
Zugversuch mit Wurzelpfahl ($d = 20$ cm) in schluffiger Anschüttung

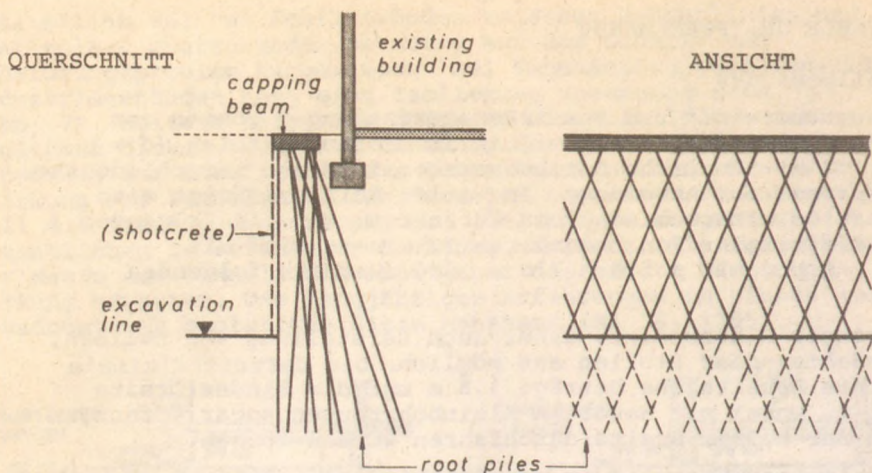


Abb. 9:
Baugrubenwand aus einem Geflecht von Wurzelpfählen (schematisch)

Von erdstatisch-bodenmechanischer Bedeutung sind vor allem die beiden letzten Eigenschaften, weil dadurch ein besonders günstiges Tragfähigkeits-Verformungsverhalten des Pfahl-Boden-Systems erzielt werden kann. Während bei konventioneller Anordnung von Vertikalpfählen der Gruppeneffekt im allgemeinen zu einer Reduktion der Tragfähigkeit des Einzelpfahles führt, bewirkt der Verbundeffekt eine Erhöhung. Dieser kann auf zweierlei Arten erzielt werden:

- Herstellung eines Pfahlgeflechtes aus sich kreuzenden Schrägpfählen (Abb. 9). Diese Lösung ist die ursprüngliche Form der Kleinbohrpfahl-Boden-Verbundkörper und wird z.B. in Italien auch gerne zur Unterfangung von alten Gebäuden verwendet (Lizzi, 1982).
- Herstellung von "Stabwänden" oder "Pfahlsäulen" aus vertikalen Wurzelpfählen, wobei die Verbundwirkung durch verschiedene Maßnahmen verbessert werden kann: Kopfbalken oder -platte, Verankerungen, Injektionen, Spritzbetonverkleidungen (bewehrt). Diese Lösung ist für Baugrubensicherungen mit Bauwerksunterfangungen, somit in den städtischen Ballungszentren zu bevorzugen, weil sich die Sicherheitsfaktoren und das Tragfähigkeits-Verformungsverhalten des Systems klarer bestimmen lassen. Im folgenden wird daher primär auf "Stabwände" und "Pfahlsäulen" aus Wurzelpfählen eingegangen (Abb. 10).

2.2 Verbundkörper Kleinbohrpfähle - Boden

Zur Herstellung von Verbundkörpern aus Kleinbohrpfählen und Boden eignen sich vorwiegend Wurzelpfähle und Injektionsbohrpfähle. Erstere werden häufiger verwendet, und zwar im allgemeinen mit Durchmessern von 20 bis 25 cm, was den einwand-

freien Einbau einer Längs- und Spiralbewehrung ermöglicht. Durchmesser unter 15 cm stellen Ausnahmefälle dar. Die Pfahlherstellung erfolgt im Drehbohrverfahren, in der Regel mit Luft-, Wasser- oder Druckspülung.

Die Verbundkörper sind im allgemeinen Fall sowohl axial auf Druck oder Zug beansprucht als auch durch Seitendrucke (Erddrücke) auf Biegung. Ihre Tragfähigkeit hängt davon ab, in welchem Umfang der Schubverbund zwischen Pfählen und anstehendem Boden wirksam ist. Die Stabwand wird zunächst näherungsweise als monolithischer Körper aufgefaßt, auf den ähnlich wie bei einer geschlossenen Pfahl- oder Schlitzwand die äußeren Kräfte anzusetzen sind. Damit kann die erdstatisch erforderliche Einbindetiefe in den Boden ermittelt werden. Aufgrund der geringen Verformbarkeit dieser Verbundsysteme ist in der Regel der Erdruchedruck in Rechnung zu stellen - vor allem bei starker Setzunsempfindlichkeit der zu unterfangenden Gebäude. Je nach Kopfeinspannung der Wand, Verankerung etc. sind entsprechende Erddruckumlagerungen zu berücksichtigen. Liegt die Einbindetiefe der Stabwand und die zugehörige Lagerungsart im Untergrund fest, können die Schnittgrößen der statisch unbestimmten Konstruktion elementar ermittelt werden. Verankerungen stellen hierbei die Auflager eines Durchlaufträgers dar; ihre Anzahl und Lage ist derart zu variieren, daß sich eine ausgeglichene Momenten- bzw. Querkraftverteilung ergibt.

Anzahl und erforderliche Gebrauchslast der Anker (meist vorgespannte Verpreßanker) sind durch den Querkraftverlauf gegeben. Anzahl und Bewehrung der Wurzelpfähle hängen bei vorgegebenem Pfahldurchmesser neben allfälligen Achsiallasten primär vom aufzunehmenden Biegemoment ab. Dieses wiederum ist eine Funktion der Verbundwirkung zwischen Pfählen und Boden, was bei der Bemessung von Stabwänden (und Pfahlsäulen) berücksichtigt werden sollte:

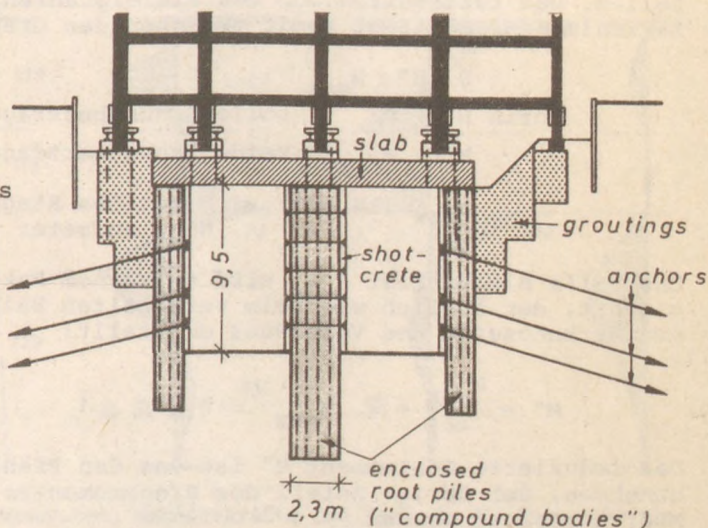


Abb. 10:
Unterfangung eines bestehenden Bauwerkes für den U-Bahnbau: beidseits "Stabwände" und in der Mitte "Pfahlsäulen" (Verbundkörper aus Verpreß bzw. Wurzelpfählen und Boden mit Spritzbetonabdeckung)

Kein Verbund:

Dabei handelt es sich um die konventionelle und vorsichtigste Form der Bemessung. Die Schubübertragung zwischen Pfählen und Boden wird vernachlässigt, und jeder Pfahl erhält die gleiche Belastung. Dieser Ansatz liefert die theoretisch größte Momentenbeanspruchung.

Voller Verbund:

Hier vorausgesetzt, daß Pfähle und Boden einen monolithischen Balken mit gemeinsamem Widerstandsmoment bilden. Die Stabwand (oder die Pfahlsäule) stellt somit einen "bewehrten Bodenkörper" dar, dessen Tragfähigkeit ähnlich wie beim Stahlbetonbalken mit Druck- und Zugbewehrung abgeschätzt werden kann. Der Boden wird dabei als Beton aufgefaßt, und zwar unter Ausschluß der Zugspannungen, die Pfähle wirken als Bewehrung. Voraussetzung für ein gemeinsames Trägheitsmoment der Einzelelemente ist eine ausreichende Haftung zwischen Pfählen und umgebendem Boden. Unter der Annahme der Bernoulli-Hypothese (Ebenbleiben der Querschnitte) kann das auf den Stabwandquerschnitt wirkende äußere Moment M in jeweils gleich große Druck- und Zugkräfte zerlegt werden^{max}, welche die Pfahlreihen nur mehr achsial beanspruchen (z.B. Abb.12). Damit ergibt sich ein unterer Grenzwert für die Belastung des Einzelpfahles.

Teilweiser Verbund:

In der Praxis liegen die Verhältnisse zwischen den beiden vorstehenden Grenzwerten, wobei je nach Pfahlausteilung und Schubübertragung sehr unterschiedliche Spannungs- und Dehnungsverteilungen über dem Stabwandquerschnitt auftreten können (Brandl, 1971, 1988). Es empfiehlt sich daher, bei der Bemessung von Stabwänden nur einen Teil der äußeren Biegemomente in jeweils gleichgroße Druck- und Zugkräfte aufzuteilen. Das tatsächlich von den Einzelpfählen aufzunehmende Maximalmoment M'' liegt somit zwischen den Grenzwerten

$$\begin{aligned} 0 < M'' < M_{\max} \\ \text{worin } M'' = 0: & \quad \text{volle Schubübertragung} \\ M'' = M_{\max}: & \quad \text{keine Schubübertragung} \\ M_{\max} \text{ (kN m/m)} & \dots \text{maximales Biegemoment je} \\ & \quad \text{Wandlaufmeter} \end{aligned}$$

Das volle Biegemoment $M_{\max}$ wird mit einem Faktor α abgemindert, der ähnlich wie beim verdübelten Balken ein Maß für den Wirkungsgrad des Verbundes darstellt:

$$M'' = \frac{M_{\max}}{\alpha} = \alpha \cdot M_{\max} \quad 0 \leq \alpha \leq 1$$

Das reduzierte Teilmoment M'' ist von den Pfählen direkt aufzunehmen, der übrige Anteil des Biegemomentes kann in Axialkräfte zerlegt werden:

$$M_{\max} - M'' = M_{\max} \cdot (1 - \bar{\alpha}) \quad C = -T = \frac{M_{\max}}{e} \cdot (1 - \bar{\alpha})$$

e Hebelarm der inneren Kräfte

C Druckkraft (Compression)

T Zugkraft (Tension)

Der Tragfähigkeitsnachweis für den Einzelpfahl ist somit wie folgt zu führen:

$$1) \quad P_{ei}'' = \frac{V}{n} + \frac{C_i}{m_i} \leq P_{zul.} \quad (\text{Druck})$$

$$= \frac{V}{n} - \frac{T_i}{m_i} \leq P_{zul.} \quad (\text{Zug})$$

wobei sich C_i , T_i ergeben aus:

$$(1 - \bar{\alpha}) \cdot M_{\max} = \sum_{i=1}^k C_i \cdot e_i = - \sum_{i=1}^k T_i \cdot e_i$$

k Anzahl der Pfahlreihen in Stabwandlängsrichtung

n Pfahlanzahl je Laufmeter

m_i Pfahlanzahl je Laufmeter für die Pfahlreihe i

und $\frac{C_i}{e_i/2} = \frac{|T_i|}{e_i/2} = \text{const.} \quad (\text{näherungsweise lineare Spannungs-}$
verteilung)

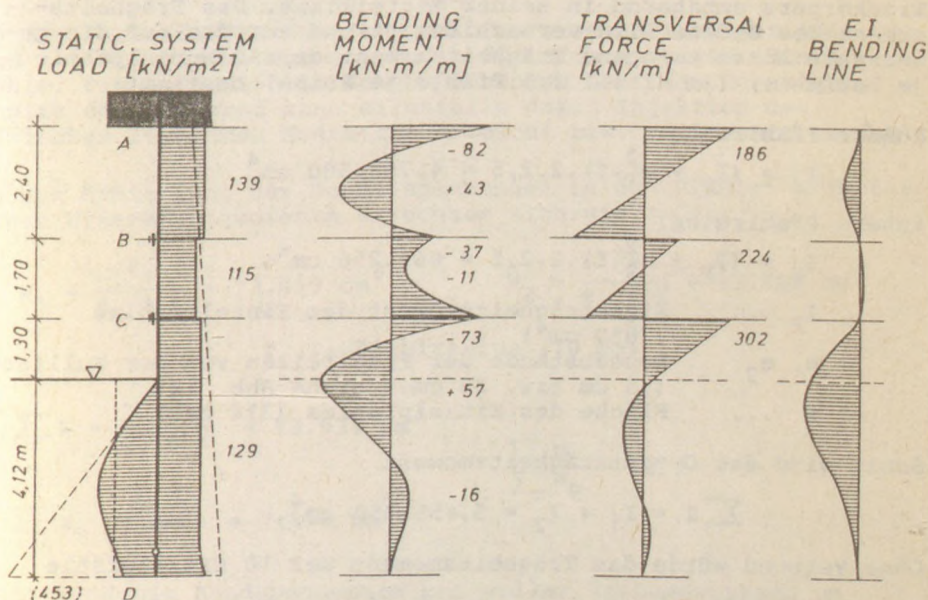


Abb. 11: Belastungssystem, Schnittkräfte und EI-fache Verformungen für eine Stabwand, welche als Unterfangung und Baugrubenwand wirkt. Beispiel aus der Praxis mit Erddruckumlagerungen.

$$2) \quad M'' = \frac{\bar{\alpha} \cdot M_{\max}}{n} \leq M_{\text{zul.}}$$

Der Fall $\bar{\alpha} = 0$ wird praktisch nicht eintreten, da sogar beim Stahlbetonbalken infolge Sekundärbiegung eine Spannungserhöhung bis zu 3 % in der Bewehrung auftreten kann. Daher empfiehlt es sich, auch bei "vollem" Verbund $\bar{\alpha} = 0,10$ anzunehmen und das auf den Einzelpfahl entfallende effektive Biegemoment durch Stahleinlagen aufzunehmen.

Rechenbeispiel:

In Abb. 11 sind das Belastungssystem und die Schnittkräfte sowie EI-fachen Verformungen für eine zweifach verankerte, beidseitig eingespannte Stabwand angegeben. Der zugehörige Grundriß geht aus Abb. 12 hervor. Das größte Biegemoment tritt demnach an der Kopfeinspannung mit $M_{\max} = -82 \text{ kNm/m}$ auf, die Vertikalbelastung beträgt 1000 kN/m .

Ohne Berücksichtigung einer Verbundwirkung ergibt sich für die Pfähle

$$P_e = \frac{V}{n} = 100 \text{ kN} < P_{\text{zul}} \quad M_{Pe} = \frac{M_{\max}}{n} = 8,2 \text{ kNm}$$

P_e (kN) ... Achsialkraft je Einzelpfahl

M_{Pe} (kNm) ... maximales Biegemoment je Einzelpfahl

n Pfahlanzahl je Laufmeter ($n = 10$ gemäß Abb. 12)

Bei vollem Schubverbund liegt die Nullfaser des Pfahl-Boden-Tragkörpers annähernd in seiner Mittellinie. Das Trägheitsmoment des Bodens wird vernachlässigt und nur das auf die gemeinsame Achse bezogene Trägheitsmoment der 10 Wurzelpfähle je Laufmeter (im Mittel 2,5 Pfähle je Reihe) bestimmt:

äußere Pfahlreihe:

$$I_1 = (I_p + e_1^2 \cdot f) \cdot 2 \cdot 2,5 = 4,788.500 \text{ cm}^4$$

innere Pfahlreihe:

$$I_2 = (I_p + e_2^2 \cdot f) \cdot 2 \cdot 2,5 = 667.250 \text{ cm}^4$$

I_p ... Eigenträgheitsmoment des Einzelpfahles (7850 cm⁴)

e_1, e_2 ... Achsabstände der Pfahlreihen von der Nullfaser (55 cm bzw. 20 cm - siehe Abb. 12)

f ... Fläche des Einzelpfahles (314 cm²)

Somit wird das Gesamtträgheitsmoment

$$\sum I = I_1 + I_2 = 5,455.750 \text{ cm}^4.$$

Ohne Verbund würde das Trägheitsmoment der 10 Einzelpfähle

$$\sum I_p = 10 \cdot I_p = 78.500 \text{ cm}^4$$

betragen.

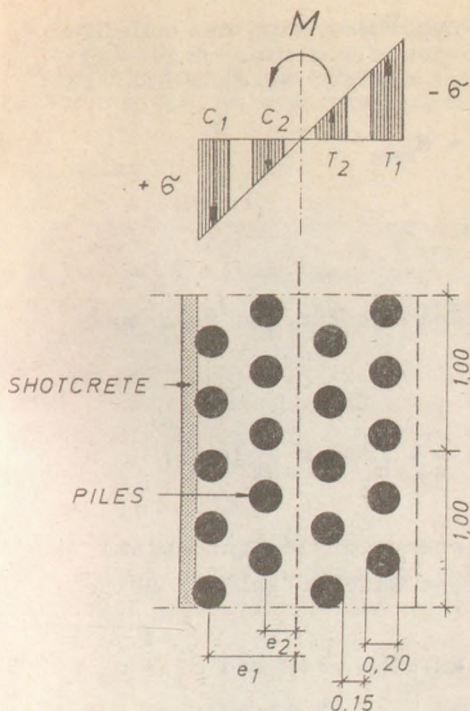


Abb. 12:
Grundriß zur Abb. 11 mit
Zerlegung des Momentes M in
Druck- und Zugkräfte (C, T),
welche die Pfähle nur
achsial beanspruchen;
Idealisierung eines vollen
Schubverbundes über dem
gesamten Querschnitt
(Analogien zum Stahlbeton
mit Druck- und Zugberechnung);
Ebenbleiben der
Querschnitte

Das Verhältnis von $\frac{\sum I}{\sum I_P} = 69,5$ bedeutet für die Praxis,

daß die Verformungen des Verbundkörpers wesentlich geringer sind als die Durchbiegung jeweils für sich wirkender Einzelpfähle. Die Aufnahme von Schubspannungen über den Gesamtquerschnitt der Stabwand kann allenfalls durch Injektion des dazwischen liegenden Erdreichs erreicht bzw. verbessert werden.

Die zur Ermittlung der Normalspannungen in den Pfählen erforderlichen Widerstandsmomente errechnen sich wie folgt:

$$W_1 = \frac{I_1}{e_1 + r} = 73.699 \text{ cm}^3$$

$$W_2 = \frac{I_2}{e_2 + r} = 22.242 \text{ cm}^3$$

r ... Pfahlradius (10 cm)

$$\sum W = \frac{\sum I}{e_1 + r} = 83.934 \text{ cm}^3$$

$$\frac{\sum W}{\sum W_P} = 10,7$$

$$\sum W_P = \frac{\sum I_P}{r} = 7.850 \text{ cm}^3$$

Somit sind die Randspannungen bei voller Verbundwirkung um etwa eine Zehnerpotenz geringer als in den einzeln wirkenden Wurzelpfählen (ohne Verbund).

Unter der Annahme der Bernoulli-Hypothese kann das auf die Stabwand wirkende Biegemoment in jeweils gleich große Zug- (T_1, T_2) bzw. Druckkräfte (C_1, C_2) zerlegt werden (Abb. 12):

$$C_1 \cdot 2e_1 + C_2 \cdot 2e_2 = M_{\max}$$

$$\frac{C_1}{C_2} = \frac{e_1}{e_2}$$

Damit ergibt sich als unterer Grenzwert für die Belastung eines Pfahles:

$$M_{Pe}' = 0$$

$$P_{e_i}' = \frac{V}{n} + \frac{C_i}{m_i} \leq P_{zul} \text{ (Druck)} = \frac{V}{n} - \frac{T_i}{m_i} \leq P_{zul} \text{ (Zug)}$$

M_{Pe}' (kNm/m) ... maximales Biegemoment je Einzelpfahl

P_{e_i}' (kNm/m) ... Achsialkraft je Einzelpfahl der Reihe i

Mit $M_{\max} = -82$ kNm/m, $V = 1000$ kN/m, $n = 10$ und $m_1 = m_2 = 2,5$ wird

$$C_1 = -T_1 = 66 \text{ kN/m} \quad \text{und} \quad C_2 = -T_2 = 24 \text{ kN/m}$$

$$P_{e_1}' = \frac{1000}{10} \pm \frac{66}{2,5} = 126 \text{ (74) kN}$$

$$P_{e_2}' = \frac{1000}{10} \pm \frac{24}{2,5} = 109 \text{ (90) kN}$$

Diese Achsialkräfte sind minimal, weshalb auch die Alternative einer Stabwand ohne Verankerung untersucht wird. Dafür errechnen sich ein maximales Biegemoment von $M_{\max} = -782$ kNm/m an der Einspannstelle des Wandkopfes und achsiale Pfahlkräfte von

$$C_1 = -T_1 = 628 \text{ kN/m} \quad \text{und} \quad C_2 = -T_2 = 228 \text{ kN/m.}$$

Mit $V = 1000$ kN/m ergibt sich schließlich:

$$P_{e_1}' = \frac{1000}{10} \pm \frac{628}{2,5} = 351 \text{ (-151) kN}$$

$$P_{e_2}' = \frac{1000}{10} \pm \frac{228}{2,5} = 191 \text{ (-9) kN}$$

Die größte Achsialbeanspruchung der Pfähle, nämlich 351 kN Druck bzw. -151 kN Zug, tritt an der Einspannstelle auf. Im dortigen Pfahlkopfbereich ist eine volle Schubübertragung zweifellos gesichert, zudem besitzen die Pfähle dort infolge der Druckluftverdichtung stets größere Durchmesser als in Rechnung gestellt. Insgesamt ist auch die Beanspruchung rela-

tiv gering, somit ist eine Reduktion der Pfahlanzahl zulässig. Zur Minimierung der Verformungen und zur Gewährleistung eines ausreichenden Schubverbundes der Stabwand wurden schließlich 6 Pfähle je lfm und eine Ankerlage vorgesehen.

Resümee:

Die Tragfähigkeit von Verbundkörpern aus Pfählen und Boden (Stabwände, Pfahlsäulen) kann mit dem vorliegenden semi-empirischen Bemessungsverfahren hinreichend genau abgeschätzt werden. Modellversuche und Baustellenmessungen bestätigten die Theorie, welche zwangsläufig von Idealisierungen ausgeht. In der Praxis liegt der Faktor α meist zwischen 0,3 bis 0,8; bei großem Pfahlabstand (ca. $a \geq 3$ bis 5 d) und geringem Verbund wird $\alpha \approx 1,0$ anzusetzen sein. Insgesamt führt die Bemessung nach der Verbundtheorie zu wesentlichen Kosteneinsparungen gegenüber den konventionellen Berechnungen.

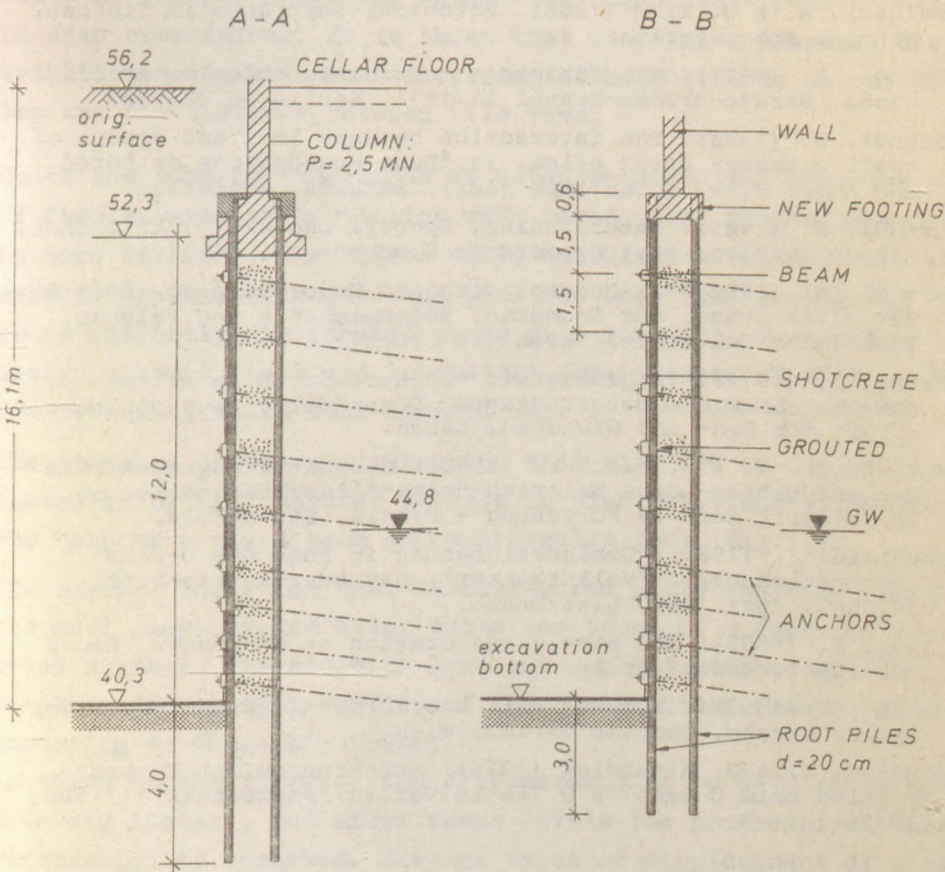


Abb. 13: Gebäudeunterfangung für eine U-Bahn-Baugrube

2.3 Ausführungsbeispiel

Im Zuge eines städtischen Verkehrstunnelbaues mußten die Einzelfundamente eines an die 16 m tiefe U-Bahn-Baugrube grenzenden Gebäudes unterfangen werden (Abb. 13). Die Fundamentblöcke wurden vom Freien und vom Keller aus vertikal durchbohrt und ein neues lastverteilendes Fundament unter der Kellerwand eingezoogen, welches die Bauwerkslasten auf zwei Reihen von Wurzelpfählen überträgt. Im Zuge des schrittweisen Aushubes für die tiefe Baugrube wurde die Stabwand laufend verankert und mit Spritzbeton abgedeckt. Das dichte "Vernageln" und Herstellen von Injizierbereichen um jeden Anker innerhalb der Wand erhöhte die Schubübertragung so weit, daß trotz des breiten mittleren Bodenkernes eine Verbundwirkung zustande kam.

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EXPERIMENTAL STUDIES OF BORED PILES RETAINING WALLS IN DEEP-EXCAVATIONS

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SYNOPSIS

The paper gives the results and analysis of field and laboratory tests of two-row retaining walls with bored cast-in-situ piles in deep excavations. It is shown that the active pressure distribution among pile rows in design schemes depends on the pile cap and the distance between pile rows.

Under the compact conditions of urban construction area the method of deep excavation protection with bored piles retaining walls is more rational. In multirow walls calculation the active pressure is applied either to the pile row last from excavation (5) or is uniformly distributed among pile rows (2). To control the design schemes field and laboratory experimental studies of two-row retaining walls were carried out.

The pressure on two-row retaining wall with 800 mm dia piles installed by a triangular lattice in plan was transferred with the help of a rigid beam and a hydraulic jack (fig.1).

The bearing layer for cast-in-situ bored piles was hard and plastic marl clay. Ground water table was found at a depth of 3,6 m below surface. The stepwise load was applied at the level which corresponded to the position of the resultant of active pressure according to Coulomb's theory.

As shown in the fig.2,a, the displacement up to 6-10 mm is approximately linear, but after these levels the progressive plastic deformation is observed. Average value of displacement of piles of the second row is 2-3 times higher than that of the first row. These differences increase with the load growth.

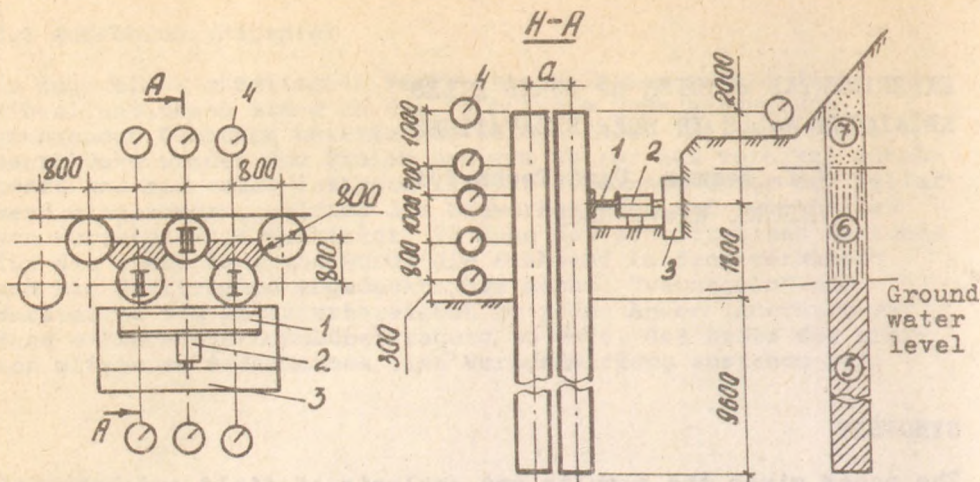


Fig. 1 Retaining Wall No 1 Test

I, II, III - test piles; 1-rigid beam; 2-hydraulic jack;
3-support slab; 4-deflectometers; 5-clay marl; 6-loam;
7-filled-up soil

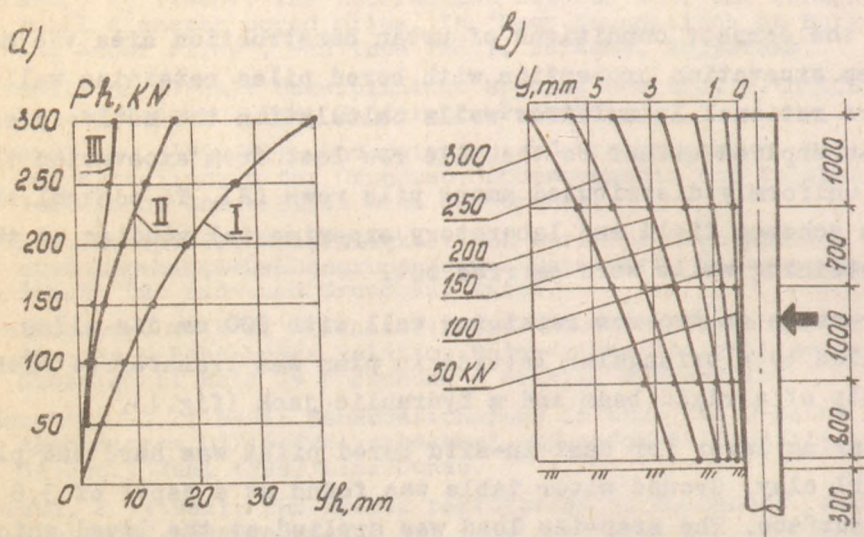


Fig. 2 Pile Head Displacement of the 1st and the 2nd rows (a) and Pile Shaft III above the Level of Excavation (b)

It is obviously that if the distance between pile rows "a" is small and equal to its diameter "d", the soil wedge (shaded in the fig. 1) is too small to transference the load effectively from the second pile row to the first one. Therefore the soil is pressed through the space between piles of the first row and

active pressure is distributed nonuniformly. In this case the first pile row receives 20-30% of active pressure, and the second one - 70-80%.

The lines of displacement of the pile shaft III within the limits of its free length (fig.2,b) confirms that under the loads of 0+100 kN a pile turns as a rigid bar and subsequent load increasing leads to the rotation of a pile and its bending. If prolonge the line of bending below the excavation level we can receive approximate position of zero point or the point of zero displacement. This point displaced upwards with load increasing, changed its position from 0,22 to 0,4 of the length of pile parts in soil that corresponded to the interval had been determined in the work (6) for the piles of final stiffness.

The comparison of experimental values of pile displacement with calculated ones according to elasticity theory if an average load on a pile is 300 kN is given in the table. It shows that the measured displacement is 1,5+3 times less than the calculated one.

T A B L E

Measurement point at the level of	Pile displacement, mm					
	measu- red	calcu- lated	measu- red	calcu- lated	measu- red	calcu- lated
	I		II		III	
excavation	6,00	30,8	13,7	30,8	2,40	7,7
applying of						
horizontal force	9,87	48,5	22,5	48,5	3,95	12,1
pile head	18,0	68,2	41,0	68,1	7,20	17,0

As shown in (1), to approximate calculated value of horizontal pile displacement it is necessary to determine the coefficient of deformation considering the piles manufacturing technology. It was proposed to calculate current value of proportional ratio considering its changes in the process of load application.

A fragment of the retaining wall No 2 consisted of two cast-in-situ bored piles of 0,8 m dia with rigid pile cap (fig.3). The piles cut through the layers of clay, sandy clay and sand. The ground water level was below the pile tip. Reinforcing was uniformly distributed along the pile perimeters as well as in the wall No 1.

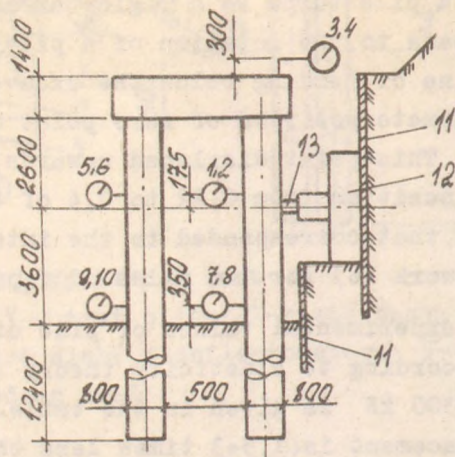


Fig.3 Tested Fragment of Retaining Wall No 2
 1+10-numbers of deflectometers; 11-sheet pile;
 12-support slab; 13-hydraulic jack

The loads from the second pile row to the first one are redistributed effectively by a rigid pile cap : pile displacement of two rows is equal to each other at the level of pile heads and it differs only 1,5 times at the level of load application and practically the same at the level of excavation. Under the equal loads (from 200 to 600 kN) the portions of more loaded piles are already in the elastoplastic stage, while the others are still in the elastic stage. The frame structure of the retaining wall is statically indeterminable and redistribution of forces as well as known for all similar systems takes place (3,4). It is possible that the excess of experimental bearing capacity of the tested frame (equal to 630 kN) versus designed one (equal to 300 kN) is connected with redistribution of forces. (In above-mentioned comparison experimental bearing capacity is taken under the deformation of piles

at the excavation level of 10 mm). This difference can be reduced if the design is carried out considering redistribution of forces. Frame structure of the retaining wall has also shown high crack resistance as the first visible crack with width of 0,1-0,2 mm emerged in the most loaded section, in the place of load application under the force of 650 kN only. Owing to frame structure of the wall and rigid pile cap, the pile being restrained in its head does not turn around zero point but bends in the place of load application. Because of a rigid pile cap the displacement of pile head exceeds insignificantly its displacement in the place of load application (within the limits of 20-25%). At the same time in analogous tests of the wall without pile cap (the wall No 1) the difference of displacement makes up 60-80% (see fig.2). This fact shows that application of frame structures of retaining walls with cast-in-situ bored piles is especially effective if it is necessary to limit displacement at the top level of a retaining wall.

Model studies of a fragment of two-row retaining wall without pile cap of 1/20 of its full scale were carried out in humid sand. In the experiments the distance "a" between the axes of the piles in a row, the distance "b" between the axes of pile rows and the pressure on the wall were changed with the growth of excavation depth in front of the wall and with the increase of loading behind the wall.

To define the components of active pressure acting on the pile row first from excavation the second one in some experiments was fixed. The pressure of the second pile row on the first one (secondary pressure E_a^s) was eliminated and this row was displaced only due to the direct action of the first active pressure E_a^f . Providing free position of the second pile row the first one receives the total pressure $E_a^f + E_a^s$.

It is determined that with constant value of "a = 2d" for both pile rows and minimum distance "b = d", where "d" is pile diameter the total pressure acting on the first row accounts for 85% of that acting on the second one. In this case as well as in the experiments with the wall No 1 the dimensions of soil body between pile rows are insufficient for receiving the whole

secondary pressure E_a^s . If $b = 2d+3d$, redistribution of active pressure among pile rows is practically uniform.

CONCLUSIONS

1. Distribution of active pressure among pile rows in the design schemes of the retaining walls with pile cap or if $b = 2d+3d$ can be accepted uniform but it should be transferred about 80% of the total pressure on the last from excavation pile row without pile cap or if the distance between pile rows is small.
2. The design of the retaining walls based on elasticity theory leads to overestimation of distributive properties of soil base-ment and pile displacement at the second phase of the soil stressed-strained state.
3. The multirow retaining walls with pile cap should be designed considering the pile manufacturing technology, the proportional ratio changed in the process of load application, and considering redistribution of forces.

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ÜBERSCHNITTENE BOHRPFAHLWÄNDE MIT MEHREREN ZWISCHENPFÄHLEN

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SUMMARY

For excavations below groundwater level bored pile walls with two or three not reinforced intermediate piles in a row have recently been used in FRG for economical reasons. However, the theoretical models given in this paper show that these variations of the classical overlapping bored pile version lead to kinematically non-stable systems, whose friction forces are not sufficient to satisfy the equilibrium conditions. Based on these results and taking into account permissible variations in pile drilling a version with three intermediate piles in form of vaults is recommended.

ZUSAMMENFASSUNG

Überschnittene Bohrpfehlwände bestehen im allgemeinen aus unbewehrten Zwischenpfählen und bewehrten Tragpfählen, die einzeln im Wechsel angeordnet werden. Aus wirtschaftlichen Gründen werden in letzter Zeit in der BRD für Baugruben im Grundwasser auch Lösungen mit zwei oder drei Zwischenpfählen in einer Ebene angeboten.

Anhand einfacher Modellvorstellungen werden Überlegungen zur Kinematik und zum Gleichgewicht solcher Wandausbildungen angestellt. Dabei werden zwischen den einzelnen Pfählen und dem Boden nur diejenigen Kräfte berücksichtigt, die auch unter ungünstigen Bedingungen stets wirksam sind. Aus diesen Betrachtungen und unter Berücksichtigung der unvermeidbaren Herstellungstoleranzen werden anhand eines ausgeführten Beispiels Empfehlungen zur Ausbildung solcher Wandformen als Ketten von Gewölben bestimmter geometrischer Anordnung gegeben.

EINLEITUNG

In der klassischen Bauweise werden bei überschnittenen Bohrpfehlwänden alternierend je ein unbewehrter Zwischenpfahl und ein bewehrter Tragpfahl in einer Ebene angeordnet (Bild 1a). Aus baubetrieblichen und aus wirtschaftlichen Gründen wird auch in Fällen mit zwei oder mit drei Zwischenpfählen eine ebene Anordnung gewünscht. Bedingt durch die Reihenfolge der Herstellung entsteht bei drei Zwischenpfählen eine symmetrische, bei zwei Zwischenpfählen eine asymmetrische Anordnung (Bild 1b,c).

sche Kette der Zwischenpfähle 1 und 2 mit der virtuellen Verschiebungsgröße $\delta\psi$ lautet: $\Sigma F_h 2r \sin \vartheta 2\delta\psi - Tr 2\delta\psi + \Sigma F_h 2r \sin \vartheta \delta\psi - Tr \delta\psi = 0$. Für den Fall, daß die Wasserdruckkraft und die Kräfte in der Rißfläche einander gerade aufheben, ergibt sich für den Grenzgleichgewichtszustand als erforderlicher Reibungswinkel $\tan \delta_R = \frac{2 \sin \vartheta}{\vartheta}$. Für die üblichen Überschneidungsmaße von $\sin \vartheta = \text{rd. } 0,85$ erhält man daraus erforderliche Reibungswinkel in der Größenordnung von über 50° . Kohäsionswirkungen verändern dieses Ergebnis nicht, weil sie die Erddruckspannungen und die Reibungsspannungen in gleichem Verhältnis reduzieren.

Eine Beurteilung des Gleichgewichtszustandes unter Heranziehung aller Kräfte läßt sich bei der Vielzahl der Parameter besser anhand eines Beispiels durchführen (Bild 3c). Als maßgebender Beurteilungsparameter wird die freie Standhöhe h der Zwischenpfähle im Grenzgleichgewichtszustand gewählt, die ab Geländeoberkante zu messen ist, da bei solchen Baugrubenwänden die Steifen bzw. die Anker in der Regel ohne Gurtung nur vor den Tragpfählen angeordnet werden. Sie wird hier in Abhängigkeit von der Rißflächenneigung α beispielhaft für die Abmessungen und Baugrundverhältnisse einer Baugrube beim U-Bahnbau in Hannover angegeben. Für die Parameter $r = 0,44 \text{ m}$, $h_w = 2,5 \text{ m}$, $\vartheta = 0,87$, $p = 10 \text{ kN/m}^2$, $\delta_R = \varphi' = 32,5^\circ$ (vgl. Bild 3) ergibt sich bei horizontalen Rißflächen der Wert $h = 5,2 \text{ m}$ und bei einer Rißflächenneigung von 20° nur noch eine freie Standhöhe von $2,6 \text{ m}$. Für einen Standsicherheitsnachweis im Sinne der technischen Bauvorschriften wäre noch ein angemessener Sicherheitsbeiwert einzuführen. Damit und unter Berücksichtigung größerer Rißflächenneigungen kann ein rechnerischer Standsicherheitsnachweis für baupraktisch relevante Werte von h nicht erbracht werden.

FOLGERUNGEN UND AUSFÜHRUNGSBEISPIEL

Aus den vorstehenden Betrachtungen ergibt sich, daß überschnittene Bohrpfahlwände mit mehreren, in einer Ebene angeordneten Zwischenpfählen mindestens einen kinematischen Freiheitsgrad besitzen und daß die nachweisbaren Reibungskräfte des Systems zur Gleichgewichtsbildung nicht ausreichend sind. Solche Wandformen können deshalb nur in Form von Gewölbeketten mit ausreichendem Stich nachweisbar standsicher ausgeführt werden. Durch die Anordnung der Zwischenpfähle in Bögen zwischen den Tragpfählen entsteht ein unverschiebliches System. Die auf die Zwischenpfähle wirkenden Lasten werden durch die aus mehreren Zwischenpfählen gebildeten Bögen auf die Tragpfähle übertragen.

Im Grundbau sind im allgemeinen etwas größere Ausführungstoleranzen zu berücksichtigen als im Hochbau. Deshalb müssen der Gewölbestich und das Überschneidungsmaß zwischen benachbarten Pfählen ausreichend groß gewählt werden. Firmen mit modernen Geräten zur Großbohrpfahlherstellung gewährleisten im allgemeinen eine Abweichung der Pfahlachse von der Sollage von weniger als $0,5 \%$, also maximal 5 cm pro 10 m Pfahllänge. Im ungünstigsten Fall können derartige Abweichungen bei benachbarten Pfählen in entgegengesetzter Richtung auftreten. Beim U-Bahnbau in Hannover wurde deshalb die Bohrpfahlwand für eine rd. 10 m tiefe Baugrube im Grundwasser, bei der 88 cm dicke Bohrpfähle zum Einsatz kamen, mit einem Gewölbestich von 15 cm und mit einem Überschneidungsmaß von $13,6 \text{ cm}$ ausgeführt (Bild 4). In diesem Fall ist bei Einhaltung der o. a. Toleranzen die erforderliche praktische Wasserdichtigkeit

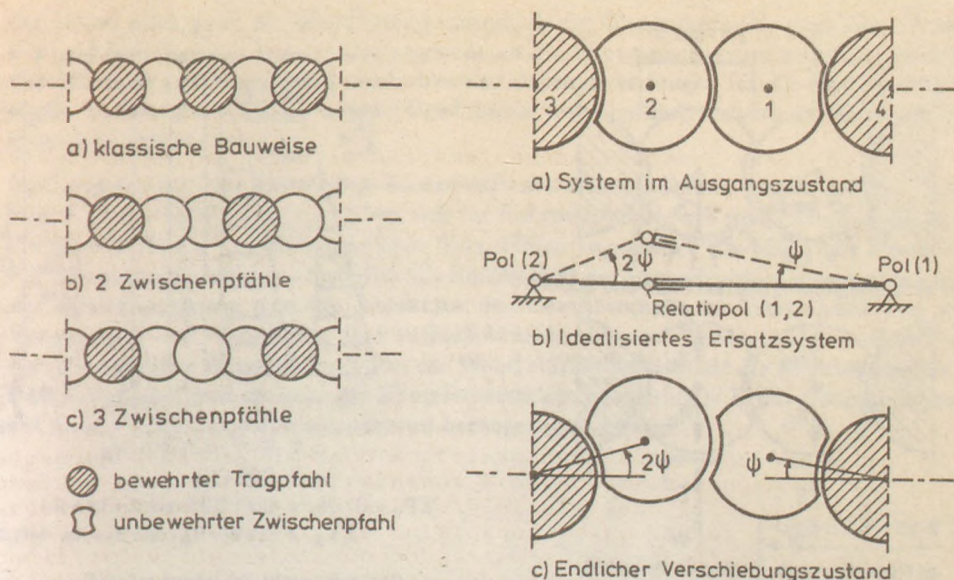
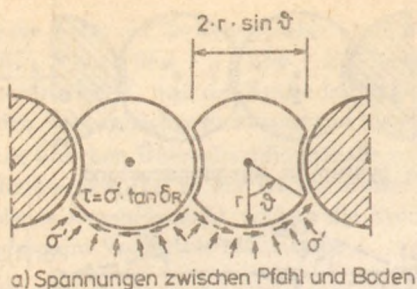


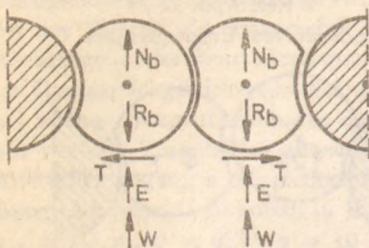
Bild 1 Varianten ebener Bohrpfahlwände Bild 2 Kinematik bei 2 Zwischenpfählen

BETRACHTUNGEN ZUR KINEMATIK

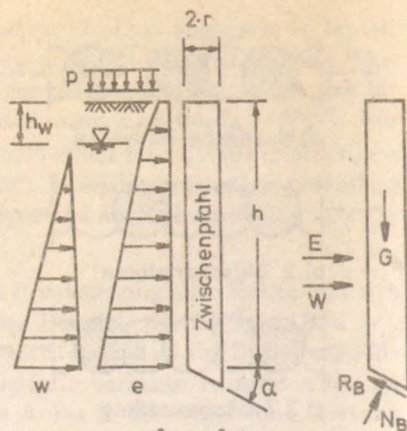
Bei einer Analyse des Tragverhaltens muß aus Sicherheitsgründen von den folgenden Annahmen ausgegangen werden: (a) Die Zwischenpfähle sind in verschiedenen Höhenlagen durchgängig gerissen. Die Risse sind eben und unter dem Winkel α zur Horizontalen geneigt (Bild 3c). (b) In den Fugen zwischen den Pfählen wird bei Belastung senkrecht zur Wandebene keine Reibung wirksam. Für ebene Wandformen mit zwei Zwischenpfählen erhält man damit im Grundriß die in Bild 2a dargestellte Anordnung von Scheiben. Eine mögliche kinematische Kette besteht darin, daß sich die Zwischenpfähle 1 und 2 jeder für sich auf einer Kreisbahn um den jeweils benachbarten unverschieblichen Tragpfahl 3 bzw. 4 bewegen, ohne sich von diesem zu lösen. Die Fuge zwischen den Zwischenpfählen hingegen darf sich öffnen. Für diese Kinematik liegen der momentane Pol der Scheibe 2 im Mittelpunkt des Pfahls 3, der Pol der Scheibe 1 im Mittelpunkt des Pfahls 4 und der Relativpol der Scheiben 1 und 2 im Mittelpunkt der Scheibe 2. Aus dieser Überlegung erhält man das idealisierte, der Anschauung leichter zugängliche Ersatzmodell aus Stäben in Bild 2b, dessen Verschiebungszustand sowohl für infinitesimal kleine als auch für endliche Verschiebungen widerspruchsfrei gezeichnet werden kann. Der zugehörige endliche Verschiebungszustand für das tatsächliche Scheibenmodell ist in Bild 2c dargestellt. Läßt man zusätzlich auch die Öffnung der beiden anderen Fugen zu, sind beliebig viele verschiedene endliche Verschiebungszustände möglich. Für Systeme mit drei Zwischenpfählen sind dementsprechend mindestens zwei symmetrische und beliebig viele asymmetrische Verschiebungsfiguren möglich.



a) Spannungen zwischen Pfahl und Boden



b) Resultierende Kräfte im Grundriß



$$G \approx \pi \cdot r^2 \cdot \sin \vartheta \cdot h \cdot \gamma_B$$

$$R_B \approx 0,85 \cdot N_B$$

$$\Sigma F_V = 0: N_B = G / (0,85 \cdot \sin \alpha + \cos \alpha)$$

$$\Sigma F_H = E + W - R_B \cdot \cos \alpha + N_B \cdot \sin \alpha$$

c) Gleichgewicht im Längsschnitt

Bild 3 Gleichgewichtsbetrachtungen bei Systemen mit 2 Zwischenpfählen

BETRACHTUNGEN ZUM GLEICHGEWICHT

Für die nachfolgenden Gleichgewichtsbetrachtungen wird angenommen, daß die ansetzbaren Reibungskräfte jeweils vollständig mobilisiert sind. Dazu sind endliche Bewegungen der Zwischenpfähle erforderlich, infolge derer sich die Kontaktflächen zwischen den Pfählen öffnen können. Deshalb darf hier keine Reibung angesetzt werden (Annahme b). Auf der Oberfläche der Zwischenpfähle werden durch die wirksamen Erdruckspannungen σ' die Reibungsspannungen $\tau = \sigma' \tan \delta_R$ aktiviert, die als Reaktionswirkungen der jeweiligen Bewegung entgegen gerichtet sind. Die Erdruckspannungen sind hier für den Wandreibungswinkel $\delta = 0$ zu berechnen, da die Reibungsfähigkeit zwischen Wand und Boden für die oben beschriebenen Reibungsspannungen zur Verfügung stehen muß. Unter Verwendung der Bezeichnungen in Bild 3a,b erhält man hierfür die resultierenden Kraftgrößen $E = 2 \int_0^{\vartheta} \sigma' \cos \vartheta r d\vartheta = 2r \sigma' \sin \vartheta$ und $T = 2 \int_0^{\vartheta} \tau r d\vartheta = 2r \vartheta \sigma' \tan \delta_R$ (ϑ im Bogenmaß). Im Längsschnitt in Bild 3c werden außerdem die resultierende Wasserdrukkraft W und die Gewichtskraft G des oberen Zwischenpfahlabschnitts sowie die Normalkraft N_b und die Reibungskraft $R_b \approx 0,85 N_b$ in der um α geneigten ebenen Reißfläche deutlich gemacht. Die sich aus den Kräftegleichgewichtsbedingungen ergebenden Ausdrücke für N_b und für die resultierende Horizontalkraft ΣF_h sind dort ebenfalls angegeben. Die nach dem Prinzip der virtuellen Verrückungen formulierte Gleichgewichtsbedingung für die oben beschriebene kinemati-

- R_i Reibungskräfte parallel zur Wandebene infolge des Erddrucks
 E_{oi} Erdruhedruckkräfte auf Stirnpfahl III bzw. IV
 W_i Wasserdrukkräfte auf Stirnpfahl III bzw. IV

Der Ansatz eines passiven Erddrucks auf die Stirnpfähle ist hier nicht zulässig, da zu seiner Mobilisierung Verschiebungen in der Größenordnung von Zentimetern vorausgesetzt werden müssen, eine gegenseitige Kämpferverschiebung von rd. 1,5 cm bei der oben genannten Gewölbegeometrie aber theoretisch bereits zum Durchschlagen des Gewölbes führt. Für die Abschätzung der Reibungskraft R_i wird der Ansatz $R_i = E_i^* \tan \varphi'$ empfohlen. Hierin sind φ' der innere Reibungswinkel des Bodens, E_2^*, E_3^* die Erdruhedruckkräfte E_o oder die erhöhten aktiven Erddruckkräfte $0,5(E_o + E_a)$ auf der Bodenseite der Wand und E_1^*, E_4^* die abgeminderten passiven Erddruckkräfte $\frac{E_p}{\eta}$ mit $\eta = 1,5$ auf der Baugrubenseite der Wand. Diese Erddruckkräfte sind für den Wandreibungswinkel $\delta = 0$ zu ermitteln, damit die Reibungsfähigkeit zwischen Wand und Boden für die Aufnahme des Gewölbeschubs vollständig zur Verfügung steht. Bei den Verhältnissen der genannten Baugrube in Hannover wäre zur Aufnahme der Gewölbeschubkraftkomponente S_H eine Wandscheibe von rd. 20 m Länge erforderlich. Bei kürzerer Ausbildung der Wandscheibe kann die Differenzkraft $[S_H - R_i - E_i - W_i]$ bei Versprünge auch über das im Bild 4 skizzierte Steifensystem D_1, D_2 und Z aufgenommen werden, wobei die quer zur Baugrubenachse ohnehin erforderlichen Steifenlagen im Bereich des Versprungs entsprechend zu verstärken und zusätzlich Druckstreben D_1 anzuordnen sind. Letztere sollten mit den Steifen Z kraftschlüssig verbunden werden, da in den Bohrpfahl V nur Druckkräfte einzuleiten sind. Dieser Bohrpfahl ist auch für Biegebeanspruchungen in der Wandebene infolge der Linienlast S_H zu bemessen, da ein ausreichender Schubverbund zwischen einzelnen Bohrpfählen der Wand nicht gewährleistet werden kann.

BESONDERE ASPEKTE DER HERSTELLUNG

Die hier empfohlene Variante einer überschnittenen Bohrpfahlwand mit je drei gewölbeförmig angeordneten Zwischenpfählen zwischen je einem Tragpfahl erfordert keine nennenswerten Änderungen im Herstellungsablauf gegenüber der klassischen Bauweise. Dagegen ist die Herstellung der Variante mit nur zwei Zwischenpfählen als Gewölbe insofern problematischer, als der zweite Zwischenpfahl jeweils einseitig in den Beton des ersten Zwischenpfahls eingeschnitten werden muß (Bild 2a). Wegen des dabei auftretenden asymmetrischen Widerstands an der Bohrkrone sind größere Abweichungen der Pfahlachse von der Sollage möglich. Weiterhin besteht bei der zuletzt genannten Variante eher die Gefahr von Verwechslungen beim Einsetzen der Bewehrungskörbe. Bei beiden Varianten ergeben sich aber im Vergleich zur klassischen Bauweise größere Probleme immer dann, wenn, beispielsweise wegen eines Findlings im Baugrund, ein bewehrter Pfahl nicht ordnungsgemäß hergestellt werden kann. In solchen Fällen können besondere "Überbrückungskonstruktionen" über 5 bzw. über 7 Bohrpfahldurchmesser erforderlich werden.

SCHRIFTTUM

- [1] Empfehlungen des Arbeitsausschusses "Ufereinfassungen", Verlag W. Ernst u. Sohn, 7. Auflage 1985

STRESS-TIME STATE OF ANCHOR RETAINING WALLS
FOR DEEP EXCAVATIONS

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Anchorage of sheet-pile, diaphragm and other retaining walls are widely used owing to the possibility to apply anchors in natural (non cohesive or cohesive) soils. These anchors start to work after prestressing when all system-soil massive in the anchor root zone (bond-to ground length) and retaining structures are stressed.

Prestressing has many advantages such as: simultaneity of anchors work (when in group) decrease of deformations of retaining structures during the excavations; improvement of carrying capacity of anchor-wall-massive system and others. On the other hand it is very difficult to predict these phenomena because it is not enough to involve in a work the anchor after prestressing, it is necessary to mobilize the tensile forces where decreasing the stress does not unload the soil massive. Therefore it is necessary to determine the pressed effort, which after decreasing during the pressing and excavation period will provide security of system structure - anchor - soil massive.

Stress-state-time in anchor depends on the change of the stress state of the soil massive and retaining structure. Neglecting relaxation in anchor steel it is possible to accept the anchor root creeping as a general reason for the pressed effort decrease. The quantitative and qualitative determination of decrease is one of the difficult problems in Soil Mechanics. In French code (TA 77) it is noted that the stressed effort decrease is

not well known (see Fig.1 part "6" - "7").

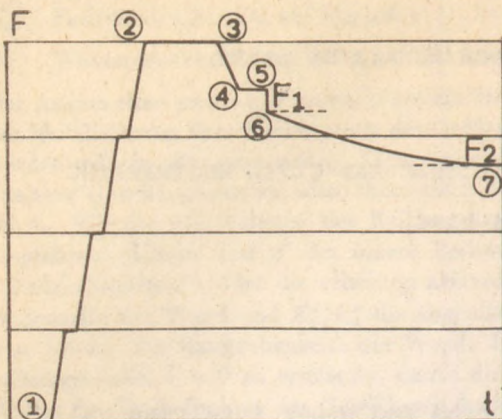


Fig.1 Prestressed anchor time behaviour.

An attempt is made in this report to give an idea for the decrease determination using a well known equation of Soil Mechanics. Principally, determination of stressed effort F_1 if we know minimum tension F_2 necessary for the system stability is considered here. If we consider anchor as an elastic element working simultaneously with soil (Fig.2) the relationship between efforts F_1 and F_2 can be present:

$$F_2 = \frac{E_0 A}{F_0 A + E_a A_a} F_1, \quad (1)$$

where F_0 and A are modulus of linear deformation and cross section of mobilized by anchor "soil", E_a and A_a are elastic modulus and cross section of anchor.

Analogical explanation method of prestress stages is accepted by Hanna and Sparks (1973).

The strain behaviour of natural soils, however is not as of an elastic material. Further more the area of the mobilized soil massive does not have a constant cross section. A more real presentation of the strain behaviour of the stressed soil massive is possible by using solutions of the linear deformation medium theory.

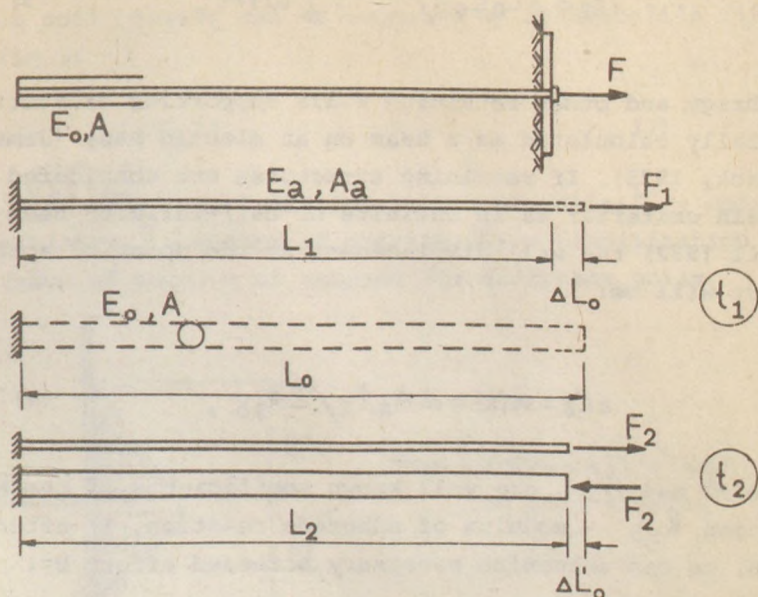


Fig.2 The prestressed anchor system

t₁ - prestressing stage, t₂ - loading stage

For example full displacement of anchor tied by means of a separate block with limited size (area A) can be determined by the well known Schleicher formula:

$$\Delta L_0' = \frac{1 - \nu^2}{E_0} \sqrt{A \cdot \frac{F_2}{A}} \cdot \omega, \quad (2)$$

where ν is coefficient of Poisson, ω - coefficient depending on the retaining structure form and geometrical parameters of deformative soil massive (Bojinov, at all, 1982).

Then for the tension effort we receive

$$F_2 = \Delta L_0 E_0 A / (1 - \nu^2) \cdot \omega \sqrt{A} . \quad (3)$$

Diaphragm and other retaining walls supporting excavations, are generally calculated as a beam on an elastic base (James E., B. Jack, 1975). If retaining structures are considered (using certain criteria) as in infinite or half-infinite beam (Bojinov at all 1982) the wall displacement at the point of action of effort will be:

$$\Delta L_0 = w(x) = \alpha A_x \cdot F_2 / 2K_{s,b} , \quad (4)$$

where α and A_x , are well known coefficients of the Winkler's solution, $K_{s,b}$ - modulus of subgrade reaction. If effort F_2 is given, we can determine necessary stressed effort by:

$$F_1 = \frac{L_0/E_a A_a + \alpha A_x / 2K_{s,b} \cdot F_2}{L_0/E_a A_a} \cdot F_2 , \quad (5)$$

A similar formula can be obtained for F_1 if structure is considered as half-infinite in the case when anchor is tied at the end of wall.

Formulas (1) to (5) give the right results on condition that displacement develop instantaneously. This condition can be accepted in cases when anchorage is made in non-cohesive soils. In the cases, cohesive soils, stress effort decrease changes in time. Stress time can be received, for example, if in formula (5) we obtain function of modulus of subgrade reaction in time $K_{s,b}(t)$.

If the function strain time of the constant load action p is

known we will obtain:

$$K_{s,b}(t) = s(t)/p. \quad (6)$$

The load on soil massive can be accepted by Schneebeli's (1972) formula (Fig.3)

$$p = F_2/2L, \quad (7)$$

where $L = \sqrt{2EI/K_s}$ is "elastic" length of walls; E - modulus of elasticity, I - moment of inertia; K_s - stabilization constant value of modulus of subgrade for different soils.

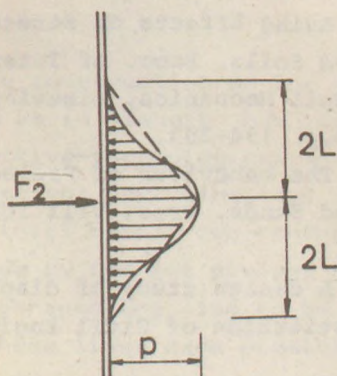


Fig.3 Schneebeli's load on Soil Massive

Deformation function can be determined by (Germanov, 1988):

$$s(t) = \frac{p \cdot L_x}{2(1+e_0)} \left\{ m_0 + m_c \left[1 - \frac{1}{\eta T_c} \psi(t) \right] \right\}, \quad (8)$$

where L_x is the soil deformation zone, which can be accepted $4L$; e_0 - initial void ratio; m_0 - coefficient of instantaneous deformations; m_c - the coefficient of creep (secondary) volum deformation; η - the parameter of creeping speed; T_c - the time of anchor prestressing;

$$\psi(t) = \exp[-\eta(t - T_c)] - \exp(-\eta t), \quad (9)$$

The analysis of the above shown formulas show that taking into account the decreasing of the stress effort (at the reason of creeping of soil) it is necessary to design the anchors with a bigger length and a smaller cross section and smaller modulus of elasticity of steel (anchor cable). A possibility for quantitative estimation of these phenomena give the idea of this report.

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BIFUNCTIONAL SLURRIES FOR SECTIONAL CUT OFF DIAPHRAGMS OF THE SOFIA METROPOLITAN CONSTRUCTION

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The construction of the Sofia Underground first section of 6,15 km in length has started since 1979. Different constructive-technological systems have been applied in the construction including cut off diaphragms. Only casted (monolithic) reinforced-concrete cut off diaphragms have been made by now but precast reinforced-concrete paneled cut off diaphragms are also to be introduced. The studies done along these lines made possible the classification of the reinforced-concrete panels as the following:

- a) light type- from 8,40m to 10,80m in length and to 36t in weight,
- b) average type- from 12m to 15,60m in length and to 60t in weight,
- c) heavy type- from 16,80m to 19,20m in length and to 72t in weight.

The cut off diaphragms will be performed in alluvial gravel-sandy soils with a clay filler and underground water presence at different depths.

The sectional cut off diaphragms construction for the Sofia Underground made necessary the study and defining of recipe compositions of bifunctional cement-bentonite slurries (BFCBS) with chemical admixtures retarding the cement setting. The reological characteristics of the slurries and the physico-mechanical characteristics of the

hard-set product have been studied under laboratory conditions at the following requirements depending on the panel type:

a) reological and technological characteristics of the slurries:

Table 1

Panel type	Volume density (G), kg/m ³	Viscosity (V), s	Bleeding (W), %	Setting time (ST), hours
light	max 1340	max 50	max 4	>24
average	max 1340	max 45	max 4	>32
heavy	max 1340	max 45	max 4	>48

b) physico-mechanical characteristics of the hard-set product:

Table 2

Panel type	Minimum compressive strength, MPa		Filtration
	on the 28 th day (R28)	on the 90 th day (R90)	coeff. (K), m/s
light	0,3	0,5	to 10 ⁻⁷
average	0,4	0,6	to 10 ⁻⁷
heavy	0,5	0,7	to 10 ⁻⁷

In order to prove the above given characteristics methods for assessing them were specified as it follows:

The viscosity was defined by means of the "Marsh" funnel, while the time for the flowing out of 1000cm³ slurry mixture in seconds, immediately after the slurry was prepared, was used as a criterion.

The bleeding is an important index, characterizing the slurry mixture stability, i.e. its propensity to sedimentation and segregation. It was measured 24 hours later using a graduated cylinder with a volume of 100m³ and a diameter of 20mm.

The setting time of the BFCBS is an important characteristic, related with its "sturdiness" still to behave as a fluid. After the preliminary examination we found out that in order to determine the BFCBS setting time

we can use the standard Vica device, attaching the pole used to determine the standard cement density, instead of the needle. It is accepted that the completion of the cement setting is that moment when the pole (with diameter of 10mm and weight of 300g) stops its sinking into the slurry. The various experimental studies, done according to this modified method, showed that using it we can rate precisely the effect of the influence of the set-retarding admixtures with a very good reiteration of the results.

The compressive strength has been determined, using cylindrical specimens with a height of 100mm and a diameter of 50mm. The latter are cured in water with a temperature of $20\pm 3^{\circ}\text{C}$ till the moment of testing.

The experimental data for the cement-bentonite mixtures show that after the 90th day the filtration coefficient change is rather slowed down [1]. The examination of it during the interval between the 28th and 90th day is satisfactory as to the requirements of the Underground construction.

The laboratory experiments are performed according to the express method (without a consolidation), using a three-axial device, at a constant pressure gradient. The experimental scheme allows the filtration coefficient and the one axial compression strength to be determined using one specimen only.

The finding out of the most appropriate BFCBS mixture compositions or composition ranges is a complex experimental problem. The object of investigation is a bicomponent silicate system comprising chemical admixtures also. Various conflicting with each other technological and technical requirements are claimed towards the system properties. This determined the mathematical designing of the experiment as the most proper one to be applied. It was accepted that the influence of the three basic factors in the BFCBS composition should be studied : X1 - cement(C),

kg/m³; X2 - bentonite (B), kg/m³; X3- admixture (A), % on C. Water quantity, ranging in very narrow limits, has been calculated to 1m³ compact volume. Two separate experiments with each of the admixtures are performed, having determined the factor change ranges by means of preliminary tests. They are presented in coded and natural co-ordinates in Table 3.

Table 3

Xi	I st experiment (ad.A)			II nd experiment (ad.B)		
	C	B	A(A)	C	B	A(B)
+1	430	50	1,3+0,4	430	50	1,4
0	350	40	1,0+0,4	350	45	1,0
-1	270	30	0,7+0,4	270	40	0,6

Plan B3 similar to the D-optimal one, with 14 experimental points [2], was used to obtain "composition-property" model of the type:

$$Y = b_0 + \sum_{i=1}^3 b_i \cdot X_i + \sum_{i=1}^3 b_{ii} \cdot X_i^2 + \sum_{i,j=1}^3 b_{ij} \cdot X_i \cdot X_j .$$

A group of models was obtained from the experimental results by means of step regression analysis, which describe adequately (according to the Fisher criterion) the influence of the selected factors (the BFCBS composition) on the studied properties within the framework of the factors' space (Table 4)

Table 4

bi	I st experiment (ad.A)				II nd experiment (ad. B)		
	V,s	W,%	ST,h	R28,MPa	V,s	ST,h	R28,MPa
b0	45,5	2,5	28,6	0,55	37,5	35,5	0,82
b1	4,2	-0,3	-5,3	0,32	0,4	-7,0	0,34
b2	11,2	-1,7	-1,5	0,03	2,7	0,0	0,00
b3	2,2	0,0	3,1	0,04	0,6	17,1	0,21
b11	0,0	-0,8	0,0	0,00	1,1	0,0	0,00
b22	2,7	0,8	0,0	0,00	1,2	0,0	0,00
b33	0,0	0,0	0,0	-0,07	0,0	1,8	-0,14
b12	1,6	0,0	0,0	-0,06	0,0	0,0	0,00
b13	0,0	0,0	0,0	0,00	0,0	-4,5	0,00
b23	2,0	-0,2	-0,9	0,00	0,0	1,5	0,00

The analytical and grapho-analytical interpretation of the models showed that the influence of the separate factors on the studied properties follows the basic tendencies known from the practice and described in the literature. That's why and in order to be brief, we are not going to do a detailed analysis on the complex of models but we will just use them to determine compositions and composition regions suiting the above mentioned requirements for the BFCBS.

Ist experiment with admixture A

The A-admixture has a moderate retarding effect, which grows up to 6h depending on its dosage. It raises up slightly the V and R28 and doesn't influence W.

Compositions are obtained within the framework of the experiment, which properties change within the following limits: $G=1180-1310 \text{ kg/m}^3$; $V=34,4-70s$; $W=0,5-5,0 \%$; $ST=18-40h$; $R28=0,15-1,03 \text{ MPa}$. Fig. 1 shows the range of compositions which satisfy the basic set of requirements for a light panel type. It is obvious that these requirements are satisfied within some comparatively wide but related to each other limits of factors' changes. Some of the compositions included in this range can be approximately described in the following way: $C=300-430 \text{ kg/m}^3$; $B=35-40 \text{ kg/m}^3$; $A(A)=1.0+0,4-1,3+0,4 \%$.

The requirements for the average panel type narrows sharply the range. The conditions are satisfied only at a high admixture dosage $A(A)=1,3\%$; $C=330-350 \text{ kg/m}^3$ and $B = 34-36 \text{ kg/m}^3$ (Fig 1- hatched area).

Solutions of the problem for still a retarded setting and a higher strength (of the material) can't be obtained using this admixture. A more powerful retarder must be used, such as the B admixture.

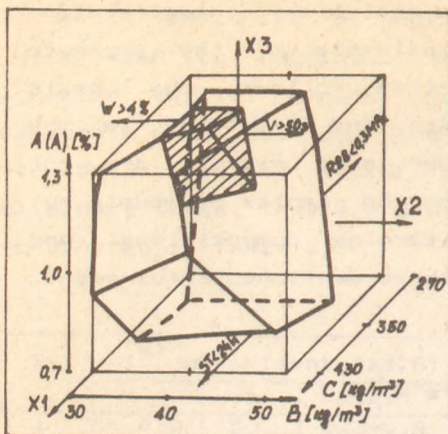


Fig 1.

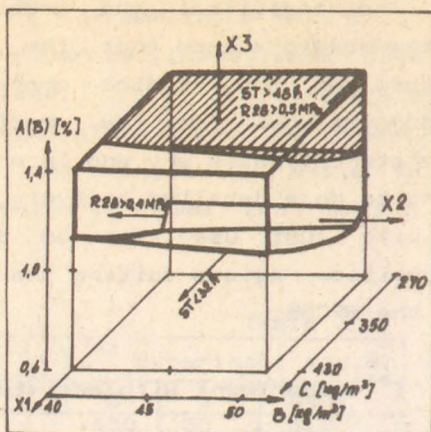


Fig 2.

IInd experiment with admixture B

The results show that the B admixture is more powerful (in retarding the setting process) and within the range of its dosage changes the retarding effect up to 34 hours. Together with this the discussed compositions have low viscosity and higher strengths on the 28th day. A tendency of increased bleeding exists.

Within the framework of the experiment the studied properties change within the following limits: $G=1190-1320 \text{ kg/m}^3$; $V=35,6-43,6 \text{ s}$; $W=2,0-12,5 \%$; $ST=15-68 \text{ h}$; $R_{28}=0,12-1,23 \text{ MPa}$.

As it can be seen from these results and from Fig 2 the basic and the raised up technological and technical requirements (at a certain compromise about the bleeding) are satisfied within the whole range of changes of C and B at a B-admixture dosage more than 1,0%.

Using B-admixture in greater quantities ($>1,2\%$) allows the obtaining of BFCBS with low viscosity ($V < 45 \text{ s}$), a great retarding effect ($ST > 48 \text{ h}$) and high strengths on the 28th day ($R_{28} > 0,5 \text{ MPa}$) within a wide range - $C=270-380 \text{ kg/m}^3$ and

$B=40 \text{ kg/m}^3$ (Fig 2- hatched area).

Table 5 represents the experimental and the model magnitude values of the studied properties at points of the experimental plan which get into the discussed compromise ranges of compositions, satisfying the basic of the raised up requirements towards the BFCBS.

Table 5.

N ^o	Composition			Magnitude values of the properties							
	C	B	A	V, s		W, %		ST, h		R28, MPa	
	kg/m ³	kg/m ³	%	exp	mod	exp	mod	exp	mod	exp	mod
I st experiment with admixture A											
5	430	30	1,3	39,0	39,8	3,9	4,2	29,0	28,8	0,70	0,76
13	350	40	1,3	47,6	47,7	3,0	2,5	31,0	31,7	0,49	0,52
14	350	40	0,7	43,6	43,3	2,0	2,5	25,0	25,5	0,44	0,43
II nd experiment with admixture B											
3	270	40	1,4	37,8	37,2	6,0	-	65,0	64,4	0,50	0,54
4	430	50	1,4	43,6	43,4	6,5	-	46,0	44,4	1,12	1,23
6	270	50	1,4	43,4	42,6	4,0	-	68,0	67,4	0,43	0,54

The results from the filtration experiments with cement-bentonite specimens show that K changes from $1 \cdot 10^{-7}$ to $1,5 \cdot 10^{-8}$ m/s, which satisfies the requirements for sectional out off diaphragms.

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THE SAFETY OF THE DEPENDENT PRESSURES METHOD

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SYNOPSIS

The Paper shows a insufficient accuracy of Winkler's elastic-plastic material instead of a pressing soil mass beyond retaining wall. Thus, a used procedures risk follows as consequence of the unaccuracy. The risk is analysed. The more accurate models of ideal materials are suggested for these purposes.

INTRODUCTION

The Dependent Pressures Method (DPM) is one the more advanced 2 practical methods used for the retaining wall analyses those are applied especially in urban areas where DPM has proved to be good. DPM is based both on the force and deformation conditions and it was considered as more or less the theoretically complete and the reliable. Two impulses to a new judging DPM were. Firstly, a new hotel building in the site of the metro underground entrance near Masaryk's railway station in Prague (IB line - st. Náměstí Republiky) where the original underground walls were analysed in 1980 by modern Blum's method [3] and the new walls for the hotel have analysed by DPM in 1990. Secondly, a steadfast belief of young engineers to the any computer outputs.

RELIABILITY OF RECENT PROCEDURE

DPM (originally Polygonal Method) originated in Metroprojekt in 1980. The autor, A. Zapletal [10], had derived it by program OSPL which is analysing tunnel structures in Winkler's elastic-plastic mass. The principle of DPM see in Fig. 1.

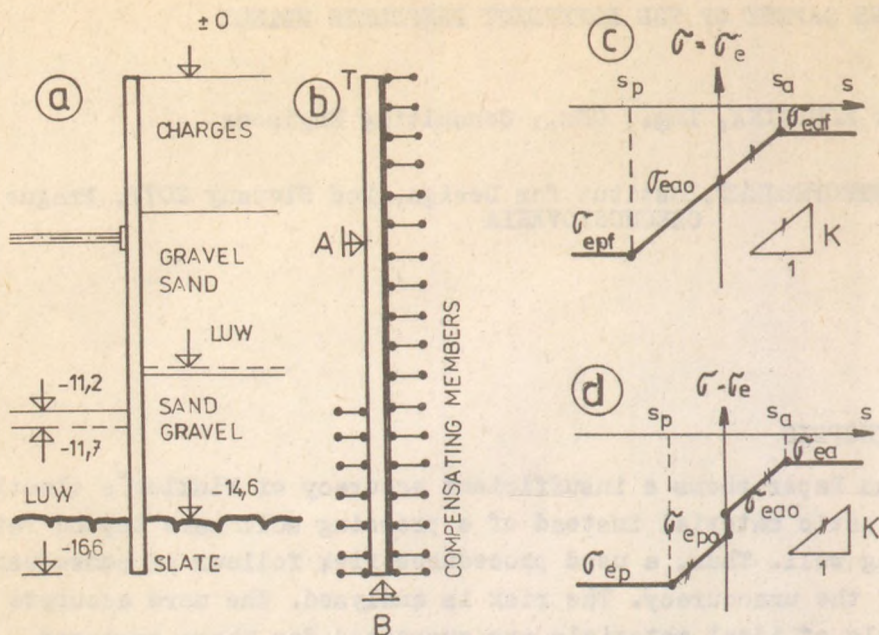


Figure 1. Principle of recent DPM. a - actual structure, b - statical scheme, c - diagram of Winkler's material, d - diagram of noncontinual behaviour of material.

Wall supports are introduced as statical supports including their displacements and the anchors are introduced as prestressing forces or as a combination prestressing force and support. A influence of a real soil mass to wall (Fig. 1a) is compensated by influence of the substitute members (Fig. 1b). These members are created off Winkler's elasticplastic material which behaviour is determined by diagram in Fig. 1c or (in last time) also off the noncontinuos elasticplastic material by diagram in Fig. 1d.

Both movement-pressure diagrams are based on the principle of the original tunnel analysis and thus, both on the coefficient of subgrade reaction K and basic earth pressure values (at rest - σ_{eao} , σ_{epo} ; top - σ_{eaf} , σ_{epf}). The movement values s_a and s_p are calculated by these inputs and they are not the necessary movement values for a mobilization of the respective pressures.

Two facts are posing against the belief in total reliability of DPM. First, the any criteria for a safety judgement are missing in the theoretical base of DPM (reliability is judged by movements of the wall and for the wall bottom by the ratio of whole calculated passive pressure to the whole top passive pressure). Then also, considered movements are in no defined relation to considered mobilization shear strength though the safety is judged by movements.

ANALYSIS OF WINKLER'S MATERIAL CONVENIENCE

According to the extend of the Paper the analysis is concentrated in four principal questions:

1. Coefficient of subgrade reaction K.

This one is expressing a remodeling earth mass behaviour at the pressure. Its investigation is not very reliable one. However, it is serious that this coefficient does not express the soil behaviour on the slip surfaces where shear strength is mobilized. The coefficient of subgrade reaction K using by Figs. 1c, 1d is not the adequate to its physical essence.

2. Noncontinuity of pressure at rest.

The pressure at rest noncontinuity in the behaviour of elastic-plastic material (Fig. 1d) is possible to consider as the correct [7] however, then it is not the Winkler's material yet.

3. Safety of wall bottom.

This one which is calculated by the recent DPM, can be the fictive in some cases and it is in the dangerous site owing to the uncorrect earth pressure division. The analysis model utilizes maximally shear strength in front of upper bottom part what is statically the most advantageous. However, in fact so quick mobilization is not to be and a shear strength in deeper area of wall bottom is utilized. That leads to the less favourable values.

4. Plastic areas.

According to reliability and safety the plastic areas of earth mass behaviour are most serious problem. Winkler's material and that one by Fig. 1d are the plastic on the top shear strength level. There is not considered a decrease of strength on the residual value (Fig. 2). The decrease makes a pressure increase

at the active wall site and also a pressure decrease at the passive wall site. The retaining wall safety is decreased including supports materially by this.

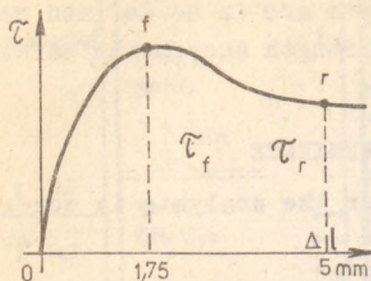


Fig. 2. Shear strengths in displacement-shear resistance diagram.

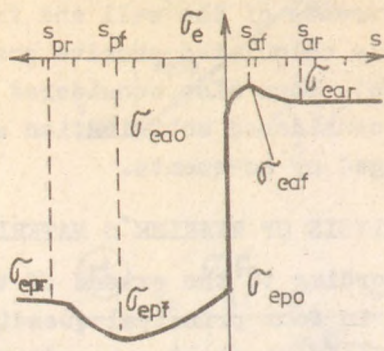


Fig. 3. Movement-pressure diagram of actual earth mass.

RISK OF THE TOP SHEAR STRENGTH REACHING

The noticed case shows well the risk of the underground walls near Masaryk's railway station (Fig. 1a) when the recent DPM procedure is used. The results of calculation by DPM and original results (1980) by the modern Blum's method are in Fig. 4.

If we do not want to exceed the limit displacement of top shear strength in bottom area (see 4) and if we want to gain the earth pressure division in accord with the real conditions of safety design, it has to modify the basic DPM diagrams (Figs. 1c, 1d) by reduction coefficient k_r (Fig. 4e). This one ensures against both the reaching and exceeding of given values. Fig. 4a shows that calculated safety coefficients with low values k_r are not in accord with safety against the top shear strength exceeding. The accord set in for $k_r \geq 1,5$ only. The bottom safety is therefore 1,5 but not 1,738 by recent procedure. Also the quick increases of all quantities for $k_r > 1,5$ and their risk is seen in Figs. 4b to 4d.

The quantities values for the strength decrease to residual values ($n_r = 0,9$) are noticed by the character R. These prove that in this case it would occur a failure of supports (or an-

chors) and, respectively, wall failure. On the other hand the original design by modern Blum's method [3], [4] is not precise one however, it ensures the unexceeding of top shear strength in front of wall bottom. Moreover, the supporting force value is the adequate and the moments deviations are in the tolerance of wall structure.

The risk of the top shear strength exceeding (see p.4) in a soil the similar to one in investigated case (Figs. 1a,4) is proved by the ratio of the earth pressure with residual shear strength to the earth pressure with top shear strength by the relation $k_{er} = k_{er}/k_{ef}$ (for active pressure k_{ar} , for passive pressure k_{pr}) in Fig. 5. It is seen in Fig. 5 that, in the other cases than the noticed, the risk is the essentially larger.

SUGGESTED MODELS OF IDEAL MATERIAL

The reliable models of the ideal materials for the compensation members must be founded on the real estimate of the actual soil mass displacements in the deciding pressure phases, i.e. the top and residual one. It can not be used the usual notion and the given values of the wall top displacements to this, because they do not reply to the reality. A mobilization of the other then the at rest values of earth pressure requires a nondifferential movement of the wall. However, the soil mass in walls area reshapes itself the whole. Also it is to see by the simplified assumption of the plane slip surfaces that active earth pressure can not come at the bottom slip surface with a toe movement and the firm bottom. Therefore, the earth pressure is not acting at the wall all the same but acting by the dependence with the movement (no turning) and depth. The acceptable material is, thus, that material which diagram approaches to the diagram in Fig. 3 but this one is the variable also.

It is possible to derive the movements at the slip surfaces that have to be, for the given phase of the shear strength mobilization Δl_f , Δl_r from the box tests or the modules of shear strength mobilization [8] Ω_f and Ω_r by the relations

$$\Delta l_f = \frac{1}{\operatorname{tg} \varphi_f} \frac{3}{2} \frac{\Omega_f}{1 + \lambda_f} \quad (1) \text{ and}$$

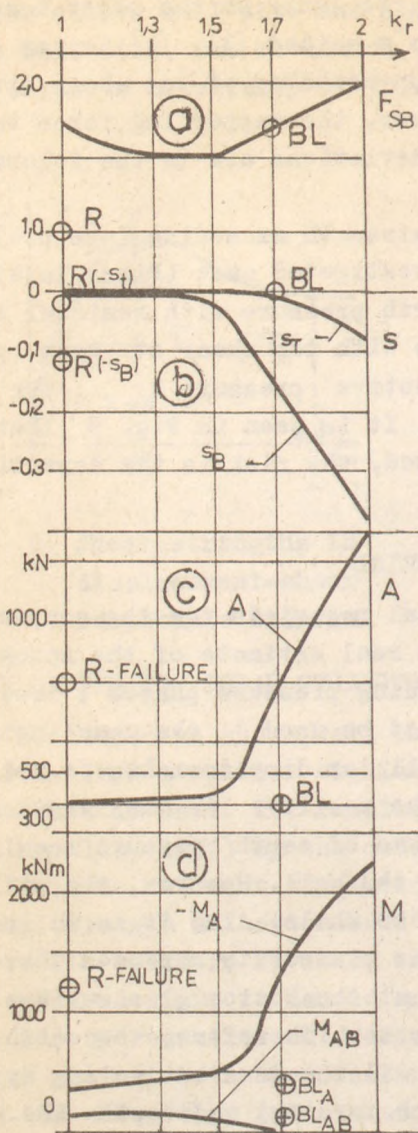


Fig. 5. Ratio of residual earth pressure to top shear strength one.

- a - measurement of depths
b - ratios of active and passive earth pressure k_{ar} , k_{pr}

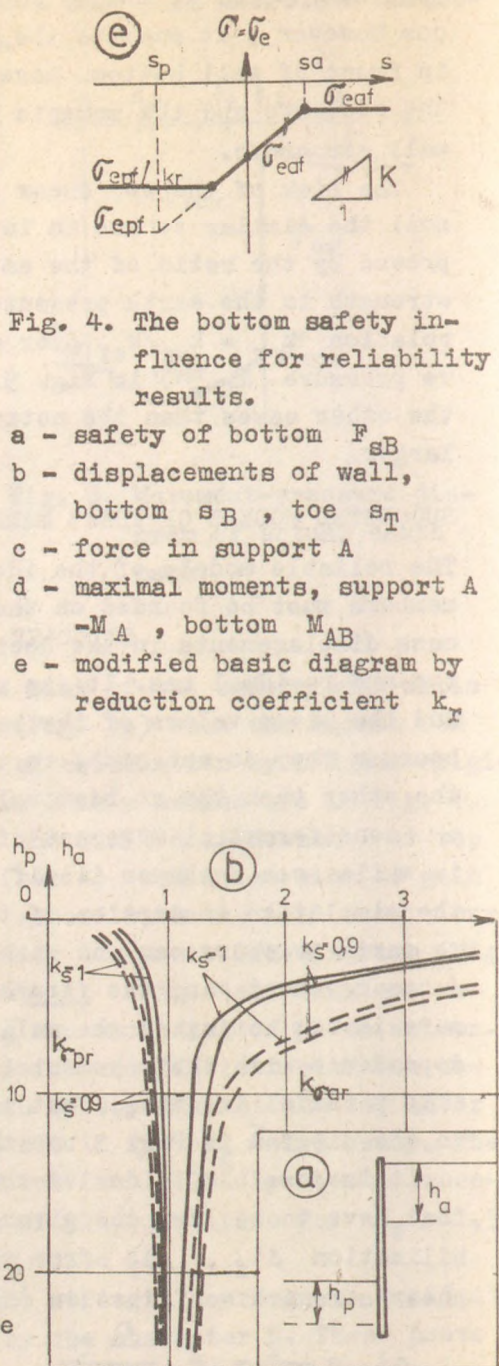


Fig. 4. The bottom safety influence for reliability results.

- a - safety of bottom F_{SB}
b - displacements of wall, bottom s_B , toe s_T
c - force in support A
d - maximal moments, support A $-M_A$, bottom M_{AB}
e - modified basic diagram by reduction coefficient k_r

$$\Delta l_r = \frac{1}{\operatorname{tg} \varphi_f} \cdot \left[\frac{3}{2} \cdot \frac{\Omega_f}{1 + \lambda_f} + \frac{2}{1 + n_r + \lambda_f} \cdot (\Omega_r - \Omega_f) \right] \quad (2),$$

where λ_f is the similarity coefficient defined similarly as for the slopes by the relation

$$\lambda_f = \frac{c}{\gamma \cdot h \cdot \operatorname{tg} \varphi_f} \quad (3).$$

The actual resultant movements of the wall with the limit pressure values can be expressed by relations

$$s_{af} = s_{afl} + s_{afK} \quad (4a), \quad s_{ar} = s_{af} + s_{ar1} + s_{arK} \quad (4b),$$

$$s_{pf} = s_{pfl} + s_{pfK} \quad (4c), \quad s_{pr} = s_{pf} + s_{pr1} + s_{prK} \quad (4d),$$

where s_{afl} , s_{pfl} , resp. s_{ar1} , s_{pr1} are the geometrical wall movements whose have to be amounted to the top (f) resp. residual (r) shear strength at the considered slip surfaces and where s_{afK} , s_{pfK} resp. s_{arK} , s_{prK} are the soil mass deformations due to a change of earth pressure. The movement s_{pfK} depends on the coefficient of subgrade reaction K only, the other three depend on the coefficient of elastic subgrade reaction K_E .

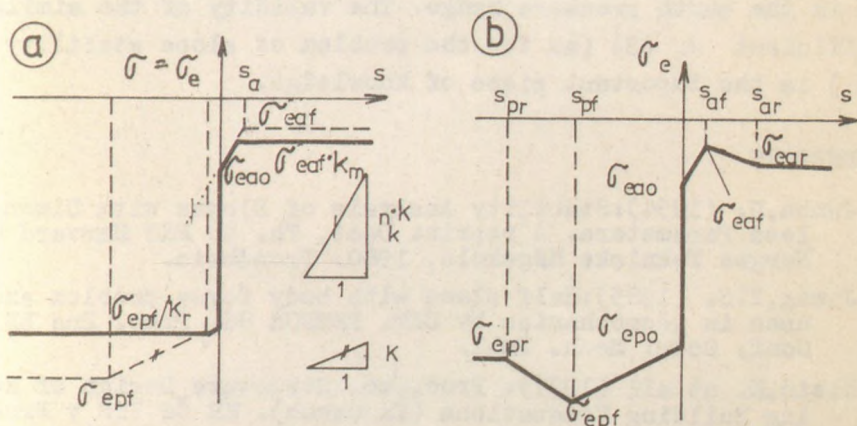


Fig. 6. Suggested movement-pressure diagrams of ideal compensating material; a - noncontinuous bilinear elasticplastic material; b - noncontinuous polylinear elasticplastic material.

It is possible to put together the movement-pressure diagrams of the suitable ideal material for the compensating members. Diagrams have to make on the base of a estimate of the wall movements in the earth pressure deciding phases. Thus, the behaviour of this material comes near to the behaviour of the actual soil mass. A type is in Fig. 6b. That material is possible to call the "variable noncontinuous polylinear elasticplastic". The recent software founded on DPM can be modified by diagram in Fig. 6a.

CONCLUSION

The recent DPM procedures have not been ensuring a reliable and safety judgement of the retaining walls. To the advance of the reliability, the safety and the economy, DPM has to be based on the definition of the variable noncontinuous polylinear elasticplastic material (Fig. 6b). The existing software has to be modified by diagram in Fig. 6a. This conception requires to complete following inputs: residual shear strength (ϕ_r , c_r), lengths of critical movements (Δl_f , Δl_r - attention for dependence on the sample size!).

It has to be respected both a earth variability and the dependence of the critical slip surfaces on the earth properties.

The Hamilton's similarity coefficient $\pi = c/\gamma \cdot h$ is the valid in the earth pressure range. The validity of the similarity coefficient λ (3) (as for the problem of slope stability [1], [6]) is the important piece of knowledge.

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RECENT TECHNIQUES OF SUBSURFACE CONSTRUCTIONS USED AT BUDAPEST UNDERGROUND LINES

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1. INTRODUCTION

At the stretches of the line North-South built before 1983 diaphragm walls have been used for lateral retaining of the excavation. This method can undoubtedly be judged as a saving one because the definite lateral part of the construction and the temporary retaining function are combined (Fig. 1.).

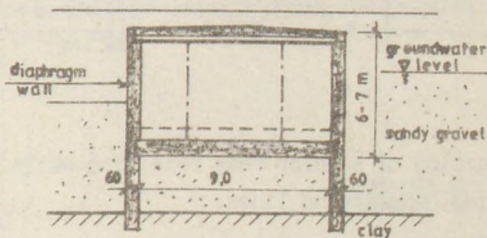


Fig. 1. Subsurface underground construction with retaining diaphragm walls

For the following section (Lot III/B) a frame-shaped reinforced concrete cross-section containing prefab crust-formwork wall-elements was chosen because of efforts to reduce the necessary labour at the site and beyond that, influenced by the hydrogeological circumstances.

This construction method, using precast crust-formwork elements concreted-in on the spot, contains advantages of prefabrication and mounting-type building, provides high quality and favourable appearance.

The track of the line section mentioned above runs parallel to the river Danube and there is a relatively high groundwater level (4 to 6 m down from surface). The groundwater flows (near) perpendicularly to the track. Traditional watertight diaphragm walls reaching down to impermeable clay layers may hinder that continuous flow which takes place in the upper layers of sandy-gravel soils.

To avoid this influence, an updated version of conventional soldier beam retaining walls combined with reinforced concrete frame-shaped cross-sections have been used along this section (see Fig. 2.).

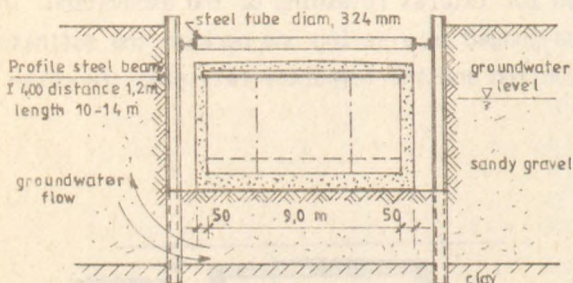


Fig. 2. Updated version of soldier beam retaining walls used for constructing underground lines.

2. EARTH RETAINING METHODS USED

It was obvious to use conventional soldier beam retaining walls for constructing line sections between stations, as shown on Fig. 2. The strong propping up of new type steel-tube struts used at the top (and, if necessary, at the bottom) of the beams simplified considerably the method.

Along sections where the ramming of steel I profile beams would have generated dangerous vibrations soil nailing was used for the lateral stabilizing of the working pit. The first testing application of soil

nailing has taken place at the Árva utca part of the section. The slope had an angle of 10^0 to the vertical and a depth of 7 m and was reinforced by nails diam. 25 mm and 4 m in length and made of Ahead-ribbed steel rebars.

The construction of the stations has also been designed as a mounting type one. Crust-formwork elements were placed by heavy duty tower cranes or sometimes by mobile cranes. Thus an internal propping up method could not come in question. It was also not possible to use tiebacks because the densely located public utilities and some buildings of poor condition. The supporting system used at the station Forgách utca is shown on Fig.3.

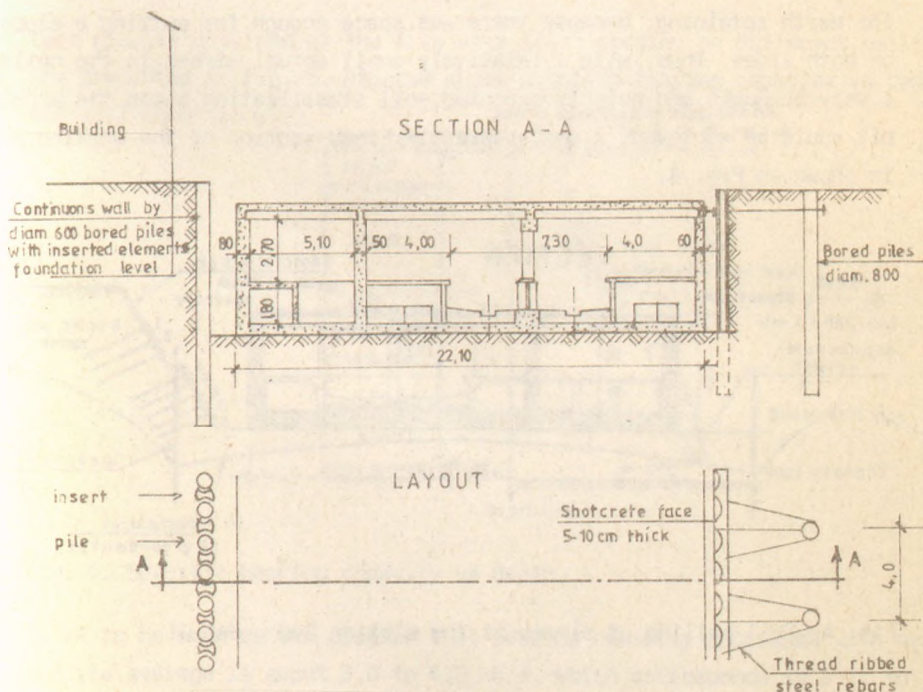


Fig. 3. Supporting system used at the station Forgách utca

In order to protect existing buildings a continuous wall of bored rc piles has been constructed. Particularity of that construction was its composition, i.e. first prefabricated "negative shaped" rc beams were placed into borcholes filled with bentonite slurry, then the piles were bored and concreted ("positive shape") between.

Along the other parts of the working pit the I profiles of the soldier beam walls were tied back to bored piles of diam. 800 mm. These piles were placed prallel to the wall and with a distance of 4 m. As tiebacks thread-ribbed, high yield steel rebars were used. At some spots, spaces between I profile beams were provided with mesh-reinforced shotcrete.

At the station Gyöngyösi út soil nailing has been chosen by the designer for earth retaining, because there was space enough for cutting a slope on both sides. Thus, with a relatively small actual stress in the nails, a very economic and quickly executed soil stabilization along the working pit could be achieved. A characteristic cross-section of the working pit is shown in Fig. 4.

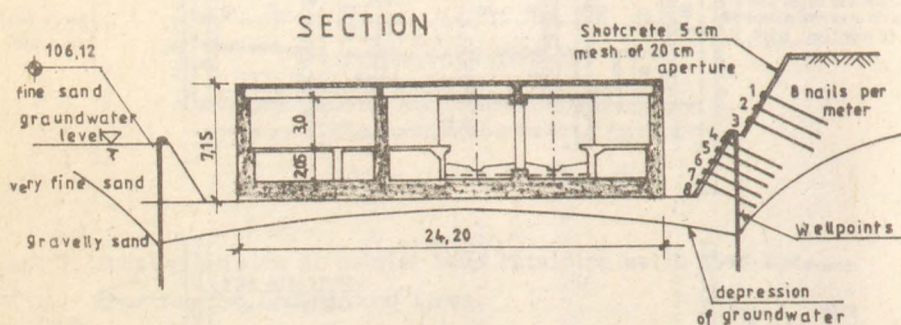


Fig. 4. Soil nailing of slopes at the station Gyöngyösi út

Groundwater was lowered by using wellpoints as shown on Fig. 4. Lowering had been finished before second phase of excavation with the lower row of nails started. Nails were simply driven into the soil, without using any kind of grouting. Load transfer between soil and nails was supposed to be presented by friction only.

Because stabilization of the slope in our case was a temporary construction, possibility has been given to use any kind of linear things that could be driven into soil and had a good friction rate to soil. The character of the construction also allowed the usage of relatively short nails with a dense distribution but a low bearing capacity in comparison.

Tests were made on different kinds of linear things; deformed bars for concrete reinforcement, thread-ribbed steel rebars, smooth steel pipes of diam. 50 mm and other pipes of 50 and 60 mm diameter, having a welded helical 6 mm wire outside.

The load bearing capacity of different kind of nails was surveyed by tension tests.

Test results obtained at the Árva utca trial section on different nails are presented in Fig. 5. Diagram shows specific bearing capacity vs depth and soil conditions.

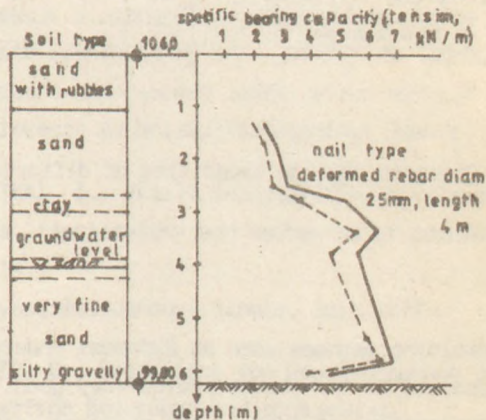


Fig. 5. Specific bearing capacity vs depth

It is to be seen on the diagram that bearing capacity increases with depth and its average is about 3,0 to 4,0 kN/m, which corresponds to a 12 to 16 kN/nail value for nails 4 m long.

It is obvious that a part of nail, stretching beyond the sliding plane, can only be taken into consideration for load bearing.

During tension tests, in addition to maximum and residual tension forces displacement values were also measured.

Finally, results of comparative tension tests with different nail types carried out at the working pit of the station Gyöngyösi út are shown in Fig. 6.

No.	Construction of soil nail	Nos. of tests	Level of the nail (m)	Measured specific tension forle (kN/m)			Displacement (mm)	Measured values of different Nails related to No.1 as 100% (Base)	
				Maximum	Residual (Average)	Residual (in percent of max.)		Residual tension forle	Displacement
1.	Ø 50 mm smooth steel pipe	4	3,04	5,20	4,67	90	5,5	100	100
2.	Ø 50 mm steel pipe with a welded helical wire outside	2	3,04	8,29	7,61	92	40	163	73
3.	Ø 60 mm steel pipe with a welded helical wire outside	1	3,04	10,6	9,83	93	40	210	73

Fig. 6. Compared bearing capacities of different type nails

3. SUMMARY

Earth retaining methods used at Budapest underground construction works and introduced in the above proved very good. Their application enabled quick construction of tunnels and stations without hindering of continuous groundwater flow.

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PRACTICAL EXPERIENCE WITH DEEP SLURRY-WALL SUPPORTED EXCAVATIONS IN CONSTRAINED AREAS

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ABSTRACT

The paper describes the experience get along the time in the deep supported excavations under slurry-walls protection in urban and constrained areas.

It is shown the evolution from the classic cast-in-situ technique to the precast panel solution.

SLAG TANK

General data. The underground tank built in 1977 in a plant in Bucharest required a deep excavation of max. 22.5 m with an almost elliptical shape in plan (fig.1), located in the vicinity of an industrial hall and of a service railway at distances as close as 5.0 m. The excavation had to be performed under the protection of slurry-walls, diged by means of a grab-bucket (Kelly-type). A row of deep wells surrounded the walls, for dewatering during inner excavation.

Design problems. Vertical panels of the walls have been calculated as linear elements of continuous beams supported by perimetral wales placed, in vertical direction, at distances of about 5.0 m. On the semi-circular zones, wales were represented by arches whose stability was insured by providing a strut in between the supports of these arches. One has to mention that in the final stage of the excavation, panels had practically no embedment. Nevertheless, due to the quality of the soil involved, a loss of the stability of the bottom was not expected.

Construction problems. a) After the first panel was completed,

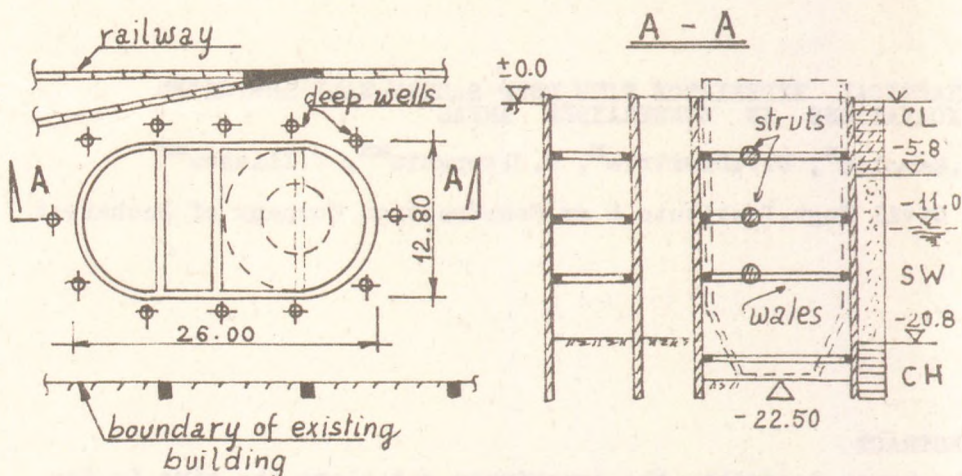


Fig.1 Lay-out & vertical section of the slag tank.

the interlocking pipe could not be extracted with the available equipment. In order to reduce the bond with the concrete, at the following panels a sheet was provided on the entire depth, attached to the reinforcement cage (fig.2).

b) Despite an aquifer made of a medium sand, dewatering of more than 2.0 - 2.5 m could not be obtained during work of the deep wells system. Any attempt to increase the rate emptied the wells. Under the circumstances, the function of the deep wells was discontinued and direct pumping was used, while walls were submitted to the additional pressure of the water column. The infiltration through the walls were reduced proving their fairly good quality, except the very first panel with the pipe

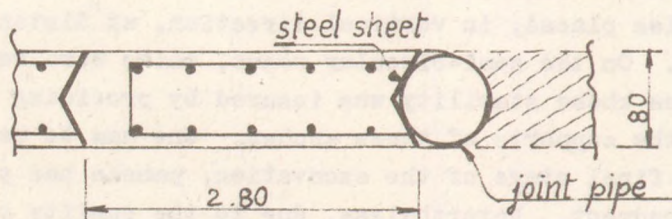


Fig.2 Panels interlocking.

left in place. In the same area, a flow of about 6 - 7 cu.m of dry sand, occurred. The remedial measure consisted of doubling the wall by an additional external panel, to close the defective joint.

c) The bottom of the excavation was closed by means of a relatively thin reinforced concrete plug. Until the completion of the internal structure of the tank, vertical holes were left in the plug and pumping of the infiltrated water was performed, thus reducing the uplift pressure on the plug.

BRACED EXCAVATION FOR THE FOUNDATION OF A 12,000 TONS JACK

General data. The work involves an underground excavation of 13.0 m depth, with a rectangular shape (fig.3), placed in the interior of an existing hall. One of the longitudinal walls of the braced excavation was adjacent to the foundation of the hall columns. Walls were made of panels excavated under bentonite mud with a Kelly grab-bucket. Concrete was placed through tremie in all panels except those located adjacent to the old foundations. A half pipe attached to the reinforcement cage

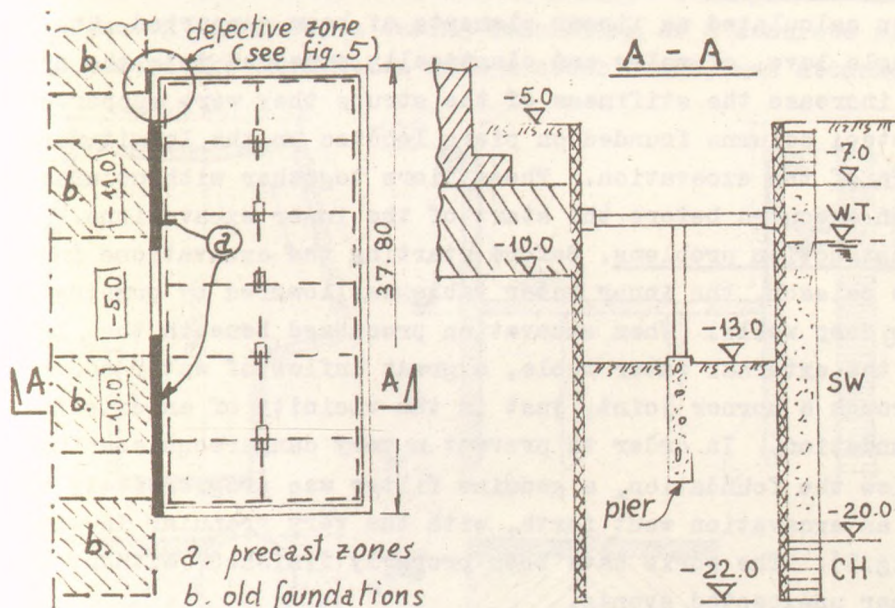


Fig.3 12,000 tf jack excavation.

and left in place formed the joint between cast-in-situ panels (fig.4a). In order to minimize the construction time, panels of precast concrete were used in the proximity of existing foundations (fig.4b). To insure sealing the joints between adjacent panels, the bentonite fluid was replaced, just before lowering the prefabricated panels in the trench, by a self

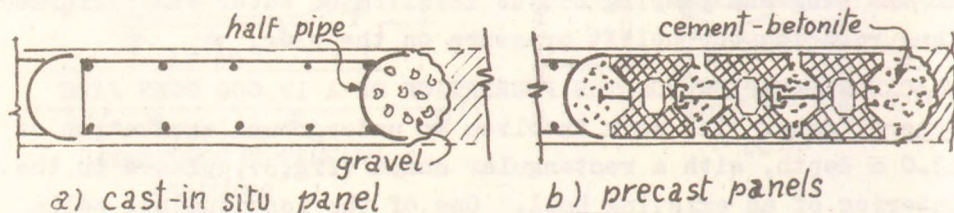


Fig.4 Panels detailing.

hardened cement-bentonite grout, with a density of 1.25 g/cu. cm. At the end of 28 days, the grout achieved a strength of 150 psi.

Design problems. Both cast-in-situ and precast panels have been calculated as linear elements of beam supported at the single level of wales and elastically embedded into the soil. To increase the stiffness of the struts they were supported by a steel columns founded on piers located on the longitudinal axis of the excavation. These piers together with columns have been executed before the start of the inner excavations.

Construction problems. Before starting the excavations inside the caisson, the inner water table was lowered by pumping from two deep wells. When excavation processed beneath the level of the external water table, a great inflow of water occurred through a corner joint, just in the vicinity of an existing foundation. In order to prevent a very dangerous scoring below the foundation, a genuine filter was progressively built-up as excavation went forth, with the very granular in-situ soil (fig.5). The works have been properly finished without any other unexpected events.

UNDERGROUND STRUCTURE OF THE LIFTS TESTING TOWER

General data. The testing tower of 100 m height was designed

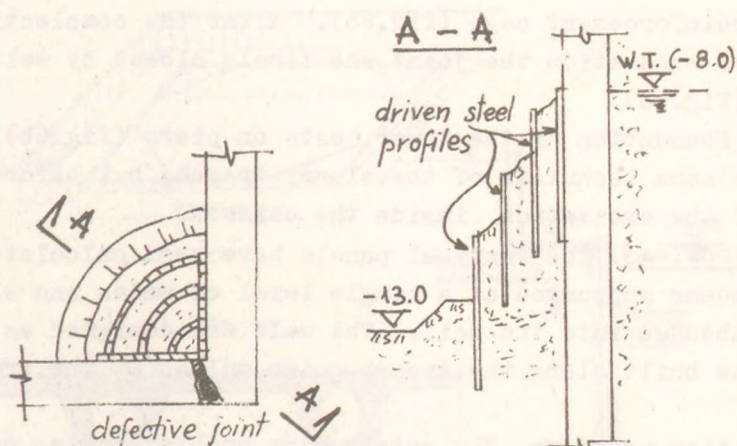


Fig.5 Detail of the defective zone.

with a deep functional underground structure of a special, in plan, shape (fig.6a). This required a 10.0 m deep braced excavation. The walls were made of panels excavated through slurry. The joint between panel were treated as for the slag tank solution. In order to obtain a perfect waterproof wall with a smooth surface, a casing consisting of a concrete slab, coated toward the excavation with a steel sheet, was attached

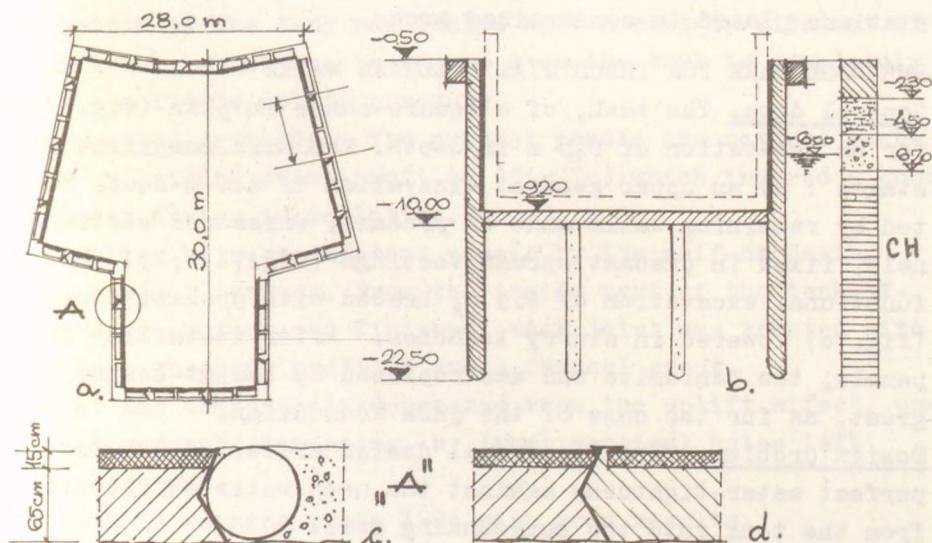


Fig.6 Layout, section & details of the tower foundation.

to the reinforcement cage (fig.6c). After the completion of the inner excavation the joint was finally closed by welding a flat (fig.6d).

The mat foundation of the tower rests on piers (fig.6b), built with the same technique of the slurry trench, but before the start of the excavation inside the caisson.

Design problems. The vertical panels have been calculated as linear beams supported at a single level of wales and elastically embedded into the soil. The wale was designed as a closed frame built along the trench guide walls, at the ground level.

Construction problems. The outstanding problem during construction was to insure a good alignment of the panel cages. In this order, the cages were suspended on to the guide walls and maintained fixed in this position during the concrete pouring through the tremies. The out-of-plane deviations of the panels were no more than 3.0 cm and the opening of the joints was under 4,0 cm.

The works have been performed by Constraction Co. of Bucharest in 1986-1987.

This solution was afterward employed several times for pumping stations placed in constrained areas.

EMBEDDED TANK FOR INDUSTRIAL POLLUTED WATER

General data. The tank, of a square shape in plan (fig.7) asked for an excavation of 8.5 m in depth. The work comprises two stages : a) an upper general excavation of 4.0 m deep, supported by retaining walls made of precast, caissoned section panels, fixed in precast spread footings (fig.7b) ; b) the final functional excavation of 4.5 m, braced with precast panels (fig.7c) lowered in slurry trenches. After installing the wall panels, the bentonite mud was replaced by cement-bentonite grout, as for the case of the jack foundation.

Design problems. The functional design criterion required a perfect water-tightness against the used water exfiltration from the tank into the surrounding soil.

It is worthy to note that before the execution of this work, the phreatic water in the vicinity area of several sq-km was contaminated with hydrocarbon rezidues.

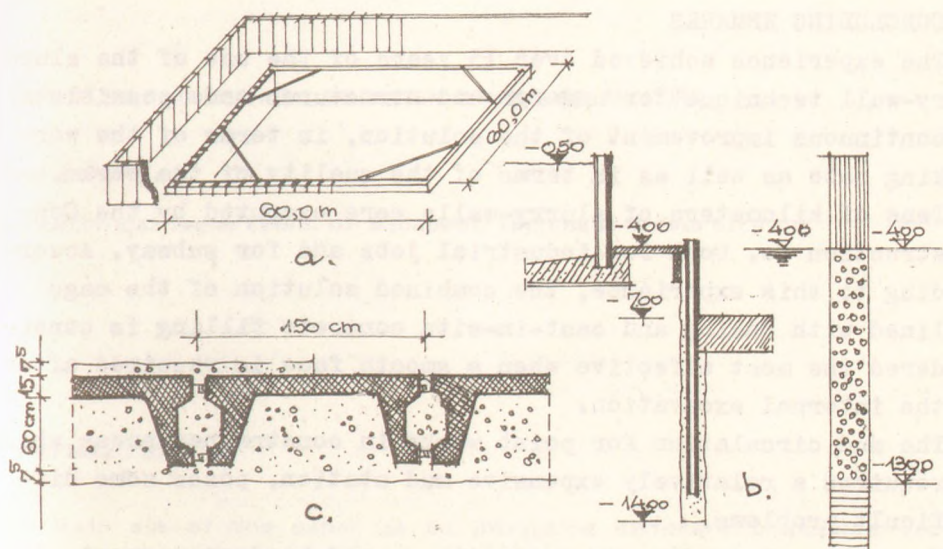


Fig.7 Used water tank.

The wall panels were computed as linear beams supported at the top and elastically embedded into the soil. The single level of the wales was realised as a closed horizontal frame strengthened with diagonal struts. The frame beams consisted of the very pavement slab, at the level of - 4.0m.

The bottom of the tank was sealed with a cast-in-situ mat of 1.5 m thick, which has to bear - when the tank is completely empty - a little uplift pressure.

Construction problems. The precast panels are caught each other by guiding steel profiles (fig.7c) which insured a good alignment during installation.

The sealing between adjacent panels by the self-hardening grout was not perfect. From the inside part of the tank, after the excavation was finished, each joint was treated with polyvinyl chloride profiles and a thiocol grout.

The mat was temporarily depressed from the uplift effect, until the concrete hardening, by local vertical holes left within the mat.

The tank is working since 1986 with good results.

CONCLUDING REMARKS

The experience achieved over 15 years of the use of the slurry-wall technique for underground structures made possible a continuous improvement of the solution, in terms of the working rate as well as in terms of the quality of the works. Tens of kilometers of slurry-walls were executed by the Construction Co., both for industrial jobs and for subway. According to this experience, the combined solution of the cage lined with casing and cast-in-situ concrete filling is considered the most effective when a smooth face is required after the internal excavation.

The mud circulation for point works in constrained areas, which requires a relatively expensive mud station, poses some difficult problems.

When precast panels are used with a self-hardening grout, the working program has to be very carefully fulfilled to avoid spoiling.

ACKNOWLEDGEMENT

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PRINCIPLES AND METHODS IN DESIGN OF GROUND ANCHORS

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1 Introduction

The main aim of the paper is to introduce different principles and methods and compare them by means of numerical examples, thus interpreting the reasons of the deviation in the results.

The fundamental matter in the anchor design process is the accurate knowledge about the load transfer mechanism and within this problem the investigation of the factors that have more effect on the anchor capacity. The deviation in the results of different methods can be attributed to the neglect of the real load transfer mechanism.

Their load bearing capacity depends on the quality and the condition of the soil, the size of the anchor, the applied technology, the grouting pressure and the material of the grout as well. Taking all the parameters into account simultaneously is rather difficult and their good theoretical foundation and satisfactory practical application is hardly possible.

It is the task of the researchers and designers to analyze the conditions and to decide which of these parameters have crucial influence on the magnitude of shearing resistance along the fixed anchor length or the load bearing of the anchorage. The first question to be answered is the following: which of the above mentioned effects have the most influence upon the calculation parameters, that is, a decision should be made in a question of principle. After reviewing many calculation methods the principles can be put into three groups.

In the first group are those methods that are founded upon the geostatic pressures, in the second and third group the geostatic pressures are neglected.

In the second group the shearing resistance that develops along the surface of anchor body during the pulling out is calculated from the dilation, a phenomenon that takes place when the soil particles roll above each other.

The third group of calculation methods sets out from the grouting pressure assuming that from the significant (above 1 MPa) pressure a 25-35 % is retained by the soil even after the hardening of the grouted body and this pressure, acting on the surface of the grouted body, contributed to the development of a higher shear resistance.

2 Anchorage calculation methods

The calculation methods based on the above listed principles differ also in the area they assume the load transfer takes place when the anchor is pulled out, that is

- both at the end and along the side of the fixed anchor length (Bendel, Lendi, Littlejohn);
- only along the side of the anchor (Jelinek, Sommerville, Petrasovits).

In the favour of the feasibility of this latter method has the anchorage making technology since in cohesive soils the size of anchor end is negligible, and only in coarse granular soils the diameter of grouted body might be 2-4 times bigger than the diameter of the bore.

2.1 Methods based on the geostatic pressure

From among the method taking both the end bearing and the side friction into account Bendel's method as the theoretically best based one will be discussed.

Bendel in this method applies Jaky's wedging strength theory for the determination of the end bearing. This theory was suggested by Jaky for the calculation of pile point resistance.

In Fig. 1 Bendel gives the failure conditions of a stripe (end bearing) in plastic medium having $2 \cdot b$ width and infinite length. As it can be seen in the Figure, no skin friction can come into being along the zone that is inside the reversing slip surfaces in failure conditions. The extent of load bearing zone, as it can be seen in Fig. 2 depends on the anchor diameter and the angle of shear resistance of the soil. In the example given by Bendel, the size of load bearing body in case of anchor having fixed length of $l_b = 3.0$ m and diameter of $D = 0.258$ m can be found at different angles of internal friction. At $\phi = 30^\circ$ the whole fixed length situates inside the slip surfaces, consequently in this case counting with side friction is not reasonable and have to be neglect.

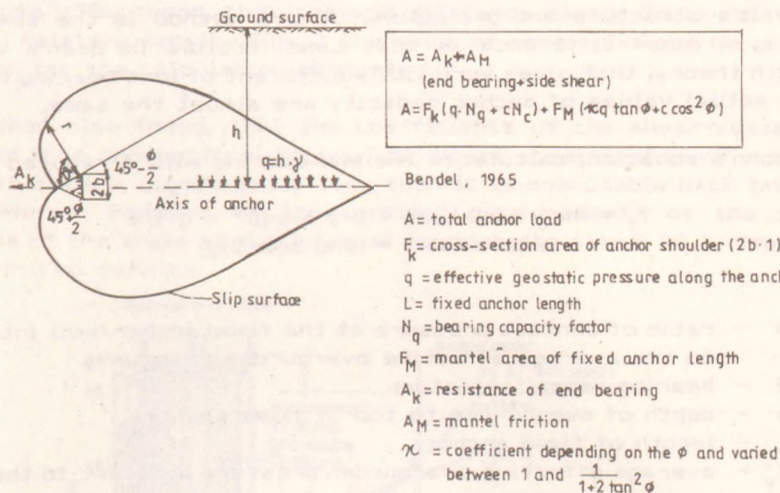


Figure 1

Size of the end bearing body around the anchor in different soils

Bendel, 1965

Jaky, 1945

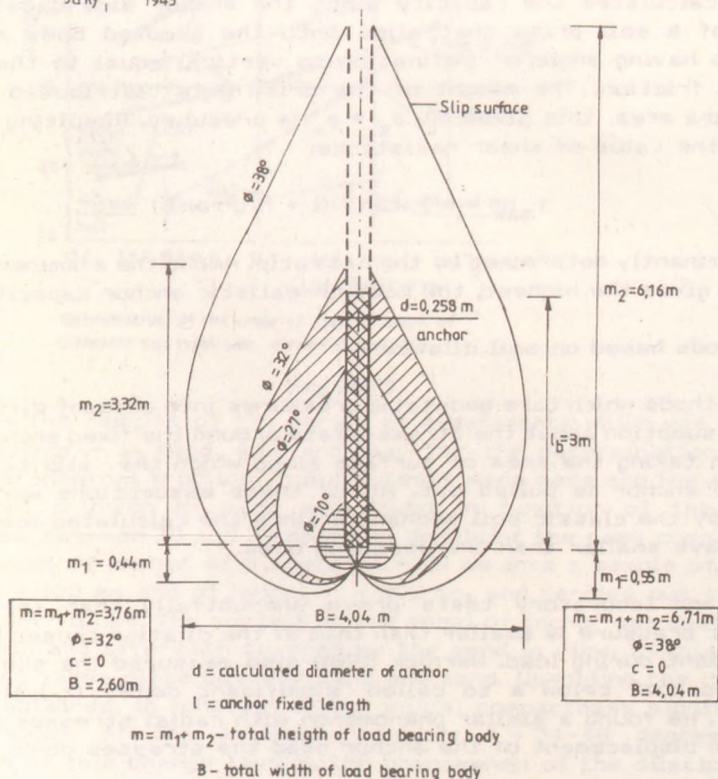


Figure 2

As for its structure and principles, Lendi's method is the same with Bendel's. A minor difference is that Lendi applies De Beer's wedging strength theory, that gives very little different of load bearing factors so the actual values of anchor capacity are almost the same.

Littlejohn's equation, calculates the end bearing with a reduced area.

$$T_f = A \cdot \sigma'_v \cdot \pi \cdot D \cdot L \cdot \tan \phi + B \cdot \gamma \cdot h \cdot \pi \cdot (D^2 - d^2)/4$$

(side shear) + (end bearing)

where

- A - ratio of contact pressure at the fixed anchor/soil interface to the average effective overburden pressure;
- B - bearing capacity factor;
- h - depth of overburden to top of fixed anchor;
- L - length of fixed anchor;
- σ'_v - average effective overburden pressure adjacent to the fixed anchor (equivalent to $(h+L/2)$ for a vertical anchor);
- D - effective diameter of fixed anchor;
- d - effective diameter of grout shaft above fixed anchor.

Jelinek calculates the capacity along the anchor side caused by the weight of a soil prism that slips onto the grouted body along slip surfaces having angle of inclination to vertical equal to the angle of internal friction. The weight of the prism gets distributed uniformly along a d'a area, this gives σ_L . $\sigma_z = \sigma_x$ is presumed. Resulting from this method, the value of shear resistance:

$$\tau_{\max} = h \cdot \gamma \cdot \tan \phi \cdot (1 + h/d \cdot \tan \phi)$$

is predominantly determined by the h/d ratio. Among the examined methods, this one gives the highest, the most unrealistic anchor capacity values.

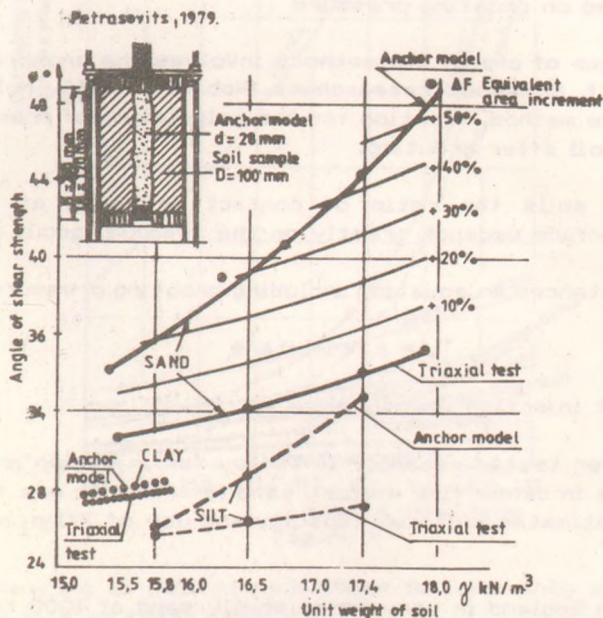
2.2 Methods based on soil dilation

Those methods which take geostatic pressures into account differ partly in the assumption about the stress-state around the fixed anchor length, partly in taking the area of surface along which the slip takes place when the anchor is pulled out. All of these assumptions were necessitated by the classic soil mechanics since the calculated load bearing were always smaller than the measured ones.

In-situ and laboratory tests prove andoubtfully that the role of geostatic pressure is smaller than that of the dilation caused by anchor displacement during load. Wernick (1981) also measured the skin friction and found that below a so called "significant depth" (z_0) it becomes constant. He found a similar phenomenon with radial stresses (σ_r): over a certain displacement of the anchor head the stresses don't increase.

Somerville (1981) found, that the soil dilatancy factor (M) is related to the soil relative density (RD) at the point of anchor failure. He gives an equation for the calculation of M , also.

The Author also found, that the coefficients of the shear-resistance obtained from conventional triaxial or direct shear tests are - in non-cohesive soils - significantly less than it is computable back from the anchor-force. Because of the confined displacements of the grains increase of the angle of the internal friction can attain 40 %, depending on the initial density.



Determination of the angle of shear strength of different soil types with different methods

Figure 3

Fig. 3 gives the results of the series of experiments which was carried - by the Author - in sand, silt and clay for the determination of the angle of the internal friction. Triaxial tests were made and the angle of the internal friction of sand was found in function of the initial compactness, between 30 and 34 degrees. Upholding the same compactness we put a model of anchor of diameter $d = 28$ mm into a sample of soil of diameter $D = 100$ mm and of height $H = 200$ mm, and carried out the test upon constant volume. The surface of the model of anchor was coated with plastic resin and afterwards covered by the sand in view. In this manner the shearing takes place between sand and sand. Upholding the constant volume we obtained, in function of the initial compactness significantly greater values of the internal friction: $\phi = 34-48$ degrees. The explanation of this change lays in the phenomenon of the dilation.

The shearing is followed by a kind of dilatation and so the horizontal stress - in sands - increases significantly. In silt this phenomenon is slightly observable, while in clay not at all, that is the methods lead to the same value of angle of the internal friction. We determined the angle of the internal friction referring to the initial diameter of the model.

Methods belonging to this group approximate much better the real phenomenon than the previous one, but special care should be taken when applied.

2.3 Methods based on grouting pressure

Decision about use of one of the methods involves the anchorage making technology itself. Numerous researchers (Robinson, 1969, Moller, 1969, Togrol, 1978) have method, equation for the calculation of pressure that remains in the soil after grouting.

In fine grained soils the ratio of contact pressure at the fixed anchor-soil interface depends greatly on the residual grout pressure.

For such circumstances an equation including grouting pressure has been used:

$$T_f = p_i \cdot \pi \cdot D \cdot L \cdot \tan \phi$$

where

p_i - grout injection pressure.

This rule has been tested recently in Turkey for injection pressure of 1000 to 2000 kPa in dense fine uniform sand ($\phi = 40^\circ$). It was found that the rule over-estimates pull-out capacity and use of $2/3 \cdot p_i$ was recommended.

Further tests in England in very dense shelly sand at 1000 to 1400 kPa injection pressure approximated the residual pressure to $1/3 \cdot p_i$.

It is considered that the in-situ permeability of the soil, the setting shrinkage of the grout and other effects make dubious the applicability of this method.

3 Conclusion and comparative analysis of the different calculation methods

In the following the results derived from the above discussed methods will be represented by means of a numerical example.

The calculated values are plotted against the soil internal friction angle in Fig. 4. Here the dimensions of the calculated anchorage also can be seen.

In case of methods that use separate end bearing and side shear calculation, we calculated and represented the results likewise and give

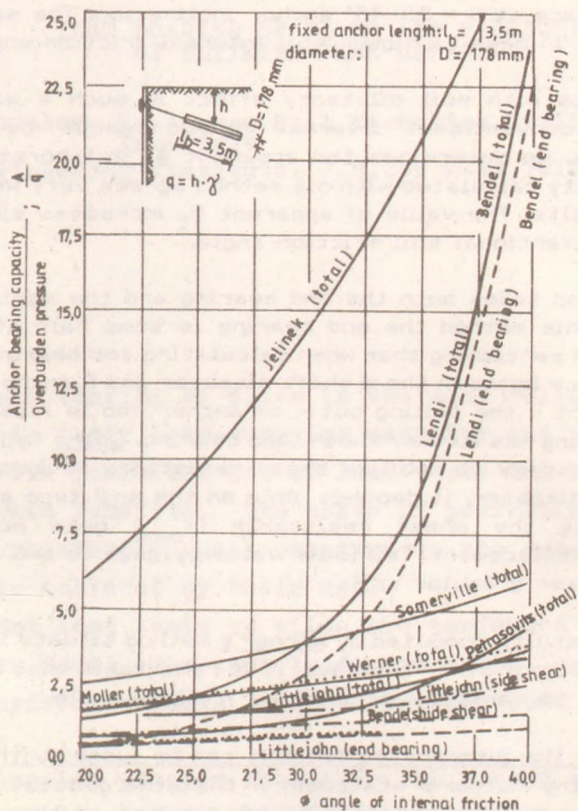


Figure 4

total load bearing as well. If side shear is taken into account only, it appears in Fig. 4 as total capacity.

The highest anchor capacities were derived from Jelinek's formula, which takes the overburden pressure with a h/d ratio, assuming that the weight of a soil prism with side angle equal to the soil internal friction.

In coarse grained soils, Bendel's and Lendi's method give high capacities, too. In these theories not only the results are suspicious, but the fact, that in opposition with the experiences, 80-90 percent of anchor capacity is resulted by end bearing in the range of internal friction angle $\phi = 32.5-40^\circ$. At lower ϕ values these theories gave actually similar total bearing capacity values with other methods.

These three methods above are very sensitive to both the angle of shear resistance and the diameter of grouted body

Somerville calculates with the wedging effect due to the soil dilatation at the soil-grout interface. The M dilatancy factor varies sharply with the anchor (β) inclination angle and the soil internal friction angle. The

minimum M values are at $\beta = 35-45^\circ$ anchor inclination. The soil dilation has bigger effect in case of higher soil internal friction angle.

The Author counts with soil dilatancy effect in such a way that he transforms the conventional internal friction angle by means of experimental data. He determines the apparent ϕ_t in laboratory tests. The anchor capacity calculated with his method agrees very well with the in-situ test results. The value of apparent ϕ_t increases significantly above $\phi = 30^\circ$ conventional soil friction angle.

Littlejohn's method takes both the end bearing and the mantle friction into account. In his method the end bearing is some half of the total capacity. It worth mentioning that when calculating end bearing, he takes the area difference between the d shaft diameter and D anchor diameter. If the displacement - the pulling out - of anchor end is less than $D/10$, then the end bearing has little role in load bearing. Cooke found that the displacement necessary to mobilize shear resistance is independent of the anchor (pile) diameter, it depends only on the soil type and density. In granular soils the shear resistance (τ_{max}) gets mobilized at $s_m = 2$ to 10 mm displacement. The lower value belongs to the denser, the higher to the looser granular soils.

Anchor capacity results computed by Werner's method situate in the upper range of the results of other theories. It is remarkable that this theory is less sensitive to variation of internal friction angle.

The reliability of the interpreted methods can be judged with the bases of the Fig. 4. If the Author's statement - that the geostatic pressure has no significant effect on capacity of grouted anchors in coarse grained soils - is accepted, the other ways should be found for the solution. Naturally, a 4-5 m overburden layer is necessary because of technological points of view. If the injection is carried out carefully, the initial stress condition gets modified and becomes steady in granular soils and the anchor gets locked in.

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TEST LOADING OF PILES AND PREDICTION OF ULTIMATE PILE CAPACITY

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1. Load testing of piles is the most reliable technique to ascertain their load-carrying capacity and to control their quality. The judicious use of load tests can lead to substantial economic benefits. The costs of performing pile load tests are relatively small compared to the substantial savings that may be achieved by their use.

2. Most load tests on piles are performed in the axial compression mode. The test loads are almost exclusively applied by hydraulic jacks, and necessitate some form of reaction setup.

The loading program specifies the method and rate according to which the load shall be applied to the pile. Among the most common procedures for compression tests are :

. Maintained load test (ML). This is the usual method of carrying out a test, especially if the load-settlement relationship is required. The procedure is to apply the load in stages, the load at each stage being maintained constant until the resulting settlement of the pile virtually ceases before applying the next increment. Evidently, the traditional ML method is rather time-consuming and, as a consequence, expensive. Consequently other loading programs which are more economical in terms of duration have been developed.

. Constant rate of penetration test (CRP) was designed to economize the test duration when only the ultimate pile capacity is sought. In carrying out the CRP test, the pile is pushed down at a typical constant rate from its position as installed, and the force necessary to produce this rate of penetration is continuously registered. As mentioned this loading

method provides no clue as to the expected settlements.

. Quick load test (QL). The CRP test calls for records of time and jacking force to be made at equal intervals of movement of the pile head, with the rate of jacking being adjusted so that the readings occur at equal intervals of time. For convenience and simplicity, the CRP test was modified to produce the QL test. It requires that loads be added in increments of 50-100 kN every 2.5 mins, until continuous jacking is required or the full capacity of the system has been reached.

. Method of equilibrium (ME) was suggested in order to combine the full load-settlement information obtained from the ML test with the time economy of the CRP test. The principle is to apply to the pile, at each stage of the test, a load slightly higher than the required value, and then to allow the load to decrease itself due to the yielding of the ground, until a state of equilibrium between the load and settlement exists. Each initial load is maintained for 5-15 mins, and is then allowed to reduce itself via downward movement of the pile. Within a few minutes a state of equilibrium is generally attained and the next increment of load then applied. The time saving may amount to two-thirds and the results show an excellent agreement with those of conventional ML tests.

3. In order to make the pile load testing a matter of routine procedure in general practice, costs should be minimized, without compromising the data obtained. To achieve both goals, and based on the information provided by many loading tests, it is recommended to use either the quick load tests method or the method of equilibrium, depending on the pile - soil system.

The QL method is limited to the determination of the pile load capacity, without providing information about the settlement. Hence, it can be used only for cases where settlement data can not be obtained from load tests (e.g., for piles resting on clayey soils). The ME test can be used when settlement data are needed and can be quickly obtained. This is the case for end bearing piles resting on sand, where settlement are needed to calculate the allowable pile load.

4. When the load-settlement curve shows a sharp break and then a well-defined failure, the ultimate pile capacity

estimation is a fairly straight forward task. In most cases the load-settlement curve changes slope so gradually that we need some rules to guide us.

The ultimate failure load is defined as the load when the pile plunges or the settlements occur rapidly under sustained load. Plunging, however, may require large movements which may exceed the loading system capacity. Other definitions of failure consider arbitrary settlement limit such as, the pile is considered to have failed when the pile head moved 10 % of the pile diameter or the failure load is at the intersection of the initial tangent to the load-movement curve and the tangent to or extension of the final portion of the curve. All these definitions are judgemental (Fellenius, 1980).

To be useful, a failure criterion must be based on some mathematical rule and generate a repeatable value that is independent of scale relations or the opinions of the individual interpreter.

5. Different authors have proposed different mathematical equations to represent the load-movement curve. This section provide a brief review of three possible analytical load-movement relationships which have been applied to different pile load tests and the results compared.

The basic idea is to use the available load-settlement data to determine the parameters defining the assumed analytical relationship and then to estimate the ultimate failure load by means of this relationship.

. In the model proposed by Van der Veen (1953) the load (P)-settlement (s) curve is assumed to be an exponential curve:

$$P = P_{cr} (1 - e^{-bs})$$

where P_{cr} is the failure load and b is an experimental parameter. An expeditive procedure is to consider the derivative of P with respect to s :

$$a = dP/ds = P_{cr} b e^{-bs}$$

and to plot $\ln a$ versus s. The straight line resulting from a linear regression of the plotted points (fig.1) yields both the ultimate load and tangent modulus of the pile.

. The method put forward by Mazurkiewicz (1972) is based on the assumption that the load-settlement curve is approxi -

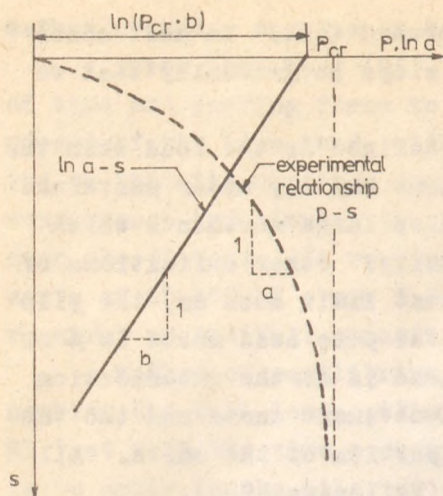


Figure 1

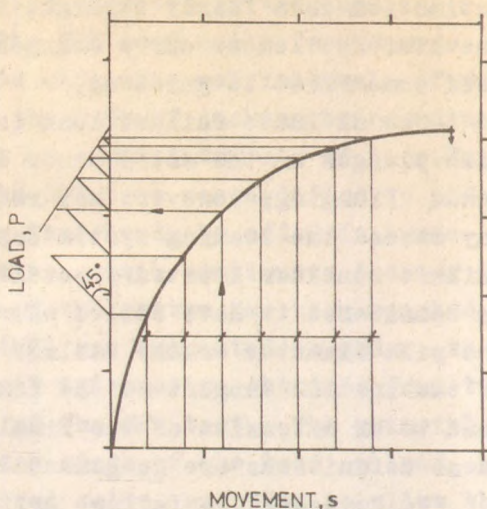


Figure 2

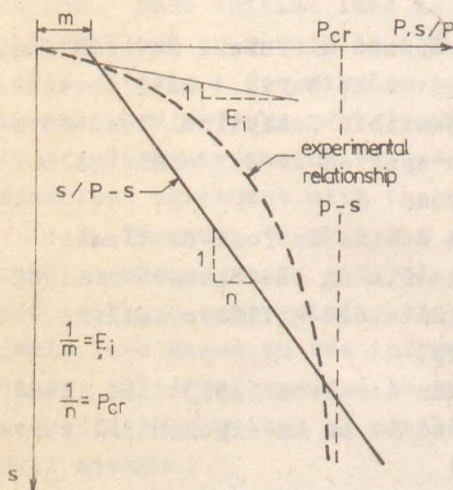


Figure 3

mately parabolic. A series of equal pile head settlement lines are arbitrarily chosen and the corresponding load lines are constructed from the intersections of the settlement lines with the load-settlement curve (fig.2). From the intersection of each load line with the load axis, a 45° line is drawn to intersect with the next load line. These intersections fall, approximately, on a straight line the intersection of which with the load axis defines the failure load.

. The method proposed by Chin (1971) assumes a load-movement curve of hyperbolic shape :

$$P = \frac{s}{m + n s}$$

If we plot s/P versus s for each loading step (fig.3), after some initial variation, the plotted values fall on a straight

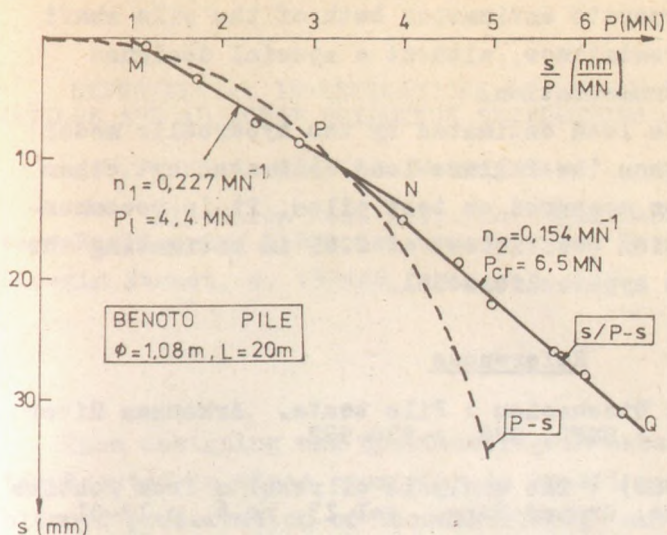


Figure 4

tests performed on bored concrete piles for bridge structures in different soil conditions (Popescu, 1988). The results have put into evidence that the hyperbolic relationship yields the highest failure load, while the exponential and parabolic relationships provide failure loads closer to each other.

The results have also shown that for end bearing piles the load-settlement curve may be represented by a single straight line in the coordinate system s/P vs. s . However for floating piles, the s/P vs. s relationship is represented by two straight lines (fig.4), the intersecting point corresponding to a full mobilization of the shaft resistance. The slope of the first line gives the shaft failure load ($P_1 = 1/n_1$), while the slope of the second supplies the total failure load, P_{cr} , which is the sum of the shaft and base failure load ($P_{cr} = P_1 + P_b = 1/n_2$).

It may be concluded that the hyperbolic model has two important advantages, namely :

- (i) allows to make an extrapolation of the load-settlement curve in order to estimate the failure load when the load-carrying capacity of the pile would exceed the equipment capacity, in which case the test cannot be carried out to failure ;

line :

$$s/P = m + n s$$

The parameters m and n have a physical meaning : $1/m$ is the tangent pile modulus, while $1/n$ is equal to the failure load of the pile.

6. The above mentioned analytical models have been applied to about twenty load

- (ii) allows separate estimation both of the pile shaft and toe resistance, without a special designed pile instrumentation.

As the ultimate load estimated by the hyperbolic model is always greater than the failure load estimated by other analytical models or measured on test piles, it is recommended to use a reduction coefficient of 0.85 in estimating the failure load by the hyperbolic model.

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EXPERIMENTAL INVESTIGATIONS OF BRACED EXCAVATIONS DEFORMATIONS AND ADJACENT BUILDINGS SETTLEMENTS CAUSED BY THEM

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When designing and constructing an excavation bracing system located in close proximity to existing buildings, the problem of preservation of those buildings should be solved. The basic reason of arising settlements of buildings when constructing an excavation in proximity to them is the bracing movement towards the excavation.

In field investigations of the static work of the braced wall (the "Serp & Molot" plant, Moscow) constructed by the "diaphragm wall" method from prefabricated vertical elements (Fig.1) at all stages of construction, bending moments, strut loads, elastic deflection curve of the wall and foundations settlement of the adjacent shop building have been measured.

To investigate the stressed-strained state of the braced wall three test elements of the bracing 14 m long and 0.5 m thick have been made. The test elements were fitted with 24 string gauges measuring stresses in the reinforcement on the height at four cross-sections and calibrated on the bending moment by the static load according to the scheme of a beam on two supports. Strut loads were measured with the aid of strain gauges with a base of 400 mm.

Observations for settlements and lateral movements of the top of the bracing as well as for the settlements of the shop building foundations were performed at ten locations on the length of the excavation. Elastic curve deflections of the braced diaphragm wall" were measured by a precision theodolite at eight sighting devices installed in the tube encased in concrete in the wall elements.

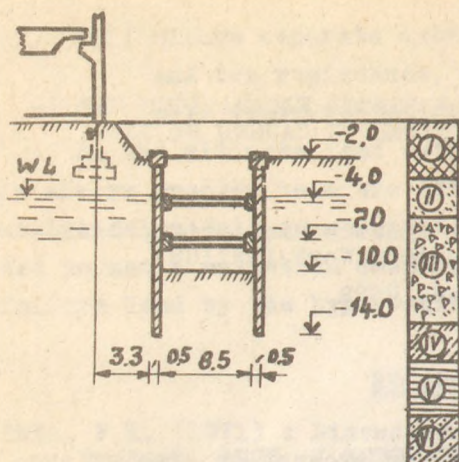


Fig. 1. The scheme of the excavation bracing:

I-filled soil; II-fine sands; III-coarse sands with gravel; IV-loams and clays; V-clays; VI-sands and loams.

Fig. 2 represents the curves of the bending moments and wall displacements constructed on the basis of the measurements and design data for the three stages of the excavation construction. At the first stage when the excavation is 5 m deep, maximum displacement of the cantilever wall top was 40.8 mm. In this case the movement of the lower part of the wall embedded in soil occurred with rotation about the wall base as an absolutely stiff one. The maximum bending moment in the wall constituted 140 kN·m.

At the second stage when the excavation was 8 m deep, the wall rotated about the support: the top of the wall moved onto the soil and the bottom moved from the soil. The lateral displacement of the top of the wall constituted 33.1 mm. Strut loads attained 217 kN/m. The bending moment in the wall at the support level constituted 160 kN·m and 120 kN·m at the span. The settlement of the foundations increased to 28.5 mm due to soil movement caused by deflections of the wall in its lower part. At the third stage when the excavation was 10 m deep the top of the wall continued to move towards the foundations and the absolute lateral displacement of the wall top constituted 29.6 mm. The inclination of the upper part of the wall onto the ground and the temperature strains of the struts caused the increase of the bending moment over the upper support (-215 kNm) and the maximum bending moment in the span of the wall consti-

a)

M, kNm -80 -160 -240 -320 160 80 0 -80 -160 160 80 0 -80 -160

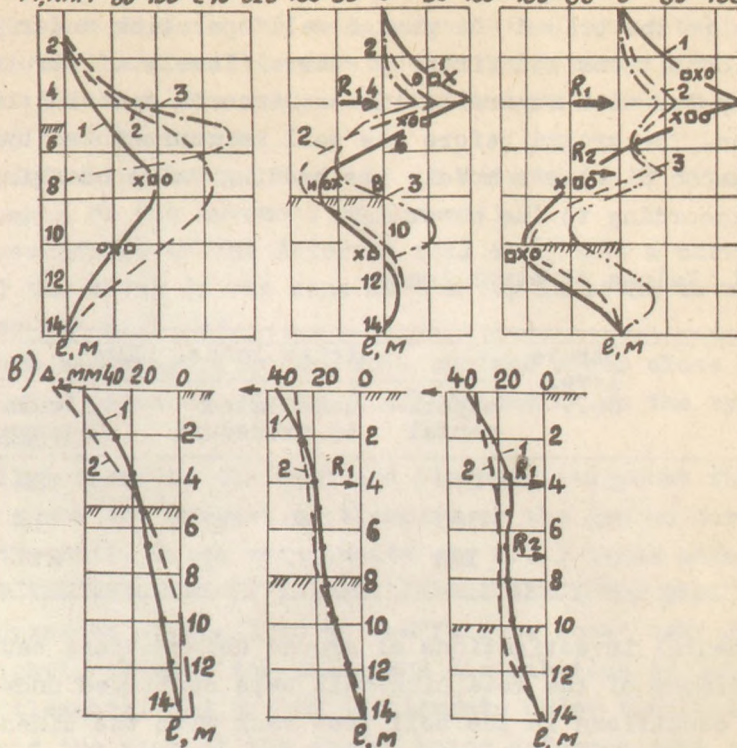


Fig. 2. The curves of bending moments (a) and movements of the wall (b) for the I-III stages of the excavation bracing construction

1-the measured ones; 2-the calculated ones (Ref.1);
3- those calculated by the Lohmeyer method;
 $\circ$; $\square$; $\times$ - indications of gauges

tuted +170 kN m. The curve of the bending moments in the wall is well coordinated with the data on measuring the "elastic deflection curve" of the wall and allows to state that in the wall at the second level of the struts the negative moment does not arise as the deflection of the wall is directed towards the excavation. The increase of the foundations settlements up to 46 mm is associated with the displacement of the bottom of the wall.

According to the procedure proposed by the authors (See Fig.2 and Table I) the comparative analysis of the wall was carried out. In it the braced "diaphragm wall" operating under plane strain conditions and fixed by several levels of struts is being computed with regard to the sequence of constructing an excavation. The ground before the wall is represented by elastic-plastic Winkler's model, its bedding value changing with depth according to the power law.

Table I. Values of strut loads

Work stage No.	Struts level No.	Struts loads, kN/m		
		experimental	calculated by procedure	calculated Lohmeyer method
II	I	217	227	158
III	I	388	160	112
III	2	348	222	216

Experimental investigations of ground deformations caused by the movement of the retaining wall were continued under laboratory conditions in the soil test tank with the dimension of 3.5x3.5x4.5 m. The walls of the tank were covered with a polyethylene film for eliminating the influence of soil friction on their surface. The model of the retaining wall represented a rigid wooden shield with the dimensions of 3.2x1.5x11 mm fixed by two levels of struts at the top and at the bottom. Sand grains were glued to the surface of the shield to create roughness. The tests were performed with the non-loaded soil surface and with a strip load of 0.30 m in width and 0.2 MPa in intensity at a distance of 0.30 and 1.25 m from the wall. In tests as soil one used fine grained sand with an internal friction angle of 30°, unit cohesion of 1 kPa, volumetric humidity of 1.5-2%, density of 1.5-1.7 g/cm³ and void ratio of 0.7.

During tests the wall was subject to different kinds of movement: rotation about the upper or lower support, parallel shear and their combination at the expense of supports dis-

placements step by step 0.5 mm each time to to a maximum value of 4.6 mm. One measured settlements and displacements of 16 deep marks embedded in the soil mass at its laying behind the retaining wall and settlements of 8 surface marks with an accuracy of 0.01 mm. The following conclusions based on the tests performed have been made:

1. Soil deforming behind the wall is determined by type, value and sequence of the movement of the wall.
2. The boundaries of the deformed soil zone have a curved outline and the width of the zone at the top is equal to the height of the wall (H).
3. Maximum settlements of the soil surface occur close to the wall or at a distance of $(0.2-0.4)H$ depending on the type of its movement.
4. Parallel shear of the wall and its rotation about the upper rib cause the largest settlements of the ground surface behind the wall. These settlements are 2...3 times more than those at the rotation of the wall about the lower rib.
5. The presence of the load at the "failure zone" behind the wall slightly affects the value and distribution of soil surface settlements. The ground settlements under the load located beyond the zone of the ground being deformed were small.
6. The finite settlement of the soil surface may be considered as the sum of settlements caused by the total wall settlements.
7. The sequence of the type of wall movement slightly affects the value and distribution of the soil surface settlements except those cases when the initial type of the wall movement is its rotation about the upper rib.

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Anchorage of diaphragm wall in Budapest

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The paper dealt with the diaphragm walling and anchorage for a three level basement construction in a builded area in Budapest. The length of diaphragm wall is 15 m, the anchor level is -2.8 m, there was used a new type of prestressed anchors. After excavation 4-5 mm of movement was measured at the anchor level. The settlement of adjacent buildings was between 1-7 mm.

The construction of deep basement in the builded area of big cities needs special supporting walling system to avoid soil movements and to ensure the stability of the adjacent buildings.

Before excavation of deep foundation pit the reinforced diaphragm walls is to be constructed. This solution very suitable if the groundwater level is high and if unpermeable - silt, clay - soil layer lies in a relatively shallow depth, about 12-15 m.

These circumstances make easier the dewatering process, because the elimination of the ground water - the dewatering of the pit easier and the lowering of the ground water table doesn't effect on the adjacent buildings.

The anchorage of diaphragm wall

To assure the practically unmovement of the diaphragm wall there are three possibilities:

- bedding it deeply into the soil mass below the designed excavation level i.g construct a cantilever beam.
- bracing
- anchoring

The cantilever beam variation needs a very deep and wide wall, the temporarily acting bedding moment and together with it the reinforcement of the wall much more high than it is after the construction of the final structure.

The braced wall is used mostly in the case of narrow pits, or if the final horizontal slabs or plates could be made advantageous from above to down it means to make the earthwork under the ready ceiling

If the pit is wide, the more often used method is the anchoring. This method is used generally if the construction of temporary anchors from technical, economical or legal point of view advantageous.

The anchoring is well proved method earlier was mostly used at sheet piling.

Anchorage

The anchorage in the last decade developed generally both from theoretical and technical point of view

The most important requirements towards the anchored structure are:

- the bearing capacity
- the stability

The bearing capacity is testing during the pre-stressing process, the method is prescribed in the various standards.

The grouted anchors is to be pre-stressed in the age 5-8 days of the last grouting. The stressing process is used for checking the motionlessness of the grouted part of the anchor, for the estimation of relaxation and also for controlling the strength requirement too. The motionlessness is assured by the friction forces acting on the surface of the grouted part of the anchors. /1/

The friction forces are depending from the shape and measures of the grouted body, and the physical properties of the soil layer in which the grouted body lies and from the overburden pressure too. The pre-stressing is to be made by steps and the extension of anchors is checking parallelly.

The stability is estimated usually by the examination of the so called deep slip line, this is based on the method of Kranz /3/ which was elaborated for anchoring of retaining wall free supporting on the soil.

The E. A. V. /2/ proposed a "calculated bottom point" for the embedded walls and this is the point below the excavated level where the shearing forces are equal to zero.

This is the model what the practice is using. The middle of the grouted part of the prestressed anchor is using as the "bottom of anchored plate."

The model described above, was criticized by Franke and Heibaum /4/ and they proposed other method for estimation of anchor length, but according to the newest opinion /5/ this proposal not acceptable.

Anchored diaphragm wall in Budapest for a three level basement garage.

Soil mechanical conditions.

On the building site earlier dwellings were placed. The shallow foundation of those were excavated at the first step of guiding wall construction. Therefore the guide walls are going down on some places to 2,0 m. Between the excavated old foundations and below of that fine sand is found to the - 3,3 m, deeper, peat was found to -6,2 m, than coarse sand to -11,3 m. Unpermeable clay layer is below of the coarse sand layer.

The ground water table lies at the depth of -3,0 m. The soil mechanical properties is to be seen on Fig 1.

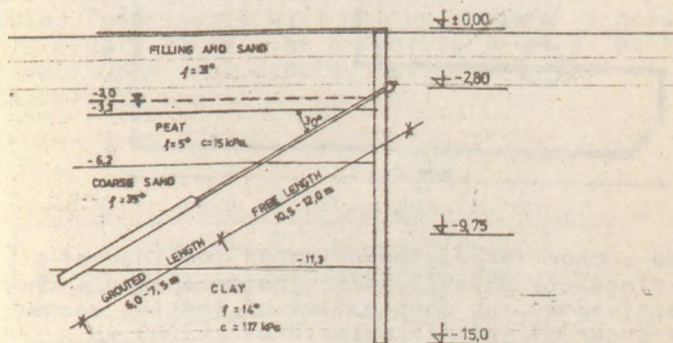


Fig 1

The diaphragm walling

The diaphragm wall is 15 m deep and their width is 0,60 m. It is constructed by the aim of bentonite slurry. The slurry was exchanged before concreting, for the better quality of the wall. The reinforced concrete diaphragm wall is made from concrete C 16. The reinforcement was made from B 60.40 quality steel bars by welding. In the welded reinforce skeleton is placed the $\varnothing$ 150 mm or $\varnothing$ 200 mm steel tubs at the anchorage level -2,80 m, apart 2-2,5 m from each other.

The anchor length was 18 m, the grouted length of the 6 strands type 6 m at the 9 strand type 7.5 m. The inclination degree is 30° . The anchors assure the practical motionlessness of the diaphragm wall /Fig 2/, and so the motionlessness of the adjacent buildings.

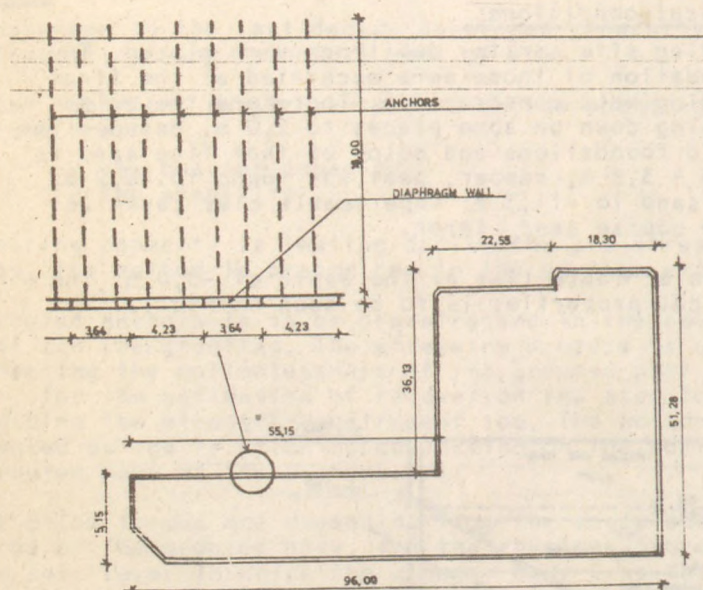


Fig. 2

The calculated anchor forces depending on the load of adjacent buildings are 350-510 kN/m. The necessary anchor forces were achieved by the combination of the two anchor types, the stronger of that is able to carry 1200 kN. The other 780 kN forces.

The diameter of the strands is 15 mm and the anchors with 9 strands has a cross section $A = 1590/\text{mm}^2$ and with 6 strands $A = 1060 \text{ mm}^2$

The anchor

The anchor is made of threaded steel strands BST 835/1030 extended with sleeve joint.

The anchor length is 18 m /Fig 3 / and consist 6 or 9 strands and one steel tube /internal dia 3/4"/ with perforated grouting holes covered by rubber valves.

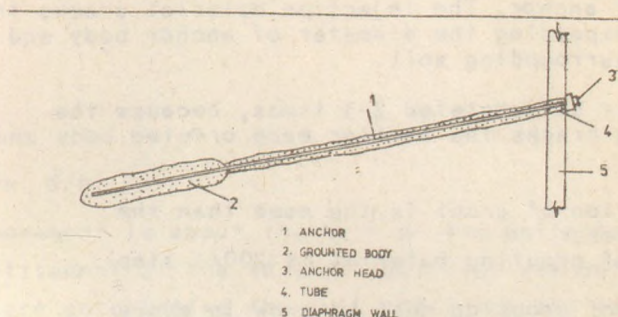


Fig 3.

The grouted part of the anchors is 6 or 7.5 m, depending of the tipe of lr.

The perforated holes are 0,50 -0,5 m apart from each other. In one cross section there are 4 holes of $\varnothing 8 \text{ mm}$ on the tube. The movable part of the anchors is covered with PVC pipe $\varnothing 32 \text{ mm}$. The extending joint is placed at the connection of the grouted and movable part of the anchors. Fig 4.

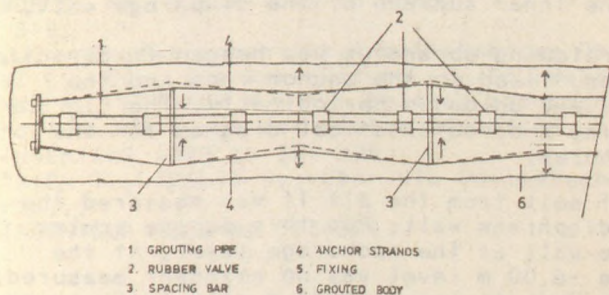


Fig. 4

Boring for the placement of anchors

The bore holes for the anchors were made by hydraulic boring machine, cement-bentonite slurry was used for this process. The slurry gets into the hole through the boring bar carries the mud to the surface and supports the boring hole. After finishing the boring the slurry was changed for cement mix. After changing the suspension the anchor is placed immediately into the hole. For the cement mix 1200 kg portland cement was used with 1000 kg water.

Grouting of the anchors

After 24-36 h strengthening of the bedding mortar begins the injection of anchor. The injection material cracks the bedding mortar, expanding the diameter of anchor body and compacting the surrounding soil.

The grouting must be completed 2-3 times, because the repeated grouting cracks the earlier made grouted body and blow it up.

The composition of grout is the same than the bedding mortar.

The volume of grouting material is 200/l step of grouting.

The pressure of grouting must be grown in every grouting step:

cracking grouting : 15-20-25 bar

fill up injection : 5-12-16 bar

The speed of grouting : 500 - 600 l/h

Stressing the anchor

There was made 151 anchors around the pit, 5-8 days after the last grouting the prestressing of the anchors was made.

The head of the anchor is made of 300x300 mm steel plate with 6x $\varnothing$ 22 mm or 9 x $\varnothing$ 22 mm holes is fitted up to the anchor rods at the inner surface of the diaphragm wall.

The hydraulic stretching apparatus was hinged in direction the axle of anchor, fixed to the anchor rods and the anchor was loaded and unloaded periodically. The Fig 5 a and 15 b is showing a characteristic diagram for the 6 and 9 rods anchors.

The excavation of soil from the pit it was measured the movement of the diaphragm wall. For this purpose scales were fixed to the wall at the anchorage level. At the excavation to the -6.00 m level was no movement measured. At the excavation down to level -9.75 m. the walls moved at the anchors level 1-5 mm. to the direction of the pit.

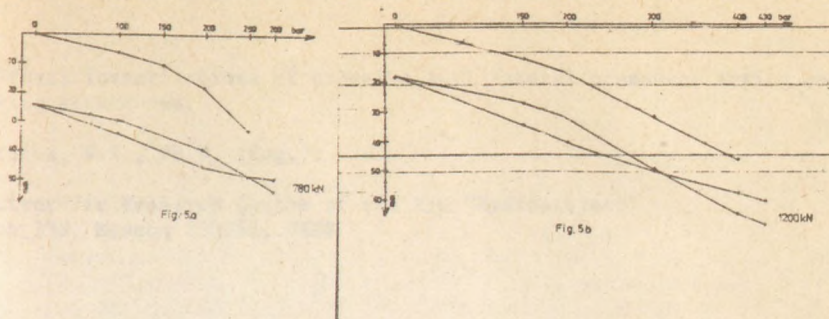


Fig 5/a 5/b

This movement is about the half of the estimated value.

The settlement of the adjacent building was between 3-7 mm.

If we are calculating the pull out force and taking into account the radii of the grouted part four times more then it is at the boring so the pullout resistance for the 6 strands anchor ~ 2700 kN and the safety factor is about $\mu = 3,50$ for the 9 strands anchors $F \sim 4400$ kN and the safety factor $\mu = 3,68$

At a trial place there were made pull out test with anchors and it had shown a similar situation

The foundation works together with the diaphragm wall was made by the KÉV-METRÓ Construcion Company and the anchorage by the Bohn Constrs. Comp. Ltd.

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Centrifugal investigations of cohesive soil lateral pressures acting on enclosing structures.

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The application of cohesive soils as a backfill for retaining walls or structures has recently become a common engineering practice. Yet a large number of retaining structures accidents has been mostly caused by cohesive soils used for fills construction. For this reason, in the new USSR Building Codes on Retaining Walls [1] the use of cohesive soils for backfills, and responsible structures especially, was abandoned. However, cohesive soils for backfills are still widely used for smaller retaining walls and less responsible structures in the absence of local sand soils; thus the problem of cohesive soils lateral pressure evaluation is a timely subject.

Present-day methods for cohesive soil lateral pressure assessment are fairly approximate and do not account for such factors as soil plasticity, swelling capacity, rheological properties etc. Experimental investigations of these soils also yield some approximate results since the development of lateral pressure, especially of cohesive soils, is mostly governed by the backfill construction technology, time-history of the backfill and deformations due to the "soil-enclosing structure" interaction.

The classical Terzaghi experiments prove that the lateral pressure coefficient of cohesive soils at rest can reach 0.75. Some later investigations revealed that for yielding clay this value can amount to 1.0, i.e. exceeding the non-cohesive lateral pressure at rest by over two times.

Considering this fact, the ultimate lateral earth pressure for the given fill and its operational conditions are of special importance. Nowadays, the influence of soil placement technology and a possible non-uniform density of the soil along the fill height can only be attained experimentally by centrifuge modelling since only this modelling provides lateral pressure development at real stresses. Generally, the chute model tests, the model being 1.5-2.0 m high, are performed at very low stresses, and the final error can alter the results both qualitatively and quantitatively. Proceeding from this assumption, the centrifuge modelling tests for determining lateral pressures acting against the walls has been widely applied in the "Hydroproject" Scientific Research Centre. This method enabled experimental studies of a sand soil pore pressure on retaining walls. The relationships between pressure variation and soil density, as well as pressure versus wall displacement were determined.

This report presents the investigation results on determining cohesive soil lateral pressures on two enclosing structures: the rigid and the mo-

vable one. The experiments were conducted in a centrifuge designed for reaching up to 320 g accelerations with a model weight nearly 250 kg. The tests were performed in a special container with a side wall which can turn around the lower support towards the soil and inversely within the assigned angle of rotation. The testing installation is fully described in [3].

In the tests, the fill of a certain retaining wall during its long-term operation was studied. However, the simulation of specific working conditions was preceded by preliminary tests to determine the cohesive soil lateral pressure on the retaining wall. For the model wall backfill, in-situ overlaying loam was used, which was placed into the sluice chamber fills. The soil compaction during placement was minimum, and the soil density in different points along the structure height and area was in the range of 16-23 %.

The tests were conducted on a backfill with uniform density, the density and moisture content of the fill varying in different tests. For loams placed with 16% moisture content and 2.05 g/cm^3 density the measured lateral pressure coefficient was 0.37. In fills with a higher moisture content and density the same coefficient amounted to 0.60. In soils with 19-20% moisture content the lateral pressure coefficient comprised 0.86. It should be noted, that in these tests pore pressures were not recorded, thus the measured pressures correspond to total pressures.

The results prove that with higher initial soil density and, consequently, plasticity the lateral pressure coefficient can increase by two or more times. Fig.1 shows the lateral pressure curve of the backfill soil with 20% moisture content. The curve is almost linear. In this case, the soil placement procedure is of comparatively little value. At the same time, the application of a less plastic soil results in a more non-uniform measured lateral pressure, and such factors as the technology of soil placement and compaction, intervals in placement and others prove to become increasingly important. This is clearly demonstrated by the measured lateral pressure curve of the soil with 16% density. The influence of the same factors was remarkable in cohesive soil backfills which gave a triangular pressure curve only for ideally uniform fills. It is undoubtedly this fact that proves the futility of modelling a real curve of the earth lateral pressure on an enclosing structure in a chute with maximum real horizontal stress of $0.2-0.3 \text{ kg/cm}^2$.

Theoretical assessment of lateral pressures provides a similar picture. If the fill soil is ideally uniform, the measured lateral pressures show good agreement with theoretical values obtained by classical methods. In case the fill is non-uniform in composition or density distribution, the theoretical and measured curves differ considerably.

Fig.2 shows two theoretical lateral pressure curves of soils with 20 and 16% moisture contents. The curves were computed using a rather perfect program based on the soil mathematical model employing a strengthening plastic flow theory. For plastic soils, the agreement between measured and calculated curves is practically ideal, whereas for less plastic soils they differ considerably. It should be stressed that in both cases the soil initial characteristics were determined in triaxial compression tests.

The testing machine used in the experiments allowed the investigation of the earth lateral pressure variations at small rotations of the wall around the lower support towards the soil and back. Maximum model dis-

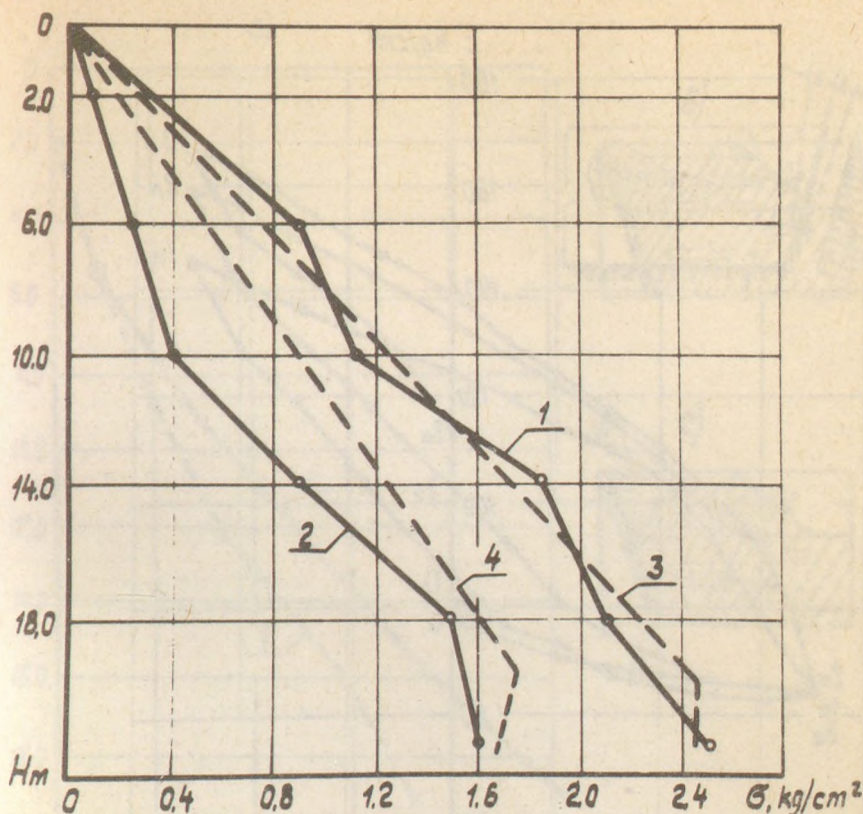


Fig.1. Curves of cohesive soil lateral pressures on the wall.
 1 - measures for the soil with 20% moisture content at placement;
 2 - measured for the soil with 16% moisture content at placement;
 3, 4 - calculated, for the soil moisture contents of 20 and 16 %.

placements in both directions were about 0.5 - 0.7 mm, corresponding to 1/500 and 1/750 of H (where H is the wall height). The results of the tests are presented in Fig.2. The test included three cycles of displacements towards the soil and back. The pressure at rest grew with each cycle after the wall assumed its initial position.

The active earth pressure occurred at wall displacements equal to about 1/750 of H . The minimum active pressure was maintained constant in all the three cycles. The initial pressure at rest was some 25% above the active pressure. It is noteworthy, that when the wall moves off the soil and resumes its initial position afterwards the lateral pressure never reaches the pressure at rest, and the reactive pressure develops exceeding the pressure at rest by over two times. The reactive pressure increases constantly with maximum displacements in each displacement cycle.

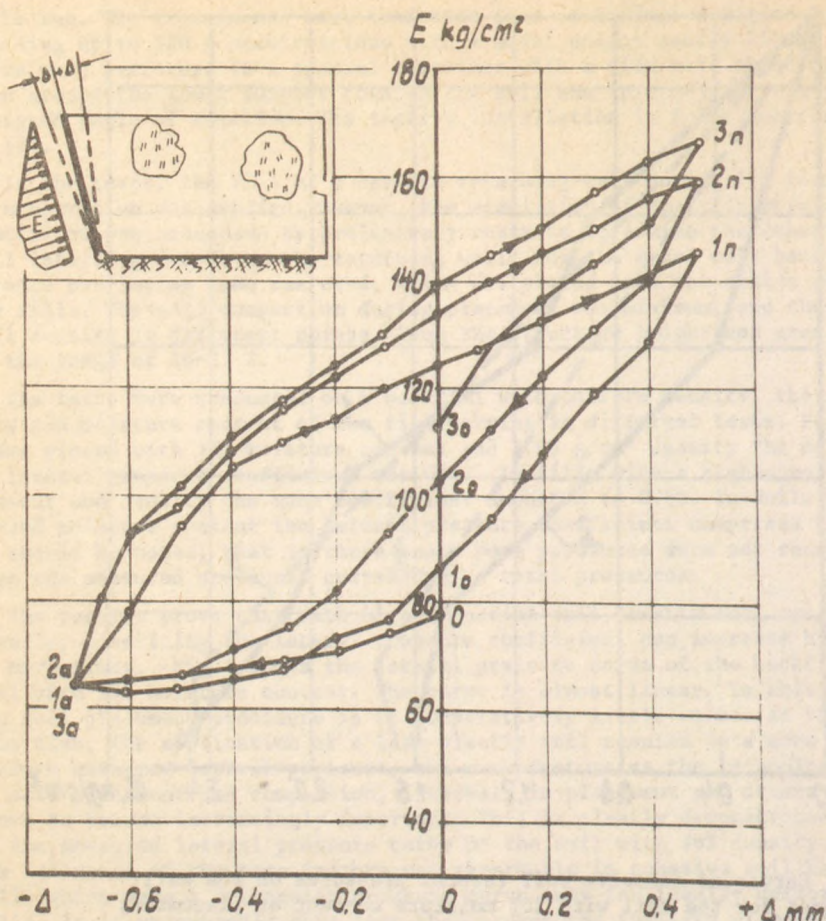


Fig.2. The diagram of earth lateral pressure variation at cyclic displacements towards the soil and back.

E - total lateral pressure; Δ - displacement of the wall top off the fill (-) and onto the fill (+), mm; 1, 2, 3 - the cycle number; 1a - active pressure; 1n - passive pressure; 1o - pressure at rest.

The obtained results are of significant value for retaining and enclosing structures subject to rotational-and-translational deformations. In this case, the cohesive soil lateral pressure can be growing over a long period of time, and the adoption of pressures based on the wall displacements alone for the rated pressure values will be most inappropriate.

In the tests where the earth lateral pressure acting on a specific wall was determined, the real fill properties were simulated. The model was based on the results of engineering and geological investigations and was non-uniform with inclusions of lenses and intermediate sand

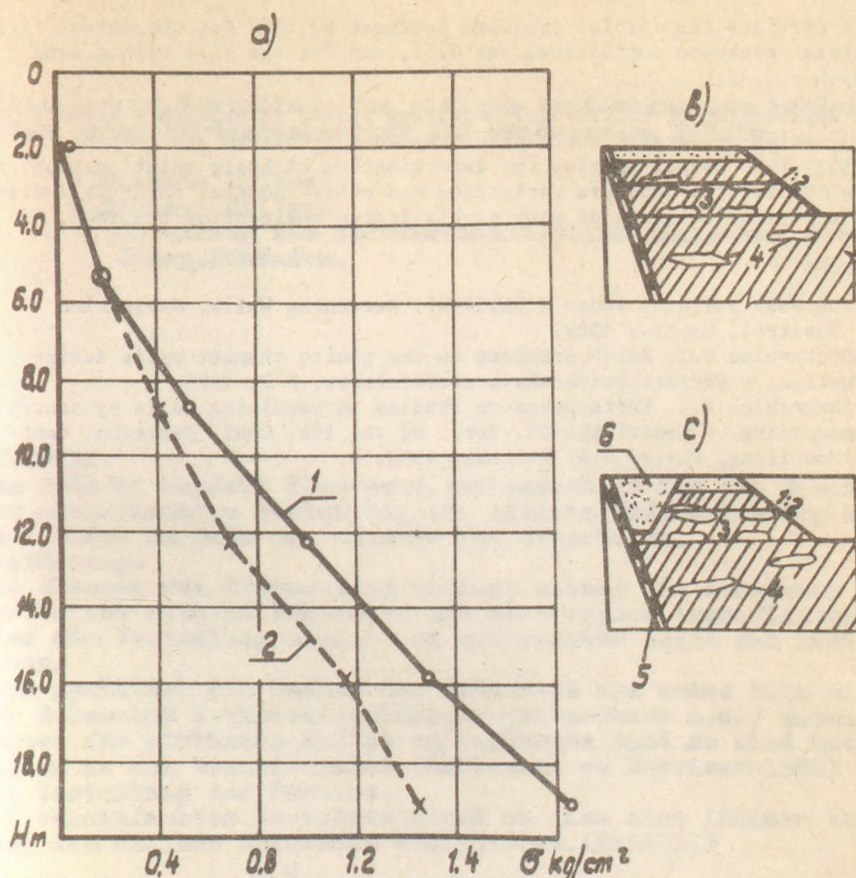


Fig.3. Measured earth pressure on the actual fill model.
a) - pressure curves; b) - model fill (pressure curve 1); c) - model fill with sand (Pressure curve 2); 3 - loam fill; 4 - sand lenses; 5 - pressure transducers; 6 - sand lenses.

layers. In the tests, major lenses and intermediate sand layers were simulated, as well as the zones of varying soil density.

The model test results show that the coefficient of lateral pressure on the sluice chamber wall for this soil is 0.56. However, considering the cyclic nature of rotational-and-translational deformation mode of the sluice chamber wall, it is only natural that during the long-term operation of the sluice this pressure will increase still further. It was implicitly proved by the fact that over the whole operation period the sluice chamber walls showed a consistent tendency of moving towards the chamber which could have only been caused by a steadily increasing lateral pressure. A method for partial decrease of lateral pressure was suggested consisting in replacing some loam immediately adjacent to the wall with the sand. The experiments with this model fill showed that the given

measure provides the initial pressure decrease by 25%. For the actual fill the lateral pressure coefficient was 0.56, and for the fill with a sand inclusion - 0.42.

The application of centrifugal modelling proved to be highly effective for evaluation of lateral earth pressures in non-uniform and combined backfills. This method enables the determination of basic relationships between the lateral pressure variations and other factors, which is indispensable for the solution of many sophisticated engineering problems.

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Nichtlineare Verteilung des aktiven Erddruckes von kohäsiven Böden unter Berücksichtigung der seismischen Kräfte.

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SYNOPSIS.

Der Beitrag handelt über eine Rechenmethode für die Ermittlung der nichtlinearen Verteilung des aktiven Erddruckes für kohäsive Böden im Fall von seismischen Ansprüchen.

Einführung.

Die Planung von Stützmauern bedingt sowohl die Kenntnis des Wertes des aktiven Erddruckes wie auch seinen Angriffspunkt oder die Verteilung längs der senkrechten Seite der Stützmauer.

Von gewöhnlich die Größe des Erddruckes und seine Lage mittels der bekannten Methoden (Coulomb, Mononobe-Okabe u.a.) nicht schwer für statische Kräfte zu bestimmen sind so sind Uneinigkeiten in der Fachliteratur (Schlosser et Darvieux-1986) über die Verteilung des Druckes.

Ein vereinfachtes Verfahren nimmt an dass eine lineare Verteilung der aktiven Erddrücke stattfindet. (Bild 1.)

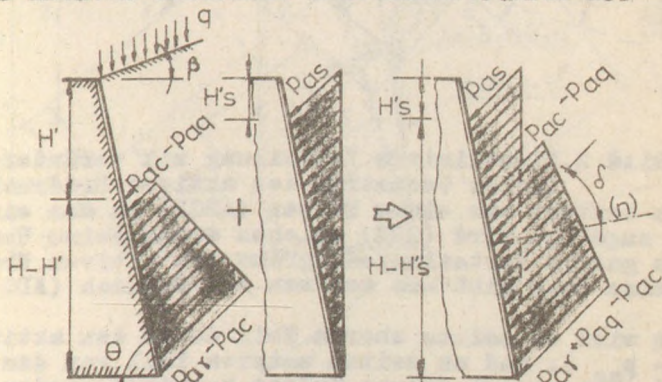
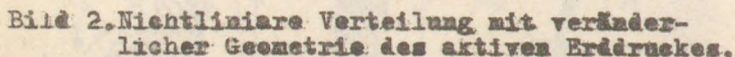


Bild 1. Verteilung der aktiven Erddrücke in Fall einer seismischen Beanspruchung.

Die Angriffspunkte werden auf Grund der Schwerpunkte der Druckdiagramme bestimmt.

Die lineare Verteilung wird aber nicht durch Versuchsergebnisse bestätigt (Dubrova-1963, Tagarelli-1969, Levilain-1973) und es folgt die Unmöglichkeit der Erfüllung der statischen Gleichgewichtsbedingungen für das Erdprisma.

1 Bild 2).



Das Diagramm wird an seinem oberen Teil durch den aktiven Erd-
druck ($p_{aq} = p_{ao}$), und an seinem unteren Teil von dem Druck
($p_b = p + d$) begrenzt, dessen Grösse bestimmt werden muss.
Die Werte von f und d erhält man durch die Diskretisierung
des Diagramms, in einzelne Komponente die in Druckeinheiten
ausgedrückt werden.

Drücke:

$$p_a(z) = A(z \cdot \cos \delta / \sin \theta)^2 - B(z \cdot \cos \delta / \sin \theta) + (p_{aq} - p_{ag}) + z \cdot \sin \delta / \sin \theta \quad (1)$$

Die Parameter der Parabel (A, B) die das Diagramm und den aktiven Erddruck am unteren Rande begrenzen werden durch folgende Bedingungen ermittelt:

- für $y=a$ $x=p$ $A = p/a^2$ (2a)

- Die Fläche des Diagramms (ABDC'E) muss gleich sein der Grösse des aktiven Erddrucks

$$A = P \quad (2b)$$

- Die Grösse der Ordinate des Schweregewichtspunktes des Diagramms der aktiven Drücke muss gleich sein der Projektion des Hebelarmes des aktiven Drucks zum Punkt B. (Bild 2).

$$Y_G = a \cdot \cos \delta \quad \text{oder} \quad Y'_G = Y'_G \cdot \cos \delta / \sin \theta \quad (2c)$$

Auf Grund der Bedingungen (2b) und (2c) erhält man:

$$P = 0,5 \cdot a(2d-f) + a(2p-3B \cdot a)/6 \quad (3a)$$

und

$$Y_G = [a^2(3d-2f)/6 + (p \cdot a^2 - 2B \cdot a^3)/12] / P \quad (3b)$$

Durch das Lösen der Gleichungen (3a) und (3b) bezüglich der Unbekannten B und p ergeben sich die Werte:

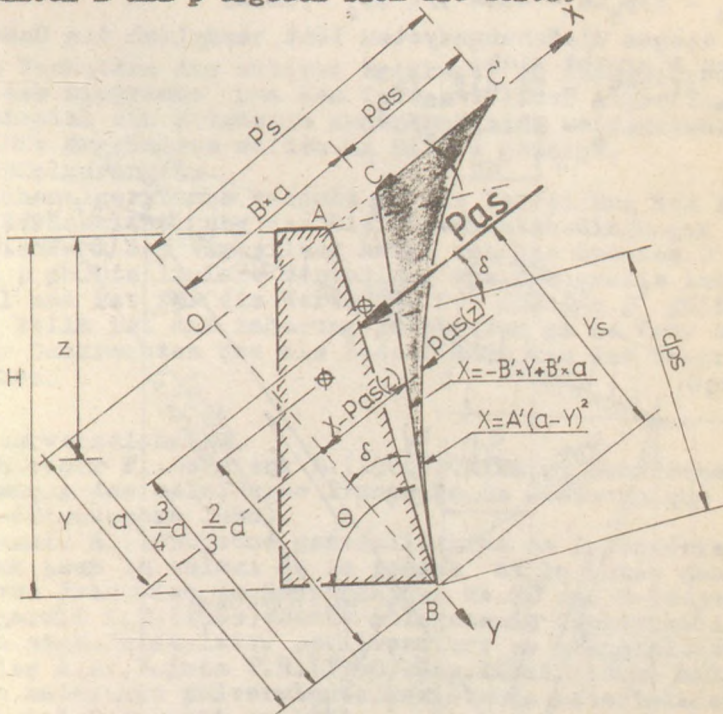


Bild 3. Verteilung der zusätzlichen aktiven Erddrücke durch einen seismischen Anspruch.

$$p = \frac{6a^2(d-f) + 12P(a-3Y_G)}{a^2} \quad (4a)$$

$$B = \frac{a^2(6d-f) + 6(p \cdot a - 4Y_G)}{a^3} \quad (4b)$$

Im Fall eines seismischen Anspruches wird folgende Verteilung vorgeschlagen für den zusätzlichen Erddruck der durch einen seismischen Anspruch ausgelöst wird. (gemäss Bild 3). Dieser zusätzliche Erddruck ist die Differenz zwischen der Fläche eines parabelischen Dreiecks (OBC'') und dem Dreieck (OCB) mit seinem grössten Wert p_{as} (beziehungsweise p_s') der unbekannt ist.

$$P_{as}(z) = A'(a - z \cdot \cos \delta / \sin \theta)^2 + B'(z \cdot \cos \delta / \sin \theta) - B'a \quad (5)$$
$$P_{aa} = (2 \cdot p'_{aa} - 3E \cdot a^2) / 6 \quad (6a)$$

$$Y_{\pm} = (3P_{\pm}^{\prime}a - 4B_{\pm}^{\prime}a^2)/(4P_{\pm}^{\prime} - 6B_{\pm}^{\prime}a) \quad (6b)$$

$$P' = (36Y_a - 24.a)P_{aa}/a^2 \quad (7a)$$

$$B_c = (24Y_c - 18.8)P_c/a^3 \quad (7b)$$

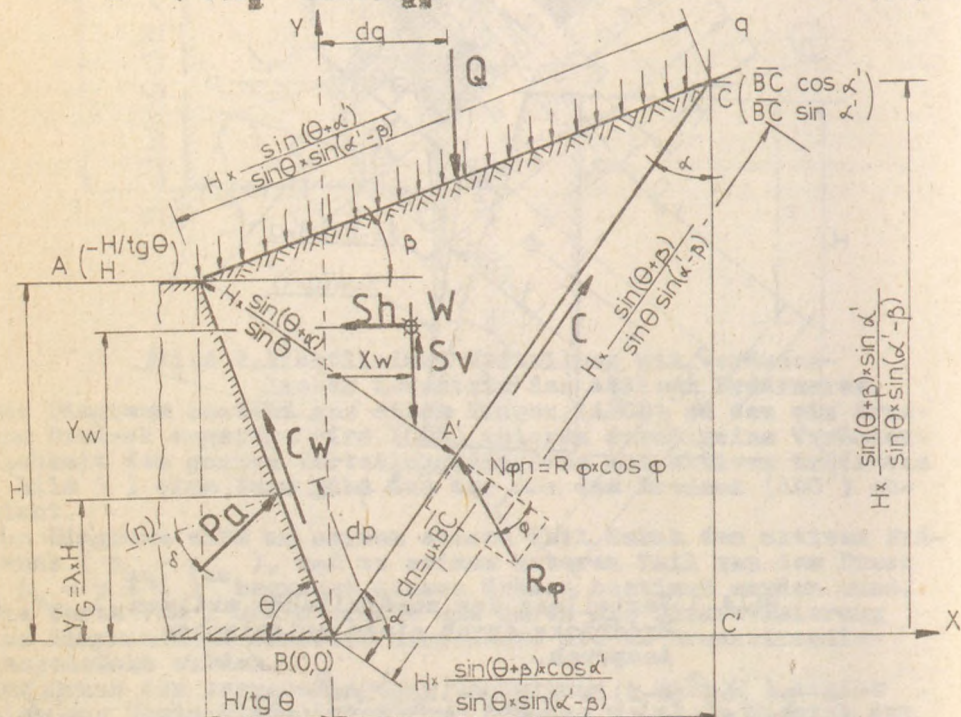


Bild 4. Berechnung des Hebelarmes des Erddrucks.

Wenn man die Werte der Parameter p' , A' und B' kennt kann man die Verteilung der aktiven Erddrücke einzeichnen. Die Grössen Y'_G und Y'_P , welche die Lage der aktiven Erddrücke P und P' bestimmen (Bild 2 und 3), werden aus der Momentengleichung aller Kräfte die auf das Erdprisma einwirken bezüglich zu Punkt B ermittelt. (Bild 4) (Stanciu 1990). Sie werden in den Gleichungen (2) und (6) benötigt.

Die Grösse Y'_G ist gleich mit $\lambda \cdot H$ in welcher

$$\lambda = \frac{\sin(\theta + \beta)}{\sin \cdot \cos \delta \cdot \sin(\alpha' - \beta)} \left\{ \frac{1 - K_{av}}{3 \cdot K_a} \cdot \frac{\sin(\theta + \beta) \sin(\alpha' + \theta)}{\sin^2 \theta \cdot \cos \theta_0} \cdot \right.$$

$$\cdot \left[\frac{\cos(\theta - \theta_0)}{\sin(\theta + \beta)} - \frac{\cos(\alpha' + \theta_0)}{\sin(\alpha' - \beta)} \right] + \mu \frac{\cos \phi}{\cos[\alpha' - (\phi - \theta_0)]} \cdot$$

$$\cdot \left[\frac{a_4}{K_a} \left(\frac{t \cdot b + e}{t - d - f} - 23 \cdot a_1 \right) - \cos[\theta - (\delta + \theta_0)] \right] \Bigg\} \text{ für } q=0$$

Um das Verhalten des aktiven Erddrucks zu analysieren auf Grund der Diagramme und des festgestellten Algorithmus wurde als Beispiel ein Erdmassiv angenommen das keine Kohäsion besitzt. Die Ergebnisse werden in Bild 5 gezeigt.

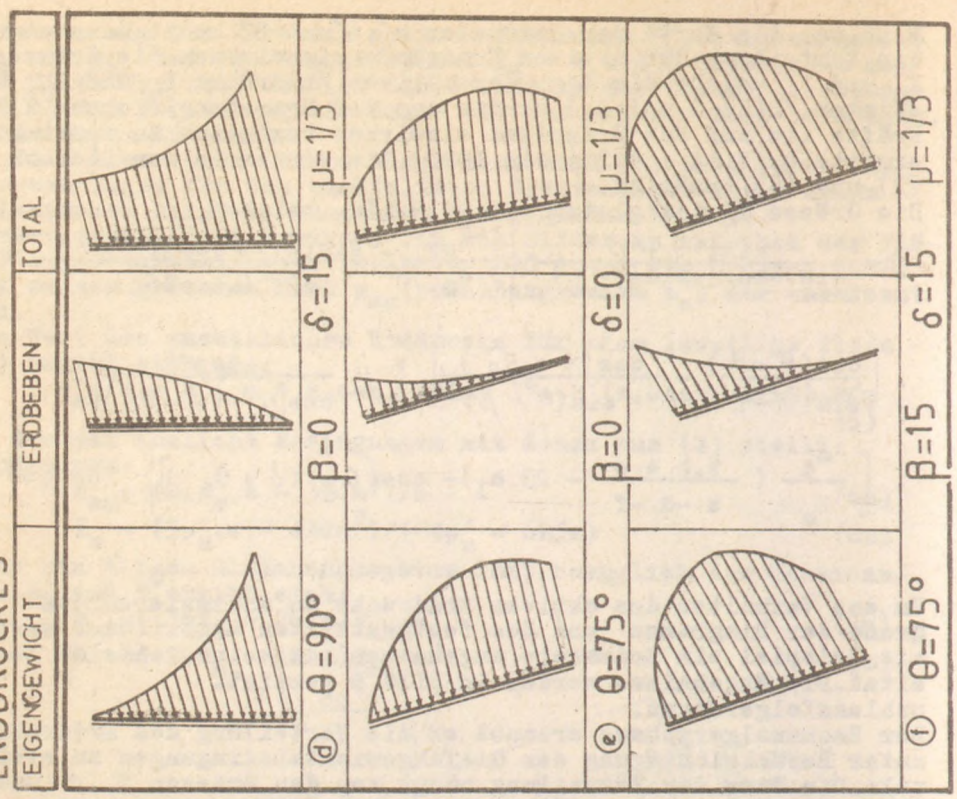
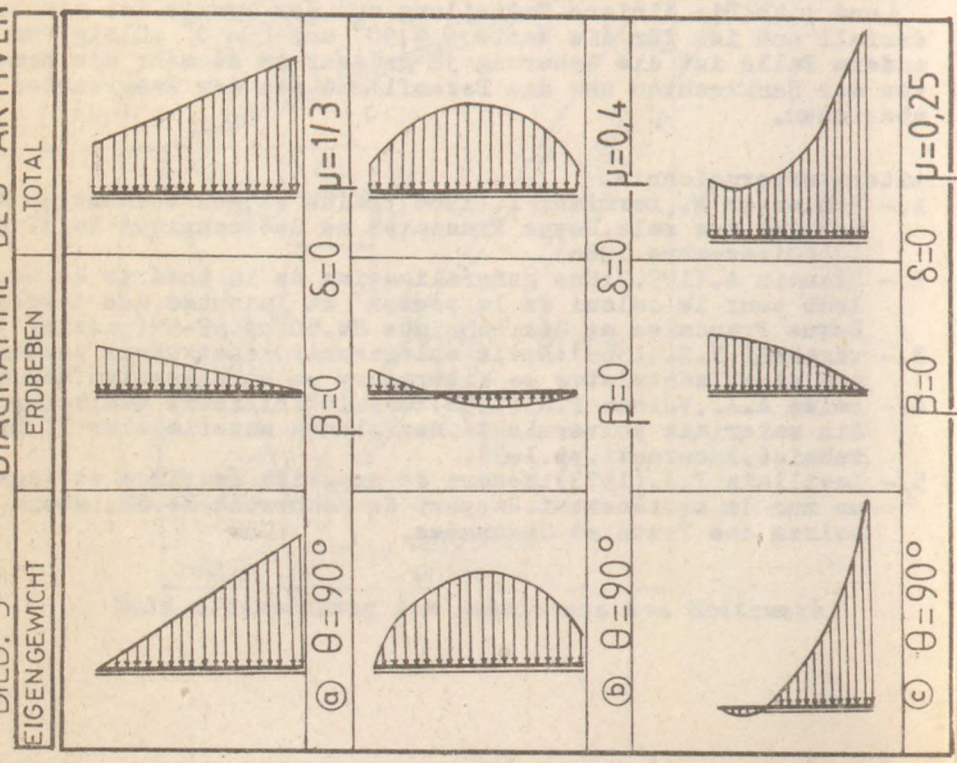
Schlussfolgerungen.

Der Rechenalgorithmus erlaubt es die Verteilung des Erddrucks unter Berücksichtigung der Gleichgewichtsbedingungen zu ermitteln. Die Form der Verteilung hängt von den Grössen θ , β und δ und μ ab. Die liniare Verteilung aus der Praxis ist ein Sonderfall und ist für die Werte $\theta = 90^\circ$ und $\beta = 0^\circ$ gültig. Für andere Fälle ist die Näherung je grösser um so mehr die Mauer von der Senkrechten und die Bodenfläche von der Waagrechten abweichen.

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BILD. 5 DIAGRAMME DES AKTIVEN ERDRUCKES



INFLUENCE OF DEEP PITS UPON UNDERGROUND WATER

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INTRODUCTION

Various ideas about the permeability of the quarternary and neogene subsoil have occurred during the preparation phase of the underground construction in Bratislava. A high underground water level in the gravel-sand layers at the right side of the Danube which corresponds to that of the Danube ought to be considered for an unfavourable circumstance.

Foundation of deep pits belongs to the pretentious geotechnic tasks. It is connected with problems of stability and dry working places securing. Engineering-geological investigation of the area must therefore be concentrated upon finding out parameters of the underground water movement. This contribution shows the possibilities of filtration characteristics determination by means of untraditional methods.

BASIC PRINCIPLES OF THE TRACER METHOD

Parameters of the underground water movement were determined by one bore-hole tracer methods based on the principle of tracer dilution and tracer vertical movement observations. The measurements were performed in the hydrogeological observation boreholes with shot perforated sections catching important horizons of the subsoil throughout the depth of 26 metres. There were also built-in some boreholes whose longer perforated sections were connecting several water-bearing layers.

Evaluation was concentrated upon computation of filtration velocities v_f in the individual depth levels. Being used the dilution method, computation was done according to the relation

$$v_f = \frac{\pi d}{4 \alpha t} (\ln c_0 - \ln c) \quad /1/$$

or at vertical movement observation it was done according to the relation

$$v_f = \frac{\Delta q_v}{\bar{\alpha} d \Delta h} \quad /2/$$

where d is internal diameter of the observed borehole,

c_0 - initial tracer's concentration,

c - concentration in time t ,

$\alpha, \bar{\alpha}$ - coefficients of drainage influence of the borehole,

Δq_v - increase or decrease of the vertical flow in the section of the height Δh .

By means of relations /1/ and /2/ filtration velocities were determined in the convenient depth intervals. The average value for the aquaous permeable part of the bore was determined from the equation

$$\bar{v}_f = \frac{\sum v_{fi} h_i}{\sum \Delta h_i} \quad /3/$$

CALCULATION OF FILTRATION COEFFICIENT

The filtration coefficient can be calculated from the found out filtration velocities at the known level gradient. It is supposed that the conditions of the Darcy's law are fulfilled

$$k = \frac{v_f}{i} \quad /4/$$

By evaluation of field observation of levels carried out many years in the form of hydroisohypses the data about level gradients at the maximum, average and minimum states of the Danube (Fig.1) were obtained. The gradient considerably reacted at the water levels of the river. During the period of minimum water states the subsoil was filled up by the infiltration from the river to which the steepest gradient ($i = 0.0125$ till 0.033) corresponded. A high gradient in the explored area was achieved also at the minimum water states when the surroundings were dained by the river ($i = 0.00625$ till 0.018). The average water states ($i = 0.00357$ till 0.00625) corresponded to the measured period. The values of the gradient achieved at the maximum water states were 3 till 4 times higher as those at the average states.

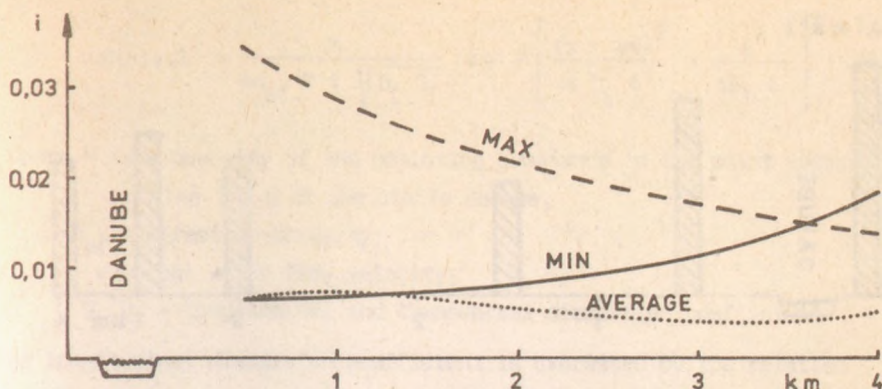


Fig.1 Gradients of the underground water near Danube

The filtration coefficient calculated according to the relation /4/ did not achieved a considerable dispersion variance. In the locality of the permeable quarry positions it range was $k_K = 4.16 \cdot 10^{-4}$ till $7.89 \cdot 10^{-4} \text{ ms}^{-1}$. Thickness of the quarter increased from 9 to 18 metres with the growing distance from the Danube. Lower laying neogene layers consisting of prevaillingly fine corn sand positions showed to be less permeable according to expectations. In the observation boreholes in the distance of 400 metres from the Danube there were located two positions of various permeability in the quarter: the first one into the depth of 13 metres $k_K = 2.24 \cdot 10^{-4} \text{ ms}^{-1}$, in the section of 13 till 18 metres $k_K = 4 \cdot 10^{-4}$ till $8.32 \cdot 10^{-4} \text{ ms}^{-1}$. Several times there was stated a higher permeability of the quarter near boundary with the neogen and it was in connection with sedimentating of a more coarse grained fraction. The neogen in the vicinity of the Danube was more permeable as in the other measured areas. In Fig.2, there are illustrated the average values of filtration coefficient in the quarter positions in dependence on the distance from the Danube in the area of underground construction.

Besides of tracer methods, the permeability of subsoil was determined on the based of pumping tests. The obtained results characterize the average values valid for the whole aquaous horizont. Tracer methods made possible to indicate heterogeneity of the medium which is important at deep pits foundation. Through confrontation of enough information was obtained for a more objective judgment of flow parameters.

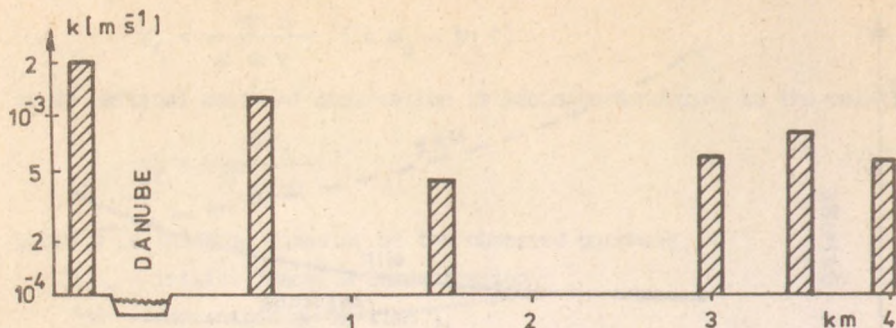


Fig.2 The average filtration coefficient of the area

WATER SOURCE POLLUTION

A high concentration of building machines as well as the use of chemical substances at deep pits foundation are potential sources of underground water contamination. In this case, it must be paid a special attention to this problem. In distance 4.6 kilometres from the concentrated construction, there is located a water source which has a considerable share upon supplying the town of Bratislava with drinking water. Hydrogeological research indicated that the gravel sediments are relatively homogeneous and the water flows into pumping walls relatively equally from the depth of 16 till 61 metres.

The rate of underground water pollution is influenced also by the kind of substance which reacts with water in various ways. So, e.g. at a single escape of crude oil substances on the terrain surface in an unsaturated zone of the observed geological conditions there can be caught 30 till 120 lm^{-3} of the pollutants. This quantity decreases through repeated escapes.

There are many factors influencing contaminated water movement. The most important of them are: hydrodynamic dispersion, adsorption, diffusion and half-life. The solution of practical tasks follows in such a way that the disturbing factors are neglected and according to the character of the flow it is considered only the dominated phenomenon. Migration in twodimensional condition with excluding of adsorption, diffusion and half-life can be analytically expressed according to Bear by the equation

$$c(x,y,t) = \frac{M}{4n_{ef} \pi t \sqrt{D_L D_T}} \exp - \left[\frac{(x - vt)^2}{4 D_L t} + \frac{y^2}{4 D_T t} \right] \quad /5/$$

where M is a quantity of the polluting substance in the point source in time $t = 0$ at the single escape,

n_{ef} - effective porosity,

v - real water flow velocity,

D_L, D_T - longitudinal and transversal dispersion coefficient.

The longitudinal dispersion coefficient is expressed by the relation

$$D_L = (0.001 + 4 d_{50} + 0.0012 \cdot 10^{0.0874 C_u}) v \quad /6/$$

where $C_u = d_{60}/d_{10}$ is a number of grane size heterogeneity. The equations /5/ and /6/ can be applicated upon the forecast of water source pollution near Bratislava. The following characteristics of the quartery soils are important for the calculation: $n_{ef} = 24\%$, $v_f = 8 \cdot 10^{-6} \text{ ms}^{-1}$, $v = 3.3 \cdot 10^{-5} \text{ ms}^{-1}$, $d_{50} = 35 \text{ mm}$, $C_u = 15$. The distance of 4.6 kilometres will be overcome by the polluted water within the period of 4.4 years. Along this distance at the dispersion coefficients ratio $D_L/D_T = 25$, there will be only 0.95% from the original pollution in the streamline. Fluctuation of water levels and the change in the gradient resulting from it will cause changes in flow directions and a prolongation of the seepage line. It will lead to a further dillution. In the case of a waste pollution in the building site there will be enough time for taking necessary measures to protect the water source from negative effects of the polluted water.

CONCLUSION

Results of field measurements serve to precisising information about the permeability of positions in the area of planned underground construction in Bratislava. On their basis the optimat protection of deep pits can be chosen which will lead to a more economical project solution. An intensive building activity can have negative influence upon the water source quality. By means of an analysis of flow characteristics and dispersion processes it can by determined the maximum pollution which will not overpass the admissable values determined for drinking water.

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EFFECT OF SOIL NONLINEARITY ON INTERACTIVE BEHAVIOUR OF UNSYMMETRICALLY LOADED PLANE FRAMES

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ABSTRACT

Study of soil-structure interaction effect in frame structures necessitates proper physical and constitutive modelling of soil mass and the structure. So far, the studies have been conducted considering the linear elastic behaviour of soil. This paper deals with an improved physical and constitutive modelling and its application to study the behaviour of an unsymmetrically loaded five storey two bay plane frame. The behaviour so obtained has been compared with that obtained from noninteractive and linear interactive analyses.

INTRODUCTION

Many investigations have so far been carried out concerning some aspects of interaction of frame structures with soil-foundation system. The modelling of any soil-structure interaction problem consists of both the physical and the constitutive modelling. The earlier studies treated soil as a winkler medium. The subsequent work considered soil as a linear elastic half space. The earlier studies included those by Grasshoff (1957), Heil (1969) and Lee et al (1970). Relatively recent studies include -

Haddadin (1971) proposed a two dimensional sub-structure approach for investigating the effect of interaction of frames with combined footings and rafts. The method accounts, in the form of iterations, for no-tension at the base of combined footing. Lee and Brown (1972) presented interaction analysis of seven storey three bay frame using both the Winkler and the half space models for soil. Whereas Winkler analysis predicts an increase in inner column load to a maximum value, linear elastic interaction analysis predicts that maximum inner column load is reduced and this maximum occurs at a much higher flexibility than the maximum for the Winkler model. King et al (1977) presented the solution for two-dimensional rafted frame assumed to be deforming into plane strain. It was clearly indicated that Winkler model did not give proper

results as also it is not realistic to consider a rafted structure as an equivalent stiffened raft for computation of different settlement. It was also found that the rough raft base did not have any significant effect on the behaviour of a vertically loaded structure. Brown (1986) examined the effect of the sequence of construction on the interactive behaviour of plane and space frames and found that the effective stiffness of a building, which is loaded progressively during construction is about half the stiffness of the complete building.

PROBLEM

On basis of the literature review, it was also realised that various authors have represented the structure-foundation-soil system using a fully finite element formulation and considered linear elastic behaviour of the soil mass. As it is the soil which is really responsible in altering the behaviour of the super-structure, it would be appropriate to consider more realistic behaviour of soil mass which is essentially nonlinear. In the present study, therefore, an attempt is made to improve upon both the physical and constitutive modelling of soil-structure interaction problem. Constitutive modelling has been improved by using hyperbolic stress-strain model for soil and physical representation by using a coupled finite-infinite, multielement formulation.

MODELLING

In order to improve the physical modelling, a new stiffness matrix formulation (Viladkar et al., 1990a) of 3-noded beam bending element for frame members and combined footing is adopted. This formulation considers the effect of both transverse shear deformations and axial-flexural interaction. The supporting soil medium is represented by eight noded curved finite elements coupled to six noded mapped infinite elements having $1/r$ type decay (Godbole et al, 1990b). The soil mass is treated as in a plane strain state. The interface characteristics between soil and foundation beam are represented by a specially developed interface element (Viladkar et al, 1990a) which is compatible with the beam element of the combined footing.

The nonlinearity of the soil mass is considered through the hyperbolic stress-strain model (Duncan and Chang, 1970) in which the tangent modulus of soil at any stress level is expressed as a function of the initial tangent modulus, failure ratio and strength mobilisation factor. The formulation developed for the purpose was used to study in detail the interactive behaviour of an unsymmetrically loaded five storey plane frame-combined footing-soil system.

Figure 1 shows the stress-strain relationship of sand obtained on basis of laboratory triaxial tests obtained at various confining pressures at a relative density of sand of 80%. The values of constants viz. k , the modulus number

and n , the exponent determining the variation of initial tangent modulus with confining pressure, were obtained on basis of the approach presented by Duncan and Chang (1970) and are presented in Table 1.

TABLE 1 - SOIL PARAMETERS FOR NON-LINEAR ANALYSIS

Description	Magnitude
(1)	(2)
Relative density	80%
E_i - Initial tangent modulus of sand	1500 kg/cm ²
K - Modulus number*	500.00
n - Exponent*	0.92
R_f - Failure ratio	0.85
C - Cohesion	0.0
ϕ - Angle of internal friction	37.50°
ν_s - Poisson's ratio of sand	0.35

* Corresponding to confining pressure, $\sigma_3 = 1 \text{ kg/cm}^2$

Incremental procedure has been adopted as the nonlinear solution technique.

NUMERICAL EXAMPLE

In order to investigate the effect of nonlinearity of soil mass and improved physical modelling, an unsymmetrically loaded five storey-two bay-plane frame supported by combined footing-soil system has been considered. Figure 2 shows the finite-infinite multielement idealization of structure-footing soil system. The bay width, storey height, cross-section of the structure members and of combined footing are all mentioned in Fig. 2.

The unsymmetrical loading includes vertical and lateral load acting on the frame with intensity of 800 kg/m and 225 kg/m respectively. The Young's modulus and Poisson's ratio values used for structure are mentioned in Fig.2. The material properties of the soil mass derived using non-linear stress-strain characteristics are presented in Table 1.

RESULTS AND DISCUSSION

The frame structure-combined footing-soil system subjected to both vertical and lateral loads becomes a case of unsymmetrical loading and therefore a full finite-infinite element mech (Fig. 2) was considered for analysis. The effect of lateral load will be to introduce the shear stresses along the base of the combined footing. This base shear has considerable influence on the response of frame structure and was taken into account by introducing an interface/joint element with zero thickness between combined footing and soil mass.

The values of normal and shear stiffness of the footing-soil interface used in the analysis are - $K_{nn} = 100,000 \text{ kg/cm}^2$; $K_{ss} = 1000 \text{ kg/cm}^2$. The value of K_{ss} is much less compared to that of K_{nn} in order to allow for the smoothness of surface and therefore the sliding along the interface (King et al., 1983).

Settlement Figure 3 shows the vertical settlement profile of the combined footing. The plots are made for alternate load increments and also for linear analysis. The final settlements due to soil nonlinearity are almost twice the settlement due to linear behaviour.

Sway The sway of the structure has been plotted in Fig. 4 for alternate load increments of nonlinear analysis. The sway obtained in noninteractive and linear interactive analyses are also superimposed in the same figure. The sway of the frame, due to consideration of soil nonlinearity, increases by about 25 percent over that of linear interactive analysis and is about 10-30 times of that due noninteractive analysis.

Contact Pressure Figure 5 shows the distribution of contact pressure below the combined footing for various load increments and also for linear analysis. The nonlinear analysis yields 30 percent higher value for the pressure at the edge of combined footing. The contact pressure at the centre of the beam is about 2.75 times the applied pressure intensity whereas at the edge, it is almost about 5.5 times.

Base Shear The development of shear stresses along the base of combined footing for different load increments and also due to linear analysis is presented in Fig. 6. The plot also indicates that non-linearity of soil mass has a tendency to increase base shear at both edges. The maximum shear stress on windward side due to soil nonlinearity is 2.45 times and on leeward side, 1.33 times the shear due to linear behaviour.

Bending Moments Table 2 gives the comparative values of bending moments in typical beam members obtained via three different analyses. The percentage variation induced in bending moment values obtained on basis of linear interactive analysis (column 5) over the non-interactive analysis (column 4) is as large as 92 percent (column 6). Reversal in the nature of bending moment has also been observed in both the beam and column members. The percentage increase in the values of bending moments due to soil non-linearity (column 8) is also significant. Majority of the beams are affected and the variation is almost ± 18 percent indicating either an increase or decrease. This means that soil nonlinearity plays a very significant role in deciding the structural behaviour.

TABLE 2 - BENDING MOMENT (kg-m) VARIATION CORRESPONDING TO VARIOUS ANALYSES
(Vertical + lateral loading)

Floor	Member Type	Ends	Non-Interactive		With Interaction			
			(3)	(4)	Linear. (5)	% difference between 4 and 5 (6)	Nonlinear (7)	% difference between 5 and 7 (8)
Foundation beam	B ₁₁	17			2549.88		2478.49	-2.80
	B ₁₂	17			- 577.12		- 472.69	-18.10
		18			6950.19		5859.84	-15.69
	B ₁₃	18			-4732.16		-4605.10	-2.77
		19			2243.28		1953.67	-12.90
I		19			- 237.34		- 150.20	36.72
	B ₉	13	2807.38		3874.91	+38.00	3595.00	-7.20
		14	955.40		1249.23	30.80	1336.39	+7.00
	B ₁₀	14	2871.17		1894.66	-34.00	1833.75	-3.20
		15	376.75		- 485.95	*	- 554.86	+14.20
II	B ₅	7	2281.73		3166.37	38.78	3221.75	+1.75
		8	- 380.20		608.07	*	669.89	+10.20
	B ₆	8	2289.18		1302.27	43.10	1242.40	-4.60
V		9	-1752.90		-1857.78	39.66	-1112.96	+5.20
	B ₁	1	794.10		1469.91	85.00	1511.50	+2.83
		2	- 442.42		422.89	*	476.18	+12.60
	B ₂	2	749.77		- 159.17	*	- 168.84	+6.10
		3	- 493.19		-1703.00	+245.0	-1211.63	28.86
I	C ₁₃	13	-1197.32		-1659.47	38.60	-1701.00	+2.50
		17	-2298.23		-1973.27	-14.20	-2006.31	+1.65
	C ₁₄	14	-1262.47		-1430.47	13.30	-1491.28	+4.25
		18	-2394.98		-2111.88	-9.60	-2255.56	-6.80
	C ₁₅	15	- 57.50		+ 88.40	*	155.54	75.96
		19	-1960.90		-2006.20	2.30	-1804.04	-10.10

* Corresponds to reversal of bending moments.

SUMMARY AND CONCLUSION

1. Attempt is made in the present study to model a soil-structure interaction problem using a coupled-finite-infinite element formulation and taking into account the non-linearity of the soil mass.
2. The use of a zero-thickness interface element between soil and foundation is essential for laterally loaded structure.
3. On basis of the interactive analysis, it was realised that nonlinear analysis yields very high values of settlement and contact pressure.
4. Both the interactive behaviour and nonlinearity of the soil mass significantly affect bending moments in various structural members and can cause reversal of bending moments.

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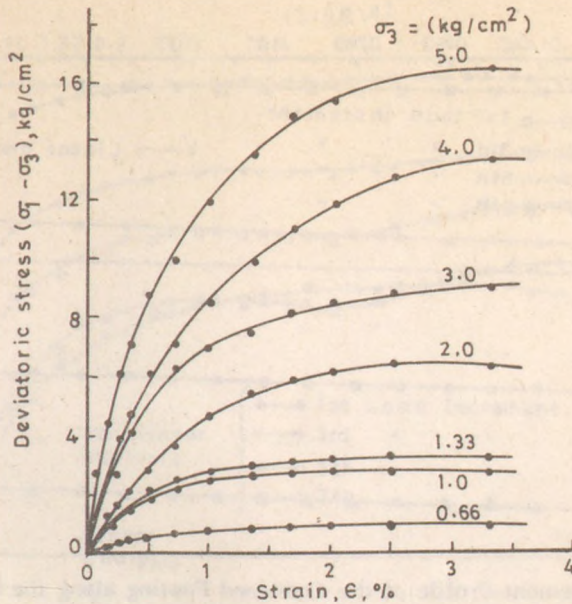


Fig. 1 - Stress-Strain Relationship at Various Confining Pressures

Not to scale

All dimensions in cm

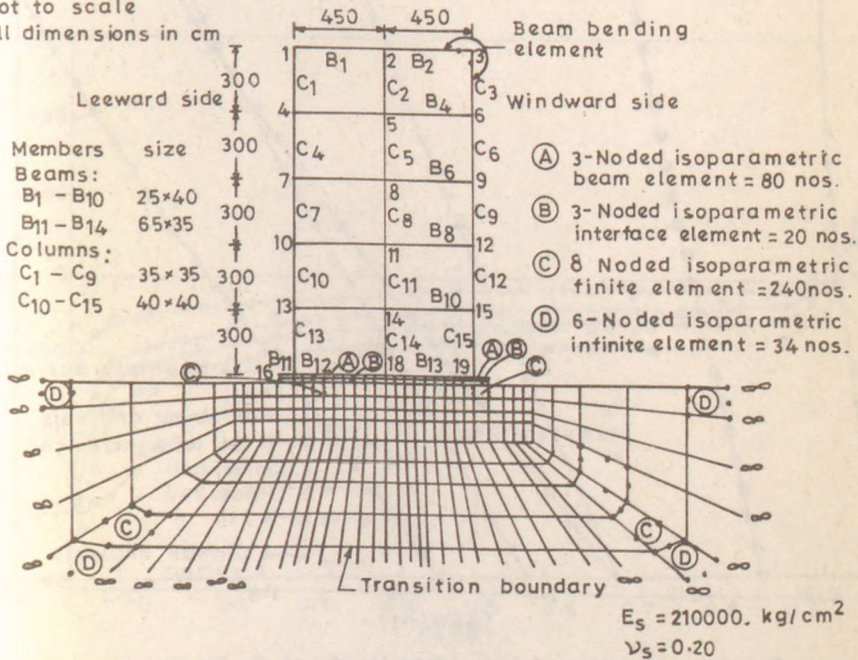


Fig. 2 - Finite-Infinite Multielement Idealisation of Multistorey Plane Frame - Footing-Soil System

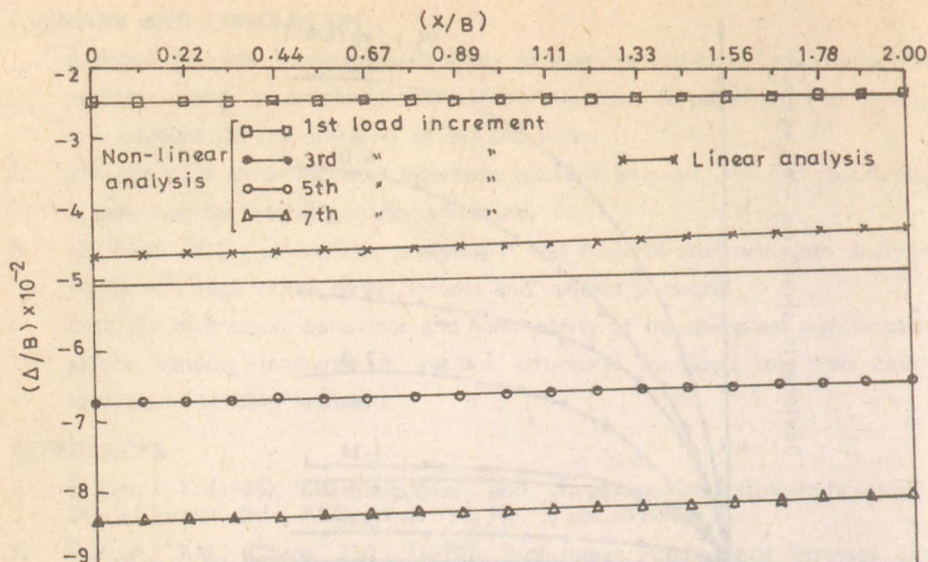


Fig. 3 - Settlement-Profile of the Combined Footing along the Base

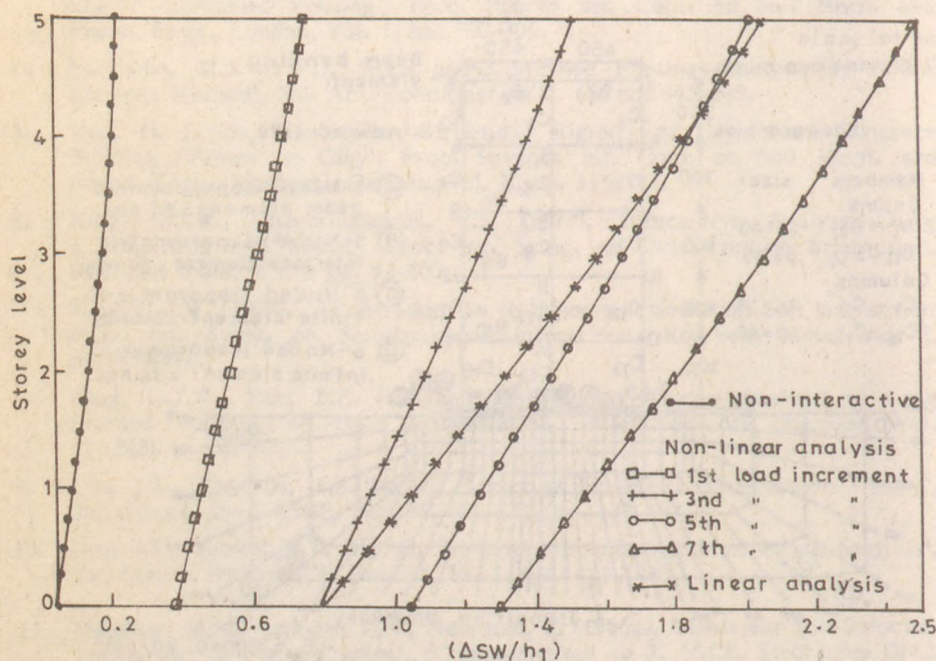


Fig. 4 - Variation of Sway along Height of the Frame

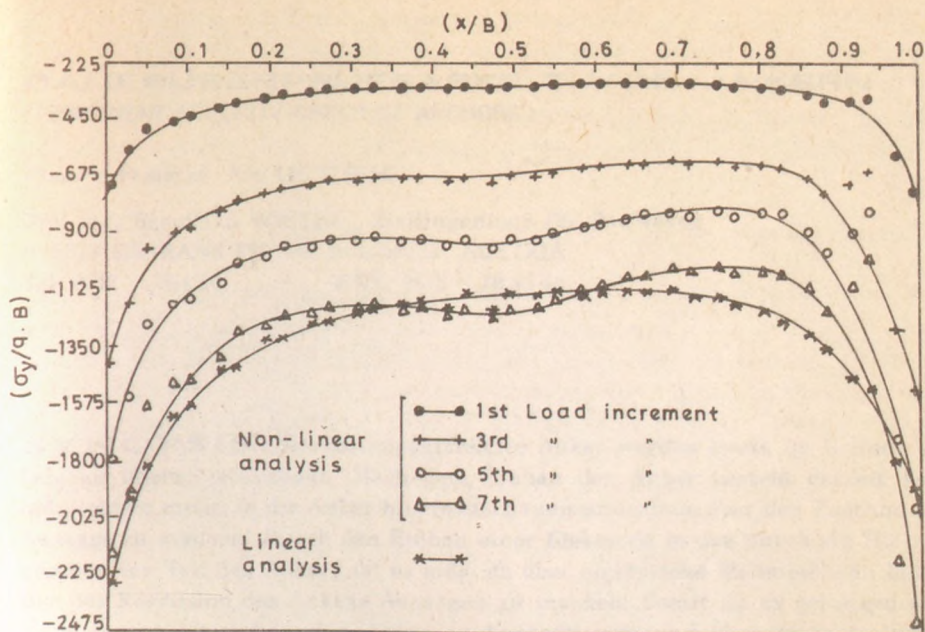


Fig. 5 - Contact Pressure Distribution Below the Combined Footing

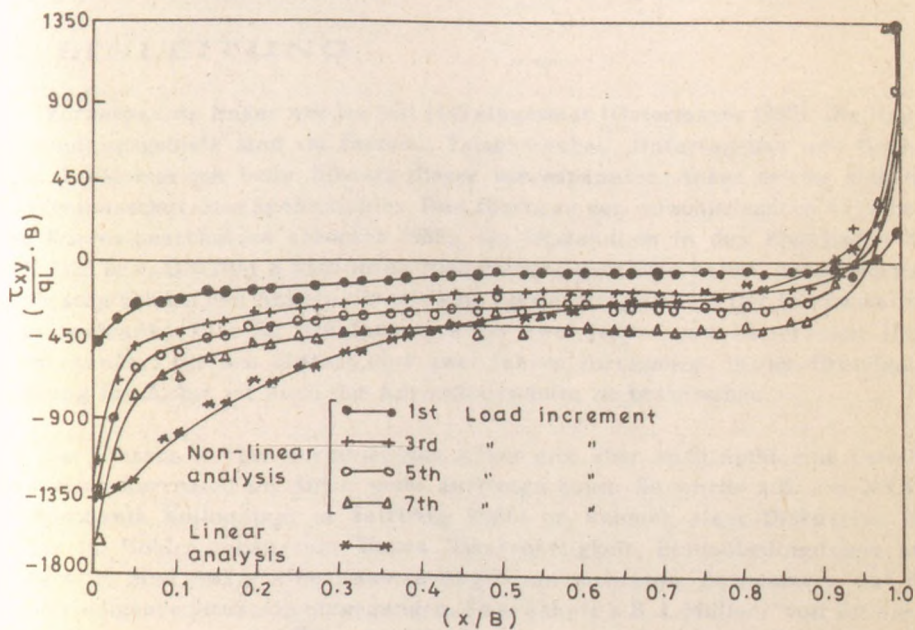


Fig. 6 - Variation of Shear Stresses Along the Base of the Combined Footing

DAUERÜBERPRÜFBARKEIT VON ANKERN

(PERMANENT SECURITY CHECK OF ANCHORS)

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SYNOPSIS : Korrosionsschutzte Anker werden heute im Grund- und Felsbau überall eingebaut. Nach dem Einbau der Anker besteht derzeit keine Möglichkeit mehr, in die Anker hineinzuschauen und somit über den Zustand eine Aussage zu machen. Durch den Einbau einer Elektrode in den durch ein Hüllrohr geschützten Teil des Ankers ist es möglich über elektrische Meßmethoden bezüglich der Korrosion des Ankers Aussagen zu machen. Somit ist es gelungen auch nach dem Ankereinbau den Anker zu kontrollieren und über seine Sicherheit Aussagen machen zu können.

1. EINLEITUNG

Vorgespannte Anker werden seit 1935 eingesetzt (Ostermayer 1982). Die Hauptanwendungsgebiete sind im Felsbau, Talsperrenbau, Untertagebau und Grundbau. Problematisch beim Einsatz dieser vorgespannten Anker erwies sich der Korrosionsschutz des Spannstahles. Dies führte zu den verschiedensten Varianten des Korrosionsschutzes (Stocker 1985), die letztendlich in den Richtlinien der DIN 4125 bzw. ÖNORM B 4455 ihren Niederschlag gefunden haben. Beide Normen unterscheiden bei den Ankern die zeitliche Verwendungsdauer. Der Kurzzeitanke (Temporäranke) ist für den Einsatz unter zwei Jahren, der Daueranke (Permanentanke) für den Einsatz über zwei Jahren vorgesehen. In der Grundsatzprüfung für Anker ist auch der Korrosionsschutz zu beschreiben.

Der Einsatz solcher normmäßigen Anker gibt aber auch nicht eine Gewähr, daß keine Korrosion im Anker mehr auftreten kann. So wurde z.B. am XXXV. Geomechanik Kolloquium in Salzburg (1986) im Rahmen einer Diskussion um Anker im Hohlraumbau zum Thema "Dauerhaftigkeit, Einbaubedingungen und Kontrolle. Sind Anker eine Dauerlösung?" von mehreren Teilnehmern auf die unbefriedigende Situation eingegangen. So erwähnte z.B. L.Müller: "von Schwierigkeiten mit ausrinnenden Öl", oder "der Zement am Anfang der Haftstrecke reißt immer"; oder R.Widmann: "daß bei 3 von insgesamt 8 Ankern zusammen 5 Spanndrähte gerissen waren"; oder F.Pacher: "Wie kann man die Lebensdauer eines Ankers anschätzen, in den man nicht hineinschauen kann?"

Nicht nur durch Materialverhalten des Ankers, sondern auch durch Bewegungen im Gebirge werden klaffende Risse in der Umhüllung des Ankers erzeugt (Schubert 1984). Somit kann bei Ankern eine abtragende Korrosion und auch örtliche Korrosion in Form von Loch- bzw. Spaltkorrosion und von Spannungsrißkorrosion auftreten.

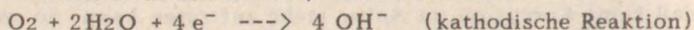
2. KORROSION

Der Korrosionsvorgang kann als das Bestreben eines Metalles angesehen werden, wieder in den ursprünglichen oxidierten Zustand zurückzukehren. Dies wird durch einen elektrochemischen Vorgang erreicht. Die meisten Metalle sind daher korrosionsanfällig, weil sie zusammen mit den umgebenden Medien wie Wasser und Luft ein thermodynamisch instabiles System darstellen, welches bestrebt ist, Energie durch Bildung einer chemischen Verbindung des Metalls freizusetzen. Eine übersichtliche Darstellung der Voraussetzungen für elektrochemische Korrosion in feuchter Umgebung findet man im sogenannten Pourbaix-Diagramm, in dem der Zusammenhang zwischen pH-Wert und Redoxpotential aufgetragen ist (Wranglen 1985).

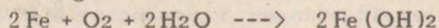
Da der Beton bzw. Zementstein, der einen Anker umhüllt einen pH-Wert meist über 12 besitzt, ist der Stahl dauerhaft vor Korrosion geschützt. Nur in Fällen wo diese Schutzwirkung durch äußere Einflüsse gestört werden, kommt es zu einem elektrochemischen Vorgang der Korrosion. Dabei wandelt sich das Metall (Fe) in ein Oxid (Fe_2O_3 oder FeO) um, das meist noch zusätzlich Wasser enthält. In den verschiedenen Oxiden liegt das Eisen 2- oder 3-fach positiv geladen vor, der Sauerstoff 2-fach negativ. Als Bedingung daß dieser Korrosionsvorgang abläuft müssen folgende Bedingungen gegeben sein:

- * es muß ausreichend Feuchtigkeit in der Umgebung des Stahles sein
- * die Sauerstoffzufuhr muß gewährleistet sein
- * der pH-Wert muß unter 9 liegen (Tritthart 1989)

Wenn Stahl in Zementstein korrodiert, beginnt dies an irgend einer Stelle mit folgendem ersten Schritt: $\text{Fe} \rightarrow \text{Fe}^{++} + 2\text{e}^-$ (anodische Reaktion) Die Elektronen wandern im Stahl an eine räumlich getrennte Stelle (einige mm bis dm) und rufen dort eine zweite Reaktion hervor, wenn die elektrochemische Umgebung (pH-Wert, O_2 -Gehalt, Wasservorkommen) an dieser räumlich getrennten Stelle sich nur ein wenig von der ersten stelle unterscheidet. So kann z.B. Sauerstoff, der in den Zementstein diffundiert, reduziert werden:



Stets müssen aber die anodischen Reaktionen und die kathodischen Reaktionen gemeinsam ablaufen. Ist ene der beiden gehemmt, so kommt auch die andere zum Stillstand. Zusammengekommen läuft bei Korrosion folgende Reaktion ab:



Das gebildete Eisenhydroxid ist unlöslich und reagiert danach mit weiterem Sauerstoff und Wasser zu wasserhaltigen Eisenoxiden (Rost) in einer rein chemischen Reaktion (Kotowski et.al. 1988).

3. VERSUCHSANKER

Es wurden im Sommer 1989 vier Versuchsanker der Schutzklasse S2 mit einem Ankerstab ($d = 32 \text{ mm}$) der Länge von 1,20 m hergestellt. Zusätzlich zum üblichen Aufbau wurde ein Elektrodendraht innerhalb des Hüllrohres eingebaut (Abb. 1).

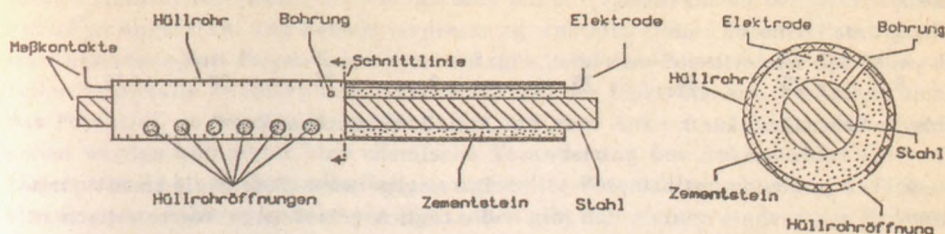


Abb. 1: Ansicht und Schnitt durch eigens hergestellte Versuchsanker

Bei jedem Anker wurde in Ankermitte ein 3 mm Loch gebohrt, das durch das Hüllrohr bis zur Stahloberfläche reicht. Zusätzlich öffnete man das Hüllrohr an 6 Stellen. Diese Öffnungen dienten zur Kontrolle der Messungen mit einer Kupfersulfat-Referenzelektrode. Für die ersten Meßreihen blieben die Anker unbehandelt, um den Aushärtungsvorgang meßtechnisch zu ermitteln und auch um Referenzwerte für die weiteren Messungen zu haben. Bei den darauffolgenden Meßreihen, die sich über Monate erstreckten, wurden die Versuchsanker unterschiedlich behandelt.

Bei Versuchsanker 1 wurde einmalig $0,3 \text{ cm}^3$ gesättigte Salzlösung in das Loch eingeträufelt und man beobachtete den Potentialverlauf zwischen Ankerstahl und eingebauter Elektrode, bis die Messungen die Grundwerte wieder erreichten.

Versuchsanker 2 erhielt täglich $0,3 \text{ cm}^3$ gesättigte Salzlösung in das Loch eingeträufelt.

Versuchsanker 3 erhielt täglich $0,3 \text{ cm}^3$ Leitungswasser eingeträufelt. Erst im späteren Verlauf wurde auf der gegenüberliegenden Seite eine weitere Bohrung hergestellt und mit täglich $0,3 \text{ cm}^3$ Salzlösung injiziert.

4. MESSUNGEN

Bei den Versuchsankern wurde der absolute Potentialunterschied zwischen dem Stahl und der eingebauten Elektrode gemessen, sowie auch der Potentialunterschied zwischen Ankerstahl und der jeweils oberflächenhaft angedrückten Kupfersulfat-Referenzelektrode. Die Messungen wurden mehrmals täglich vorgenommen.

Die Ergebnisse der Potentialmessungen zwischen Ankerstahl und eingebauter Elektrode sind in den nachfolgenden Abbildungen aufgetragen.

Potentialmessungen bei den einzelnen Versuchsankern:

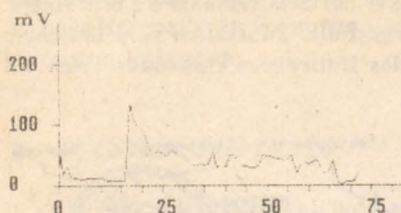


Abb. 2: Versuchsanker 1

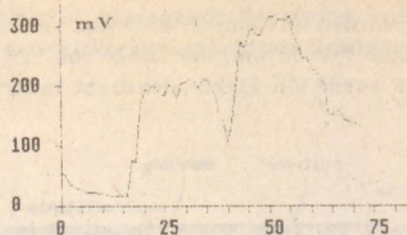


Abb. 3: Versuchsanker 2

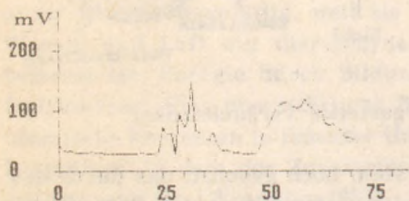


Abb. 4: Versuchsanker 3

Die angegebenen Potentiale in mV sind die täglich gemessenen Werte zwischen Ankerstahl und der eingebauten Elektrode bestehend aus platinisiertem Titandraht. Messungen mit der Kupfersulfat-Referenzelektrode sind hier aus Platzgründen nicht angeführt.

Man erkennt deutlich, daß bei Meßanker 1 zuerst eine Abflachung der mV-Kurve eingetreten ist. Dies ist im Zeitraum des Zementabbindens. Anschließend erkennt man ab der 16. Ablesung eine sprunghafte Steigerung des Potentials um ca. 130 mV. Der folgende Potentialabfall ist sehr unruhig, jedoch ist die generelle fallende Tendenz klar erkennbar. Es wird langfristig wieder das Ausgangspotential erreicht, da keine weitere Zuführung von Salzlösung erfolgt ist. Bei Meßanker 2 nach der Abflachung des Ausgangspotentials durch die dauernde Zugabe der Salzlösung ab der 16. Messung rasch eine Potentialsteigerung von ca. 200 mV erreicht. Dieser Potentialsprung blieb auch bis zum Aussetzen der Zugabe der Salzlösung etwa konstant und fällt erst ab der 37. Messung wieder ab. Von der 42. Messung ab wurde wiederum konstant bis zur 55. Messung Salzlösung beigegeben. Man erkennt dies sehr deutlich bei den Potentialen. Bei Meßanker 3 kann nach der anfänglichen Abflachung ein sehr unruhiger Potentialverlauf mit Sprüngen bis max 100 mV erkannt werden. Erst ab der 47. Messung beginnt eine gleichmäßige Potentialsteigerung. Diese stammt aus der Zugabe von Salzlösung am gegenüberliegenden Stabseite und ca. 1 m von der Meßstelle entfernt. Es scheint hier ein gewisser Dämpfungseffekt mit zunehmender Entfernung einzutreten.

Die Kontrolle der jeweiligen Stahloberfläche ergab, daß bei Meßanker 1 direkt nach dem ersten Potentialsprung Korrosion entstand, diese jedoch nach ca. 7-10 Messungen keinen weiteren Fortschritt nahm. Bei Meßanker 2 war ab der 18. Messung deutlich die Korrosion der Stahloberfläche zu beachten, die bis zum Ende der Messungen aufrecht blieb. Bei Meßanker 3 konnte bei der mit Wasser behandelten Stelle keine Veränderung der Oberfläche festgestellt werden. Nur auf der Meßstelle an der später Salzlösung aufgebracht wurde, konnte eine sofortige Reaktion in Form von Korrosion erkannt werden.

5. PSC-ANKERPRINZIP

Durch die Versuchsanker konnte nachgewiesen werden, daß eine Messung der Korrosion bei doppelt geschützten Ankern - im vorliegenden Fall Einstabanker - möglich ist, wenn innerhalb des Hüllrohres eine oder mehrere elektrisch kontaktierbare Elektroden eingebaut werden. Die so eingebauten Elektroden erlauben es, jenes Potential festzustellen, welches sich bei der Korrosion an der Grenzfläche zwischen Ankerstahl und Zementverpressung einstellt. Dieses Potential stellt sich im stationären Fall (keine Korrosion) entsprechend dem Potential der Spannungsreihe der Metalle zwischen dem Ankerstahl und der Elektrode ein. Verändert sich das Potential, so ist dies ein Indiz dafür, daß vom Ankerstahl Elektroden abgegeben werden und somit eine chemische Veränderung des Ankerstahles eintritt (Korrosionsbeginn). Eine allenfalls festgestellte Potentialverschiebung zwischen einzelnen im Anker angebrachten Elektroden gibt daher einen eindeutigen Hinweis darauf, daß der doppelte Schutz des Ankerstahles durch Alkalität des Zementsteines und Hüllrohe nicht oder nicht völlig ausreichend gegeben ist.

Ein besonderer Vorteil dieses beschriebenen Systems liegt darin, daß es im eingebauten Zustand erlaubt, einen allenfalls festgestellten Korrosionsvorgang zu beenden, indem durch eine äußere Spannungsquelle ein schwacher Strom zwischen Ankerstab und Elektrode zu fließen gebracht wird, welcher gerade so viele Elektronen an den Ankerstahl liefert, wie beim ungestörten Ablauf der Korrosion vom Stahl an den Sauerstoff abgegeben werden. Diese Maßnahme ist als "Kathodischer Korrosionsschutz" bekannt. Ein besonderer Vorteil des kathodischen Korrosionsschutzes ist im vorliegenden Fall darin zu sehen, daß die den Ankerstahl umgebende Zementmörtelschicht nicht entfernt werden muß, wenn sie aufgrund von äußeren Einflüssen ihre Alkalität verloren hat, sofern sie nicht auch mechanisch bereits zerstört ist.

6. INTERPRETATION DER MESSUNGEN

Bei den mit mehreren Elektroden ausgerüsteten Anker können sofort nach der Herstellung die Messungen beginnen. Aufgrund der Messungen der einzelnen Potentiale können nun wegen der zeitlichen Reihenfolge bedingt verschiedene Fehlerarten bei den Ankern gefunden werden, die derzeit nicht oder nur unzureichend aufgespürt werden können:

- a) Bestimmung von Herstellungsfehlern vor dem Ankereinbau
- b) Feststellung von Transportschäden vor dem Einbau
- c) Erkennung von Einbauschäden (Kontrolle während dem Einbau)
- d) Messung von geringsten Langzeitveränderungen bei eingebauten Ankern
- e) Messung von elektrochemischen Korrosionsvorgängen (Makroelemente)
- f) Messung jedweder Aggressivität des Korrosionsumfeldes

Die Messungen d,e und f können nach dem Einbau des Ankers jederzeit durchgeführt werden. Sinnvollerweise wird hier eine periodische Überprüfung empfohlen, die dann auch über das Langzeitverhalten eine absolute Auskunft gibt.

7. AUSBLICK

Die Entwicklung für dieses Meßsystem ist nun nach ca. einem Jahr soweit gediehen, daß an eine Verwirklichung in der Praxis gedacht werden kann. So werden derzeit nach zahlreichen Versuchsankern auch Anker für den Einbau auf Baustellen hergestellt. Diese Anker werden nach dem System von Dyckerhoff & Widmann (München) in Österreich von der Fa. Allspann (Salzburg) hergestellt. Die Kontrolle und Auswahl der eingebauten Elektroden wurde von der Fa. Heraeus durchgeführt. Die Potentialmessungen und deren Auswertung werden von unserem Ingenieurbüro durchgeführt.

Das derzeit vorliegende System soll nicht nur auf Einstabanker mit oder ohne Muffenstoß beschränkt bleiben. Es ist nach ausreichender Erfahrung mit dem Einstabanker auch an eine Erweiterung des Systems für Litzenanker gedacht, sodaß alle Langzeitanker mit doppelten Korrosionsschutz auch diese zusätzliche Möglichkeit der Überwachung haben.

Die Kosten des zusätzlichen Meß- und Schutzsystems bei einem Anker bedingt eine Preissteigerung von 10 bis 20%. Bei Bauwerken mit langer Lebensdauer (> 15 Jahre) ist dieser Mehrpreis sicherlich zu vertreten, da auch nach vielen Jahren das System kontrollierbar bleibt und durch den eingebauten kathodischen Korrosionsschutz eine Lebensdauer um mehrere Jahrzehnte verlängert werden kann.

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THE EUROCODE - SAFETY APPROACH AS UPRIGHT TO FOUNDATIONS

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Some unusual terms and procedures will be contained in the Eurocode (EC) generation of standards caused by probabilistic safety considerations. Therefore a query is raised: Shall we change from the deterministic to a probabilistic safety approach also in foundation engineering, working with a non-man-made material the properties of which are scattering by far more than the ones of concrete and steel? To answer this question we have first to be informed about the state of development.

It is already known that the new generation of standards will contain 5 elements which may be more or less unusual up to now:

1. The use of partial safety factors
2. The principle of consistent limit state equations
3. The principle of limit state design
4. The principle of different geotechnical categories
5. The observational method

In the following text these 5 items are explained (beginning at the end).

TO ITEM 5: OBSERVATIONAL METHOD

Definition of Therzaghi (published by PECK 1969): " In the past, only two methods have been used for coping with the inevitable uncertainties: either to adopt an excessive factor of safety or else to make assumptions in accordance with general, average experience..... The first method is wasteful; the second is dangerous. Soil mechanics, as we understand it today provides a third method which could be called the experimental method. The procedure is as follows: Base the design on whatever information can be secured. Make a detailed inventory of all the possible differences between reality and the assumptions. Then compute, on the basis of the original assumptions, various quantities that can be measured in the field.... On the basis of the results of such measurements, gradually close the gaps in knowledge and, if necessary modify the design during construction..... Soil mechanics provides us with the knowledge required for practical application of this 'learn- as-you-go' method."

Instead of this very suitable term it was then called 'observational method' and recently it is spoken of 'monitored decisions'.

The definition according to EC 7 writes: "Because prediction of geotechnical behaviour is often very difficult, it is sometimes appropriate to adopt the approach known as 'the observational method', in which the design is reviewed during construction. When this approach is used, the following requirements must all be met before construction is started.

- The limits of behaviour which are acceptable must be established.
- The range of possible behaviour must be assessed and it must be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits.
- A plan of monitoring must be devised which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring must make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully. The response time of the instruments must be sufficiently rapid in relation to the possible evolution of the system.
- A plan of contingency actions must be devised which may be adopted if the monitoring reveals behaviour outside acceptable limits."

TO ITEM 4: GEOTECHNICAL CATEGORIES

In order to establish minimum requirements for the extent and quality of geotechnical investigations, calculations and construction control checks, the difficulty and complexity of each geotechnical design must be clearly identified. To facilitate this, three 'Geotechnical Categories' are defined.

Geotechnical Category 1

This only includes small and relatively simple structures:

- for which it is possible to ensure that the performance criteria will be satisfied on the basis of experience and qualitative geotechnical investigations
- with no risk for property and life

Geotechnical Category 2

This category includes structures for which quantitative geotechnical data and analysis are necessary to ensure that the performance criteria will be satisfied, but for which conventional procedures of design and construction may be used. These necessitate the involvement of qualified engineers with relevant experience.

Geotechnical Category 3

Geotechnical Category 3 includes very large or unusual structures, structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions or structures in highly seismic areas.

The involvement of experienced engineers with relevant experience in the related geotechnical problems, will be necessary in these projects.

TO ITEM 3: LIMIT STATE DESIGN METHOD

Whenever a structure or part of a structure fails to satisfy one of its performance criteria it is said to have reached a 'limit state'. This Code is based on the 'limit state method' in which limit states are considered separately in the design and their occurrence is either eliminated or shown to be sufficiently improbable. The limit states considered are selected as to encompass the complete range of possible limit states.

Different classes of ultimate limit states (ULS) are considered in this code:

- Type 1: an ultimate limit state at which either
 - (A) a failure mechanism is formed in the ground, or
 - (B) a failure mechanism is formed in the structure due to movements in the ground
 - (C) a failure develops in the foundation (e.g. in a pile)
 - (D) loss of static equilibrium (e.g. by sliding or uplift)
- Type 2: a serviceability limit state at which deformation in the ground will cause loss of serviceability in the structure. (Relevant serviceability limit states include e.g. deformations or deflections which affect damage to finishes or non-structural elements.)

TO ITEM 2: THE PRINCIPLE OF CONSISTENT LIMIT STATE EQUATIONS

An example for non-consistent limit state equations is shown for the case of a bridge abutment like the one of fig.1 at the top of a slope:

There is the demand to show that (with τ_f , τ and R acc. to fig.1)

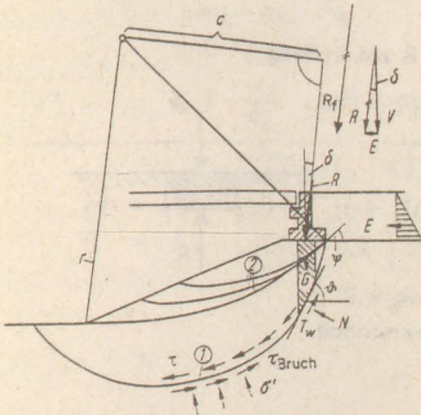
- safety requirements against slope failure are fulfilled, using the Fellenius rule

$$F_{\tau} = \frac{\tau_f}{\tau} \geq 1.3.....1.4 \text{ (DIN 4084)} \tag{1}$$

- safety requirements against failure of the abutment (by a bearing capacity case) are fulfilled, using the ratio

$$F_R = \frac{R_f}{R} \geq 2 \text{ (DIN 4017)} \tag{2}$$

How to avoid this inconsistency (by use of always the same safety definition) is shown in connection with explanations for the next item 1.



$$F_{\tau} = \frac{\tau_f}{\tau} \geq 1,3$$

$$F_R = \frac{R_f}{R} \geq 2,0$$

Fig.1 Different safety verifications F_{τ} and F_R for slope stability and bearing capacity of the bridge abutment are inconsistent

TO ITEM 1: THE USE OF PARTIAL SAFETY FACTORS

Due to EUROCODE 1 partial safety factors are defined in terms of a probabilistic safety approach. Therefore this approach must be explained first. It is tried to do this in a very simplified manner only using a few figures, but it is claimed that if one has understood these few figures one is enabled to judge the advantages and the disadvantages of the probabilistic so-called β -method.

Another reason to explain the probabilistic approach is that in the generation of new standards a terminology is used which must be learned like a strange language. The meaning of the words of this new language can only be understood by getting knowledge of the β -method. New words are e.g. 'characteristic value' of a property, 'safety index' β , 'failure probability' p_f , 'basisvariables', 'limit state equation'.....

To introduce the probabilistic approach we first throw a view backward: How have we defined safety up to now?

We have used a safety factor (now called global or deterministic safety factor) in form of the ratio

$$F = \frac{\text{stress or force at failure}}{\text{stress or force under working conditions}} \quad (3)$$

This ratio is now more exactly distinguished as

$$F = \frac{R}{S} = \frac{\text{Resistances (of building materials)}}{\text{Actions (of weight, traffic,.....)}} \quad (4)$$

(see the example of fig.2)

Now we have to consider that the resistances R as well as the actions S are scattering, they are variables (called basisvariables of our safety considerations).

Compared with equ.(3) and (4)

$$F = \frac{m_R}{m_S} \quad (5)$$

where m_R and m_S are the mean values of R and S (see fig.3)

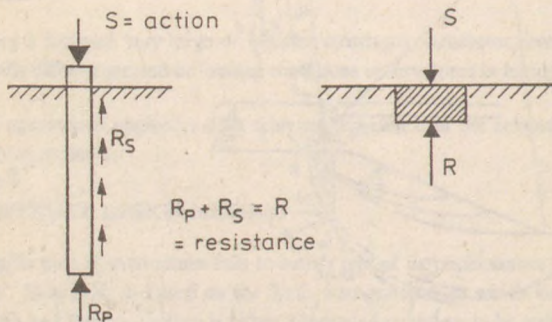


Fig. 2 Actions S and resistances R (often consisting of more than one component)

The derivations are done for simplicity for only 2 basisvariables R and S.

Generally n components of R and S require for derivation in a n-dimensional space.

Fig.2 shows that we proceed. " From the unsafe safety to the safe unsafety" (HAMPE 1989).

By figures 3, 4 and 5 is shown the derivation of partial safety factors γ_M and γ_F which are now applied separately to the resistances and the actions. Besides this the role of 'sensitivity factors' α is shown.

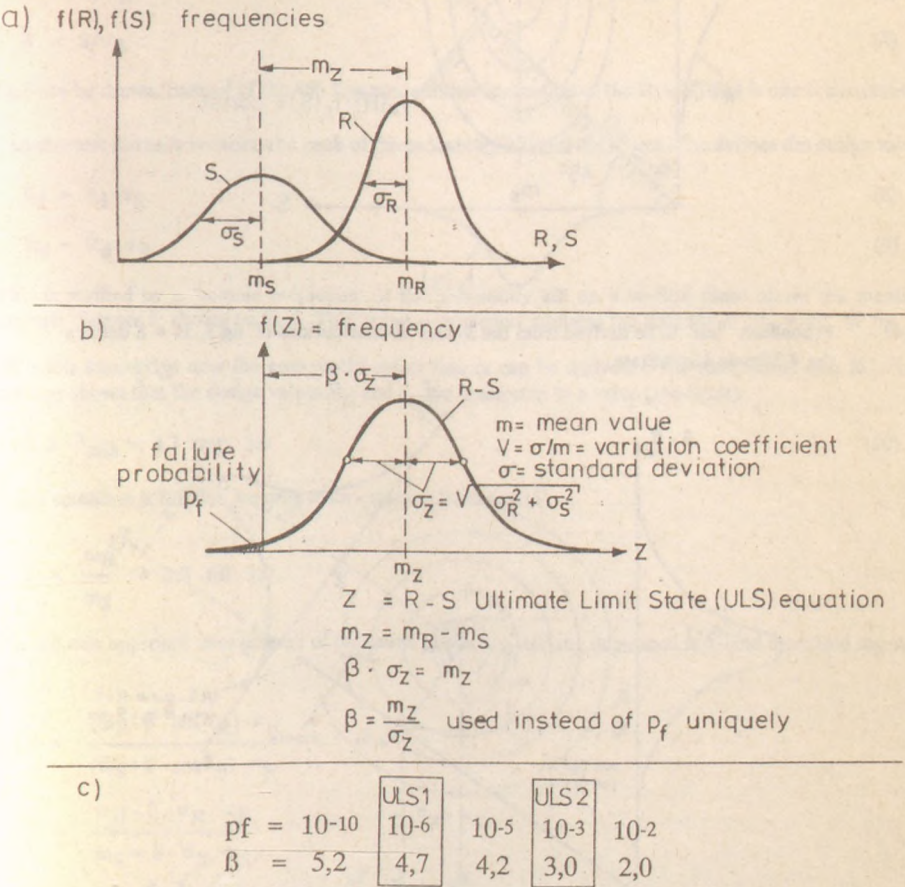


Fig. 3 Scattering S- and R-values and their position to each other

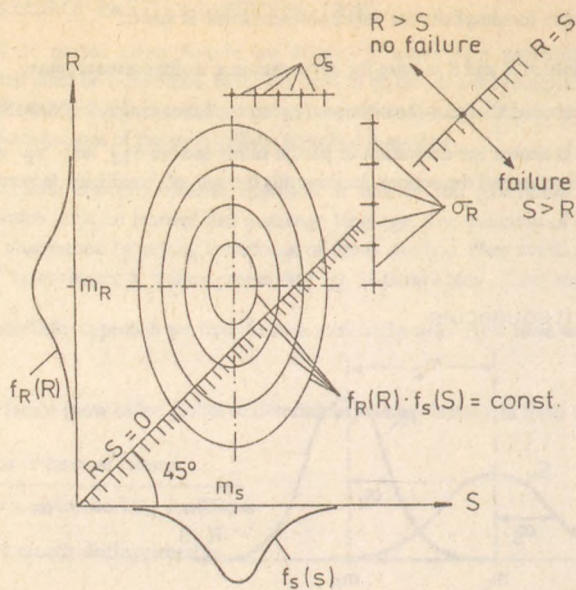


Fig. 4 Probability 'hill' to be derived from the S- and R-distributions of fig.3, $R = S$ defines the Ultimate Limit State

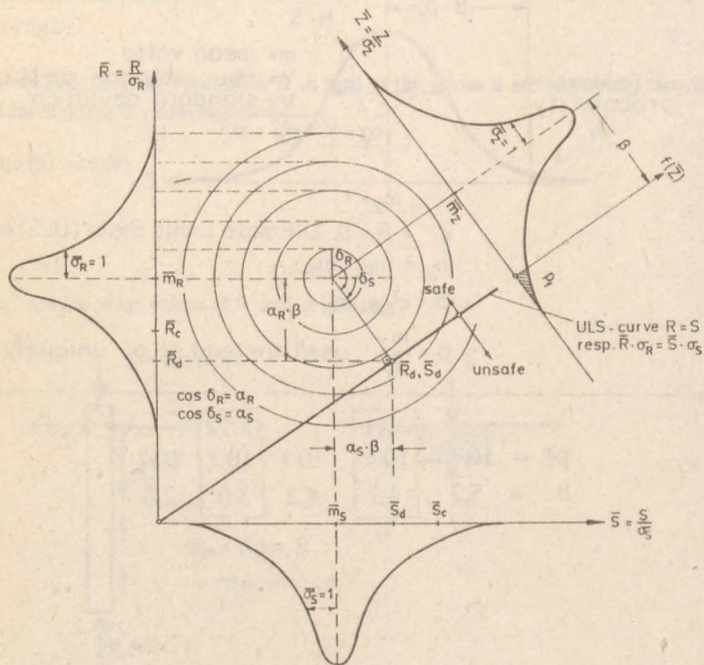


Fig.5 Normalized shape of fig.4

Fig.3 shows that -accounting for the scatter of S and R- always a failure probability p_f does exist because the Gauss-distribution functions are running from $-\infty$ to $+\infty$.

As m and σ are uniquely defining the shape of Gauss-functions $m_z = \beta \cdot \sigma_z$ can be used to replace p_f (shown in fig.3c).

Fig. 4 shows another possibility to express the safety probability of design cases e.g. the ones of fig.2. Each point of the probability hill defines the R- and S-values of the considered situation. $R = S$ defines the Ultimate Limit State. Referring R and S to their standard deviations σ_R and σ_S

$$\bar{R} = R/\sigma_R \quad (6)$$

$$\bar{S} = S/\sigma_S \quad (7)$$

fig.5 can be drawn. Instead of the 45°- line now another inclination of the $R = S$ - line is one consequence.

The shortest distance between the peak of the probability hill and the $R = S$ - line defines the design values

$$S_d = \bar{S}_d \cdot \sigma_S \quad (8)$$

$$R_d = \bar{R}_d \cdot \sigma_R \quad (9)$$

This is verified by a 'volume-projection' of the probability hill on a vertical plane above the mentioned shortest distance β , shown on fig.5. This 'volume projection' contains the distance $\beta \cdot \sigma_z = m_z$ of fig. 3b.

With this knowledge now the new partial safety factors can be derived: For each actual case in principle must be shown that the design values R_d and S_d are connected to a value (see fig.3c)

$$\beta > \beta_{\min} = 4.7 \text{ resp. } 3.0 \quad (10)$$

If this condition is fulfilled we may write - starting from equ.(5) -

$$F = \frac{m_R}{m_S} \geq 1.3 \text{ till } 3.0$$

But the new approach uses instead of the mean values m_R and m_S decreased m_R - and increased m_S -values, i.e.

$$\begin{aligned} F &= \frac{(\bar{m}_R - \beta \cdot \cos \delta_R) \cdot \sigma_R}{(\bar{m}_S + \beta \cdot \cos \delta_S) \cdot \sigma_S} \\ &= \frac{m_R - \beta \cdot \sigma_R \cdot \alpha_R}{m_S + \beta \cdot \sigma_S \cdot \alpha_S} \end{aligned}$$

and with the variation coefficient $V = \sigma/m$

$$= \frac{m_R(1 - \beta \cdot V_R \cdot \alpha_R)}{m_S(1 + \beta \cdot V_S \cdot \alpha_S)} = \frac{R_d}{S_d} \quad (11)$$

Using equ.(11) instead of equ.(5) we may decrease the required safety factor from 1.3 (for slope stability) till 3.0 (eg. for piles) to 1.0, i.e.

$$F = \frac{R_d}{S_d} \geq 1.0 \quad (12)$$

respectively the now introduced Ultimate Limite State equation

$$R_d \geq S_d \quad (13)$$

As seen from equ.(11) equ.(13) it can be written

$$R_d = m_R(1-\beta \cdot V_R \cdot \alpha_R) \geq m_S(1+\beta \cdot V_S \cdot \alpha_S) = S_d \quad (14)$$

or

$$R_d = \frac{m_R}{\gamma_M} \geq m_S \cdot \gamma_F = S_d \quad (15)$$

By the coefficient comparison above now we have found the partial safety factors

$$\gamma_M = \frac{1}{1-\beta \cdot V_R \cdot \alpha_R} \quad (16)$$

$$\gamma_F = 1+\beta \cdot V_S \cdot \alpha_S \quad (17)$$

which are applied to the basisvariables R and S and replace the global or deterministic safety factors F in the future.

Instead of mean values m_R and m_S so-called characteristic values R_c or S_c may be used not changing the shown principles.

Special emphasis earns the sensitivity factors α_R and α_S . They are defining the direction of β in fig.5, i.e. they determine the 'weight' of R and S accounting for the feed back between them.

CONCLUSIONS:

The high aim of the probabilistic approach can be better understood now: Its aim is to establish the same failure probability p_f for all parts of buildings including the subsoil. But as it is impossible to derive design values R_d and S_d for each actual case (like it is done sometimes in space and off-shore technique) it is tried to derive constant partial safety factors

$$\gamma_i = f(\beta, \alpha_i, V_i)$$

In principle it is impossible to derive constant partial safety factors, because there is a interdependence between all γ_i of one problem, expressed by the sensitivity factors α_i . But easy usage of standards in daily practice is not possible without constant partial safety factors γ_i . Therefore these factors γ_i are assumed to be constant if they lie in between a strip of ± 0.5 besides the required β -value of 4.7 for ULS 1 or 3.0 for ULS 2. Moreover for the γ_F -values of actions S and the γ_{Mi} -values of the resistances R the real interdependence is replaced by applying mean values of the corresponding sensitivity factors α_i .

It is this sort of simplification of the β -method which is causing mistakes in its application. These mistakes are the larger the smaller the available data basis is.

As a consequence in case of foundation engineering we have to consider:

- We have no man-made material, i.e. the material tests cannot be done one for all times like for steel or concrete.
- The material tests must be done separately for each actual case.
- Therefore the number of tests is regularly too small as to justify probabilistic derivations by the β -method.
- In this situation the derivation of partial safety factors is possible only by calibration to existing experience.

Derivation of partial safety factors by calibration to existing experience:

Why?

It is often argued now that the non-applicability of the probabilistic β -method at least in foundation engineering does not justify a change from our approved methods to the partial safety concept. Indeed, the main advantage doing it must be seen in the unification of terminology and of the safety calculations throughout the Civil Engineering profession, i.e. europewide the same things are done and discussed in the same way.

How?

Calibration of partial safety factors means to perform parameter studies, designing buildings resp. civil engineering structures from which we know by experience that they would fulfill all requirements if they were constructed. This is done first applying the usual global resp. deterministic safety factors. Then it is done a second time now applying constant partial safety factors to the mean 'basis' - values of actions S and resistances R (the variability of which is unknown) and adjusting them to the results of the old-fashioned design.

Comment:

We have begun this work and it proves to be rather difficult to derive partial safety factors in this manner esp. for retaining structures. For piles it is not possible to derive consistent partial safety factors for shaft and base resistance, only safety factors referred to the sum of shaft and base resistance forces makes sense.

FINAL REMARKS:

Even if besides the advantages by unification no improvement by the introduction of partial safety factors would be reached, it is said that by the 23,000 different standards in the EC a loss of 350 till 520 10^9 DM/a is caused. Insofar the big efforts necessary to create Eurocodes is justified.

Even the change to the partial safety concept by calibration to existing experience may be connected to dangers, because this calibration does not result in identically designs. Remember:

- "The building of soil mechanics is consisting of a complex mixture of theory and experience, very sensitive against the exchange of single stones by innovations" (Lang 1970).
- "Subsurface engineering is an art, soil mechanics is an engineering science. This distinction, often expressed but seldom fully appreciated must be understood if we are to achieve progress and proficiency in both fields of endeavour" (Peck 1962).

At least it must not be forgotten that safety cannot be achieved by pure analytical considerations. Safety is firstly a matter of design, e.g. expressed as follows:

- "The Designer's estimate of adequate safety is partly analytical and partly a matter of judgement - the accident-prone structure may be difficult to define, but can be recognised by the experienced. Human error cannot be eliminated, but good design will minimize its probability and consequences..... While the treatment of the 'conventional' stability aspects of a structure can readily be identified and then tackled by some formal calculations, robustness can only be treated by what I term the three E's: by eye, by experience and by engineering common sense!" (Rowe 1989).

One simple rule is certainly as important as safety calculations: Besides all calculations it is always important to design structures as soft as possible and as stiff as necessary.

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Bestimmung der Tragfähigkeit von Tiefgründungen auf Grund zahlreicher Probelastungsergebnisse

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I. Einleitung

Der mathematische Ausdruck der Probelastungskurve von Tiefgründungen hat mehrere Vorteile: die groben Ablesungsfehler können eliminiert werden, es können Gesetzmässigkeiten mit der Bearbeitung zahlreicher Proben zwischen den Bodendaten und der Tragfähigkeit festgestellt werden und anstatt der auf Grund der Praxis subjektiv festgestellten Tragfähigkeitsdaten /Bruchlast, Grenztragfähigkeit/ objektivere Daten können angegeben werden.

In Ungarn ist etwa 90 % der Probelastung von Tiefgründungen von FTV ausgewertet worden. Ein Teil davon /ca. 1000 Stück/ bezog sich auf vorgefertigte, Stahlbeton-Rammpfähle, ein anderer Teil /auch ca. 1000 Stück/ auf Bohr- und Rammpfähle und Schlitzwände verschiedener anderen Technologien. Die wichtigeren lokalen und geometrischen Daten der probelasteten Tiefgründungen, sowie die Last-Setzungs-Datenpaare der Belastung wurden im Interesse weiterer Untersuchungen in einen Computer eingetragen.

II. Beweisführung

Im Besitz der zahlreichen Probelastungsdaten wurden von uns unterschiedliche Funktionen /Parabel zweiten und dritten Grades, exponentielle Funktion, Hyperbel, Kombination einer Gerade und Hyperbel/ zur Annäherung der Last-Setzungs-Kurve ausprobiert. Die beste Annäherung ergibt sich bei der Funktion, deren Funktionstreuung die kleinste ist und auch den bodenmechanischen theoretischen Überlegungen auch nicht widerspricht. Laut unserer Untersuchungen werden diese Bedingungen durch die Kombination der Gerade und Hyperbel am besten befriedigt. Die Parameter der Funktion können von den Da-

tenpaaren mit der Methode der kleinsten Quadraten bestimmt werden, zweckmässigerweise mit Hilfe eines Computers, wozu ein Programm von unserem Kollegen László Szalay /FTV/ verfertigt wurde.

Unsere Grundformel ist die folgende:

$$P = a \cdot s + \frac{s}{s + s_H} P_{kt} \quad /1/$$

- wo a : ein Parameter für die Neigung der Gerade /kN/mm/
 s_H : ein Parameter für die Krümmung der Hyperbel /mm/
 P_{kt} : die senkrechte Asymptote der Hyperbel /kN/
 $\underline{P}$: die Last, die die Setzung $\underline{s}$ verursacht /kN/
 $\underline{s}$: die Setzung, die durch die Last $\underline{P}$ verursacht wird /mm/

Die Tieffundamente übergeben die Last dem Boden teilweise durch die Sohlen, teilweise durch ihre Mantelfläche,. Beide Komponenten sind Funktionen der Setzung $\underline{s}$, ausgelöst durch die Aussenlast $\underline{P}$.

Die Funktion der Mantelreibung soll eine Formel sein, die ausdrückt, dass es einen Lastgrenzwert gibt, bei welchem die Einsenkung der Tiefgründung unendlich gross ist, grössere Lastaufnahme ist nicht möglich. Bis zu dieser Last ist offenbar eine allmählich zunehmende Setzung zu erwarten. Mehrere Theorien und Vorschriften haben diese Kurve mit zwei oder drei geraden Strecken steigender Neigung annähert trotz der Tatsache, dass in den reellen Prozessen im allgemeinen keine Brüche gibt. Die von uns vorgeschlagene Hyperbel sichert die Kontinuität, also befriedigt die erwähnten Annahmen der Praxis.

Der Sohlenwiderstand kann linear mit der Setzung bis zu den Belastungen, die hinsichtlich der Praxis in Rechnung kommen, angenommen werden. Jene Probelastungen, bei denen die Formel der Setzung mit einer Potenz von 1 abweichend untersucht wurden, haben immer grössere Funktionstreuungen gezeigt, die Annäherung war also schlechter.

In der nächsten Tabelle führen wir das Ergebnis unserer Untersuchungen vor in denen die quadratischen Mittelabweichungen zwischen den Last-Setzungswertpaaren - bestimmt durch Probelastung von unterschiedlichen Pfahltyp und Schlitzwänden, verfertigt in unterschiedlichen Böden - und der mit Berechnung bestimmten Kurve berechnet wurden. In der Tabelle bedeutet der erste Buchstabe der Abkürzung den Pfahl- bzw. Tiefgründungstyp /F = Franki-, B = Benoto-, V = VUIS-, K = Ofenschlacke-Pfähle, D = Stöpselfundament unabhängig von Bodenart R = Schlitzwand/, der zweite Buchstabe die Bodensorte

/k = körnig, a = kohäsiv, e = Übergangsböden bei der Sohle/. Die Funktionsstreuung wird in KN bestimmt. Es wurde untersucht welchen Verteilungstyp der Datenhaufen bezüglich der Streuungen zeigt. Dazu wurden auch die Varianzen, die Schrägheit der Frequenzkurve und ihre Flachheit bestimmt. Auf Grund dieser Daten kann es festgestellt werden, dass die Verteilung der Streuungen mit guter Annäherung als normal betrachtet werden kann.

Funktionsstreuung der Annäherung von Probelastungen 1. tabelle

Typ	Stück	Durchschnitt $\bar{x}$ (kN)	Varianz σ_v^2	Schrägheit σ_g	Flachheit σ_k
Fe	151	43,5	0,592	1,011	0,886
Fa	153	41,0	0,505	1,036	1,557
Fk	210	39,9	0,553	0,741	0,386
F	514	41,4	0,615	0,002	-2,988
Be	7	60,9	0,604	-0,120	-1,347
Ba	26	86,0	0,538	0,480	-0,948
Bk	7	62,9	0,801	1,027	0,413
B	40	81,1	0,600	0,653	-0,178
Ke	40	15,7	0,627	0,645	-0,726
Ka	25	11,2	0,771	1,812	2,590
Kk	11	16,5	0,351	-0,293	-0,498
K	76	14,3	0,639	0,873	-0,135
LD	39	32,2	0,805	1,235	1,082
Se	8	106,1	0,625	0,036	-0,556
Sa	25	89,7	0,621	0,592	-0,264
Sk	15	79,9	0,609	0,892	-0,310
S	48	89,2	0,616	0,579	-0,447
Ve	6	16,0	0,509	-0,276	-0,500
Va	9	23,9	0,602	0,904	-0,020
Vk	17	20,6	0,569	0,635	-0,307
V	32	20,8	0,572	0,812	0,225
R	313	57,8	0,730	1,589	3,923

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Es ist sichtbar, dass der Durchschnitt der Streuungen verhältnismässig klein /ca. 10 % der in der Praxis festgestellten Bruchlast/, ihre Varianze dagegen ziemlich gross /ca 60 %/ ist.

Auf Grund unserer Berechnungen haben wir festgestellt, dass die untersuchte Funktion unabhängig vom Typ der Tiefgründung sowohl bei Pfählen, als auch bei Schlitzwänden die tatsächliche Probelastungskurve gut annähert.

III. Tragfähigkeit von Tieffundamenten in Funktion der bodenmechanischen Parameter von Böden $/E_s, \varnothing /$

Im weiteren wird es versucht, einen Zusammenhang zwischen der Tragfähigkeit der Tieffundamente und dem Zusammendrückungsmodul bzw. dem inneren Reibungswinkel des Bodens zu finden.

Sohlenwiderstand

Die Setzung ist laut des herkömmlichen vereinfachten Berechnungsmodells mit dem Sohlenwiderstand und mit der sich zusammendrückenden Schichtenstärke linear und dem Zusammendrückungsmodul des Bodens umgekehrt proportional. Unter Tiefgründungen kann die sich zusammendrückende Schichtenstärke im Fall eines Pfahls mit Kreis-Querschnitt auf $D/2$, bei viereckigen Schlitzgründungen auf die Hälfte der kürzeren Seitenlänge angenommen werden. Dieser ungewöhnlich kleiner Wert kann durch die grössere Tiefe und die Vorbelastung begründet werden. Auf Grund obiger kann die folgende Formel aufgeschrieben werden:

$$s = \frac{P_{\text{Sohle}} \cdot \frac{D}{2}}{\frac{D^2 \cdot \pi}{4} \cdot 2 E_s} = \frac{P_{\text{Sohle}}}{D \cdot \pi \cdot E_s} \quad /2/$$

daraus: $P_{\text{Sohle}} \approx 3 D E_s \cdot s$

Das mit dem ersten Glied der Formel /1/ verglichen

$$a \approx 3 D E_s$$

So kann aus der Probelastungskurve der Wert von E_s berechnet werden oder mangels einer Probelastung kann aus E_s das erste Glied der Probelastungskurve geschätzt werden.

Mantelreibung

P_{kt} im zweiten Glied der Formel /1/ ergibt den Bruchwert der Mantelreibung. Dieser hängt vom inneren Reibungswinkel des Bodens, bzw. daraus vom die Mantelfläche wirkenden Erddruck, sowie vom Reibungswinkel zwischen dem Material der Tieflgründung und dem Boden ab.

Bei Tiefgründungen ergibt sich entlang der Mantelfläche ein dem passiven Erdwiderstand ähnlicher Zustand wegen der grösseren Tiefe und der Verdichtungstechnologie, die im Laufe der Verfertigung des Fundaments angewandt wird. Davon berechnet im Fall einer homogenen Schicht ist die Mantelreibung:

$$P_{kt} = \frac{D \pi L^2}{2} \cdot \text{tg}^2(45^\circ + \varnothing/2) \cdot \text{tg} \varnothing \quad /3/$$

d.h. die spezifische Mantelreibung ist:

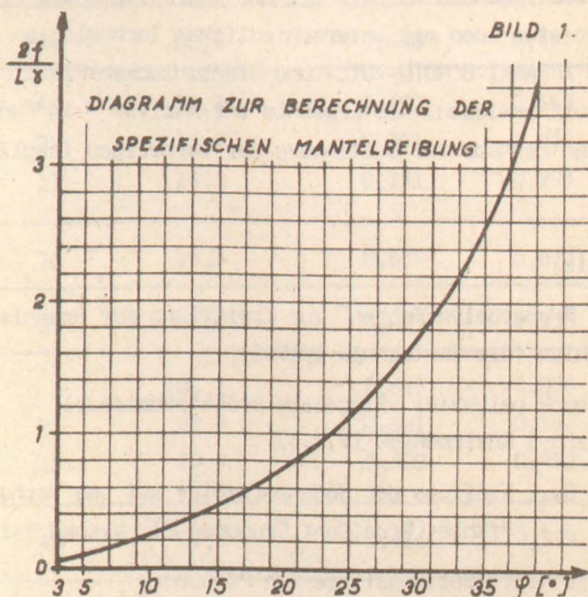
$$f_t = \frac{L \pi}{2} \cdot \text{tg}^2(45^\circ + \varnothing/2) \cdot \text{tg} \varnothing \quad /4/$$

Im Fall von beschichteten Böden sind der Erddruck und die Mantelreibung je Schicht zu berechnen und zu addieren.

Die vollkommene Formel für Last-Setzung ist also:

$$P = 3 \cdot D \cdot E_s \cdot s + \frac{s}{s + s_H} \cdot \frac{L^2 \pi}{2} \cdot \text{tg}^2(45^\circ + \varnothing/2) \cdot \text{tg} \varnothing \cdot D \cdot \pi / 5/$$

Wenn P_{kt} von der Probelastungsdaten bestimmt wird, kann der Reibungswinkel des Bodens ausgerechnet werden, oder umgekehrt: in Kenntnis des Reibungswinkels des Bodens ist P_{kt} bestimmbar. Dazu leistet das beigelegte Diagramm eine Beihilfe.



Der Wert s_H kann zwischen 0 und variieren. Bei diesen zwei Grenzen wird die Probelastungskurve linear, es handelt sich im ersten Fall um vorbelastete Böden, im zweiten Fall gibt es keine Mantelreibung.

Unserer Erfahrung nach kann dieser Wert in ungarischen Böden um $s_H = 1,0$ mm aufgenommen werden.

Aus den gegebenen Formeln wurden für die Daten der einzelnen Probelastungen die Zusammendrückungsmodule des Bodens bei der Sohle und die durchschnittlichen inneren Reibungswinkel der Böden dem Mantel entlang berechnet.

Der Reibungswinkel hat eine ausreichend gute Übereinstimmung mit dem im Labor bestimmten Wert gezeigt. Der Beiwert des Sohlenwiderstands und so der Zusammendrückungsmodul ist aber im Fall von Rammpfählen in körnigen Böden wegen der grossen Verdichtungsarbeit sehr gross, im Fall von Bohr- oder ausgeschachteten Pfählen dagegen auffallend klein. Der Grund für den letzteren Fall kann wohl in der Pfahlausführungstechnologie gesucht werden, man muss nämlich an der Sohle - besonders im Fall der Anwesenheit von Grundwasser - mit einer beträchtlichen Auflockerung rechnen.

IV. Zusammenfassung

Die statistische Analyse der heimischen herkömmlichen Probelastungsdaten zeigt, dass der Spitzenwiderstand annähernd linear und die Mantelreibung nach einer Hyperbel von der Setzung abhängt. Diese Tatsache wurde von den Verfassern in dem vorliegenden Aufsatz mit der Bearbeitung der Probelastungsergebnisse von etwa 1000 mit unterschiedlichen Technologien ausgeführten Tiefgründungen /FRANKI-BENOTO-VUIS- und Ofenschlacken-Pfahl, Stöpfel- und Schlitzfundament/ bewiesen. Als Ergebnis der Analyse - in Funktion von E_s und ϕ - wird eine Methode zur Bestimmung der vorherigen Tragfähigkeit vorgeschlagen.

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Funktionstreuung der Annäherung von Probabelastungen 1.tabelle

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F	514	41,4	0,615	0,002	-2,988
Ba	7	60,9	0,604	-0,120	-1,347
Ba	26	86,0	0,538	0,480	-0,948
Bk	7	82,9	0,801	1,027	0,413
B	40	81,1	0,600	0,653	-0,178
Ke	40	15,7	0,627	0,645	-0,726
Ka	25	11,2	0,771	1,812	2,590
Kk	11	16,5	0,351	-0,293	-0,498
K	76	14,3	0,639	0,873	-0,135
CD	39	32,2	0,805	1,235	1,082
Se	8	106,1	0,625	0,036	-0,556
Sa	25	89,7	0,621	0,592	-0,264
Sk	15	79,9	0,609	0,892	-0,310
S	48	89,2	0,616	0,579	-0,447
Ve	6	16,8	0,509	-0,276	-0,500
Va	9	23,9	0,602	0,904	-0,020
Vk	17	20,6	0,569	0,635	-0,307
V	32	20,8	0,572	0,812	0,225
R	313	57,8	0,730	1,589	3,923
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Model Investigation for Pile Dynamics Problems

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ABSTRACT

The purpose of the present research work is to find a model that is the most suitable to describe the connection properties between the pile and the surrounding soil under dynamic and static loading, and to select a measuring technique and minimization algorithm that give good results. We use the stress waves induced by an impact load to determine the parameters of the model. The histories of the impact force and top velocity are calculated from measured strains in two cross-sections (or strain and acceleration in one cross-section) of the pile. We applied a variant of the generalized secant method stabilized by the singular value decomposition to perform the parameter identification.

INTRODUCTION

The idea of using stress wave measurements during impact loading to determine the behaviour of piles under static loading became popular from the 1960 s, after Smith presented a new model and calculation procedure [1]. Several large-scale research projects were set out to prove the applicability of the method in practice. Despite the encouraging results some problematical cases occurred indicating the necessity of further development of the model. One part of the studies tried to apply alterations on the equations determined by the assumed behaviour of soil-pile interaction. Some researchers attempted to change the linear material laws into non-linear

ones : Zandwijk et. al. applied non-linear damping [2], Nguyen et. al. used separate and independent damping parameters [3], Bredenberg and Brooms proposed bilinear elastic-plastic material law for point-bearing piles [4].

Experimental results show that one of the main shortage of this model is the incapability of following the effect of the rheological phenomena taking place in the surrounding soil during and after penetration, i.e. the increase of static bearing capacity and decrease of driving resistance in cohesive soils in consequence of pore water pressure dissipation, or increase of driving resistance due to the rearrangement of the grains in granular soils [5,6,7]. The "so called" relaxation phenomena usually observed during redriving come from the exchange of driving equipment cause no change in static bearing capacity, thus real and apparent relaxation have to be separated also [8].

Another significant source of errors is the uncertain knowledge of the driving equipment parameters [9].

The purpose of the present research work is to find a pile model that is suitable to handle some of the previously mentioned problems.

MODEL IMPROVEMENT PROCEDURE

As a first step we chose the pile model presented by Smith and made some changes to diminish the number of uncertain parameters we needed to start the calculation, and created a calculation procedure to identify the model due to the dynamic tests. With the help of the known parameters the static load test simulation and its comparison by the field test results can be done, upon what experience we may correct the initial model keeping our attention on the earlier mentioned phenomena. Here we summarise the main steps of the model improvement procedure :

1. Selection of the model and the measuring techniques.
2. Identification of the model according to the dynamic tests.
3. Simulate the static load test, and compare the results with field tests' data.
4. Correction of the model to have better correlation between

the behaviours of the model and the real structure.

CALCULATION PROCEDURE

We propose the "two-point strain measurement" technique [10] to eliminate the errors come from the uncertain properties of the driving equipment. Figure 1. shows the scheme of the calculation procedure.

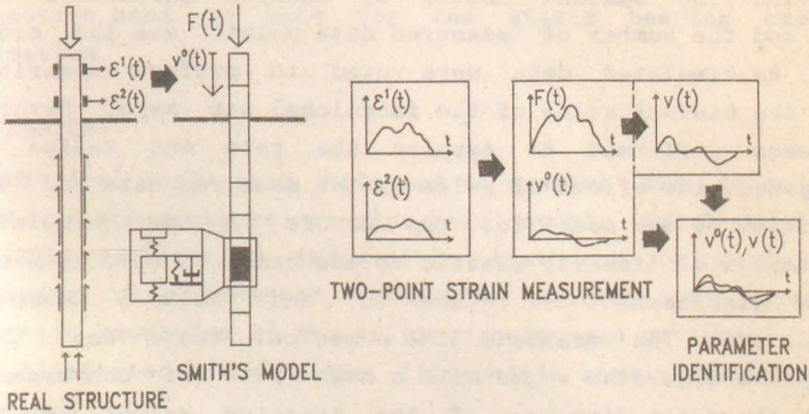


FIG. 1. Calculation Procedure

By measuring strains in two different cross-sections (or strain and acceleration in one cross-section) we can calculate the history of the force ($F(t)$) and velocity ($v^o(t)$) at the top of the pile. Applying the calculated impact force as an excitation on the model we can evaluate the top velocity response ($v(t)$) of the model for any combination of the parameters characterize the pile-soil interaction.

If we define a function as the difference between the measured and calculated histories of velocity ($f(t)=v^o(t)-v(t)$) then we may introduce a quadratic functional ($F=\int f^T(t)f(t)dt$) to be minimized i.e. the sum of the squared differences in the case of discrete measured data points ($\Sigma(v^o(t)-v(t))^2 \rightarrow \min.$). We used a minimization method based on a variant of the generalized secant method for which we applied the singular value decomposition to eliminate the instability of the method. The algorithm was completed by a

stochastical search that called forth of a significant increase in the rate of the convergence.

NUMERICAL TESTS

For numerical tests we considered simulated experiments that have been carried out by analysing a pile model with 9 elements embedded into the soil. We used the computed velocity response as fictitious "measured data" for parameter estimation. The maximal number of unknown parameters was $3 \times 9 = 27$ and the number of "measured data points" was the order of 100. As simulated data were used to perform numerical tests, the minimal value of the functional was zero. Several tests were performed to examine the rate and radius of convergence, the effect of noise in the measured data and the sensitivity of the model for the errors in the calculated parameters i.e. linearly elastic spring constant (c), maximal elastic displacement or quake (q) and "Smith's damping constant" (d). The results of the numerical tests show that the applied algorithm works with a high rate of convergence despite the great distance of the starting values of the parameters from the optimal solution. This fact coincides with the practical need of the problem as we have a very poor a priori knowledge of these values. Figure 2. shows an example evaluated from the numerical tests.

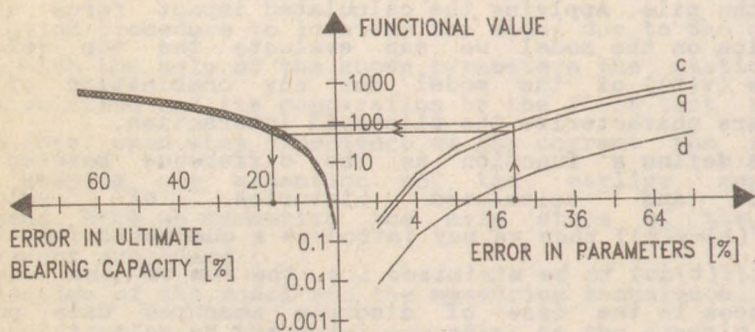


FIG. 2. Sensitivity of the Model

For example if we have 20% error in one parameter (c, q or

d) that means 17% error in the ultimate static resistance ($\Sigma c_i q_i$) of the model. These numerical tests also give us information about the relation between the range of the functional value and the errors in the estimated parameters. Controlling the singular values we found that the quake values can be determined unambiguously only for those pile elements where plastic deformations occur, and we have no information about this parameter for the rest of the pile. This fact shows the disadvantage of this model, as we cannot determine the parameters need to know for the static bearing capacity estimation.

DISCUSSION

Numerical tests were performed to examine the calculation procedure and a chosen dynamic pile model (Smith-model) to give some preliminary information that can be useful for analyzing real measured data. The errors come from the uncertain knowledge of the driving equipment or impact hammer were eliminated by using the "two-point strain measurement" technique.

The minimization procedure gives an automatic routine to check the limitations of the model by controlling the singular values, as it was shown in the case of the quake values. The results show that the applied model cannot be identified unless plastic displacements occur through the whole length of the pile. The proposed minimization algorithm can be used for any mathematical model to perform similar numerical tests.

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STRENGTHENING OF FOUNDATIONS OF SETTLED TANKS

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SUMMARY

600 cu.m fermenting tanks of a brewery have been supported on columns with block foundations. Soaked loess soil 6 to 8 m deep under the foundation plane has slumped when the tanks were first filled; the high load caused an important consolidation subsidence. 15 to 20 cm subsidence differences between neighbouring columns caused deformation of horizontal structural members. Foundations got reinforced by steel tube piles driven into the sandy gravel 15 to 16 m deep below the ground surface - without dismantling the tanks and the supporting steel structure.

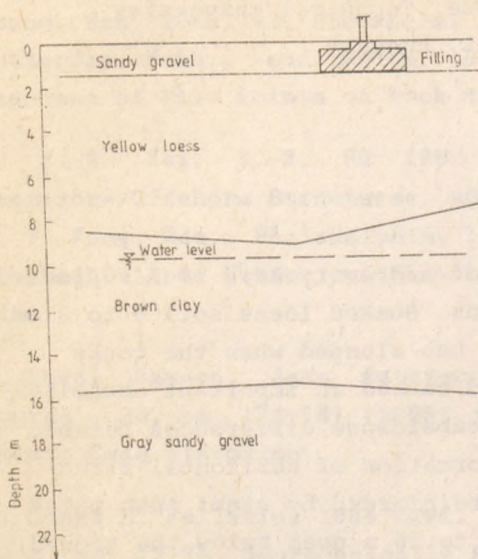
INTRODUCTION

Open-air fermenting tanks of a brewery in North Hungary, with 6 to 14 mm wall thickness, made of steel plates (ten tanks $\varnothing$ 6 m, 25,9 m high, 600 cu.m, and two ones of 140 cu.m) have been supported on base plates 4,8 m over ground level, joining the cylindrical shell by a short "skirt". The reinforced concrete slab with "perforations", with spacings equal to tank diameters, was supported on steel pillars spaced at 7,5 by 7,5 m. Pillars had r.c. block foundations of 3 by 3 m, and 4 by 4 m ground plans, depending on the load, - 1,8 m deep below the expected floor level. Foundation loads ranged from 1865 to 7060 kN. For the foundations, a work trench 33 by 27 m, 1,8 m deep has been excavated, then foundations were shuttered, reinforced, concreted. Meanwhile the work trench stood open, and the copious spring rainfall was seeping into the absorptive,

macroporous loess soil. After the end of a year of construction and tank installation works following foundation works, tanks were filled with water.

SOIL CONDITONS

A characteristics soil profile is shown in Fig.1.



Under the foundation plane there are 6 to 8 m thick macroporous slime fine sand and lean clay layers (loess) inclined to subsidence. Under this a 6.5 to 8.4 m thick clay layer can be found. The upper level of the thick and stable sandy gravel drifted soil ballast of the near river Hernád lies at the depth of 15.6 to 15.8 m. The maximal level of the ground water is at the depth of 5 to 6 m under the ground surface.

Figure 1. Soil profile

The shearing strength of the cohesive soils has been determined by the help of triaxial compressive experiment on undisturbed soil samples taken by boring. During tests change of pore water pressure has been measured.

For determination of deformation characteristics, compressive tests have been carried out. At the various load stages the samples have been inundated by water too. Surprisingly, no sudden subsidence has been experienced after inundation even on macroporous (loess) samples. On the other hand, under load between 90 and 140 kN/m² - irrespective of inundation - loess soil has indicated signs of subsidence (construction decay). Compression curves are to be seen in Fig.2. Coefficient of subsidence computable from here has changed between $i_r = 0 - 0.1$. We note here that at the first loading of the near silo building a subsidence (subsoil slump) of almost 30 cm has been measured.

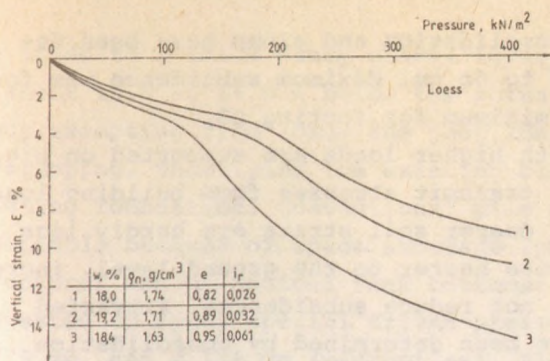


Figure 2. Compression curves

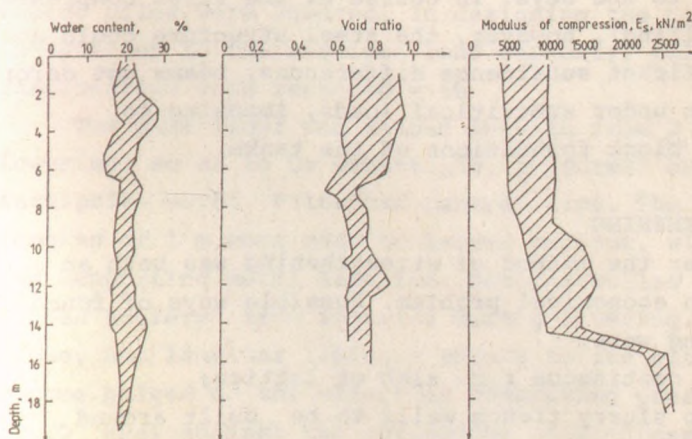


Figure 3. Variation of water content, void ratio and modulus of compression with depth

Fig. 3. shows the changes of the water content (w), void ratio (e) and modulus of elasticity (E_s) in function of depth.

INVESTIGATION OF THE SPREAD FOUNDATION

Values of foundation stresses below block foundations 3 by 3 m or 4 by 4 m in ground plan of columns spaced at 7,5 by 7,5 m supporting the twelve tanks have been plotted in Fig.4. Safety factors to soil failure were calculated to be 1,7 or higher for all the footings, offering, however, no safety to structural damage due to subsoil deformation.

Applying conventional subsidence determination methods - taking also interaction between adjacent footings into consideration - approximate subsidence values of separate

pillars due to primary consolidation and slump have been determined to vary from 17 to 60 cm. Maximum subsidence was found for footing E3, and the minimum for footing C3.

Although pillars with higher loads are supported on bigger footings, wider footings transmit stresses from building loads to greater depths. Since deeper soil strata are hardly less compressible than are those nearer to the ground level, increased foundation areas did not reduce subsidences. Expected subsidence durations have been determined by consolidation analyses. Accordingly, 90 % of subsidences were supposed to take place in 250 to 260 days. In course of the first compressive test (by water filling), however, the steel structure could not support the significant subsidence differences, beams got deformed, damaged even under subcritical loads, imposing to strengthen twenty block foundations of the tanks.

FOUNDATION STRENGTHENING

To decide over the method of strengthening was both an engineering and an economical problem. Possible ways of foundation strengthening were:

- incorporating a continuous r.c. slab or lattice;
- load transfer to slurry trench walls to be built around the construction;
- strengthening by piles.

Even incorporating a continuous r.c. spread foundation distributing fermenting tank loads all over the floor area of the construction, subsidences of the decimeter order would arise. Subsidence differences values are, of course, function of the slab stiffness. Doubts arose in relation to the safety of connection between block foundations and a slab ulteriorly incorporated between the pillars. A solution possibility emerged, such as to transfer tank loads to a continuous slurry trench wall along the building outlines reaching down to the sandy soil, by incorporating a trustworthy lattice superstructure - involving, however, serious technical difficulties and economicalness problems.

In final account, it has been suggested to strengthen the foundation by piles about 16 m long, transmitting build-

ing loads to a thick sandy gravel layer beginning at a depth of about 13,8 to 14,0 m below the actual plane of reference, about exempting from loads the poor load-bearing strata prone to slumping, underlying the existing block footings, and the existing foundation. Conventional pile driving methods were unsuitable because of space shortage (presence of supporting structures and of conical tank bottoms). Therefore it was suggested to apply special driven-pressed steel tube piles $\varnothing$ 219 mm, and $\varnothing$ 114 mm (micropiles). Along perimeters of existing block foundations, trenches 2,1 m deep were dug, whence piles were lowered. In designing the pile spacings, in addition to the required load capacity, pile pressing possibilities were reckoned with.

The pile liner was welded in-situ from 2 m lengths, during lowering, so as to be watertight, performed by the contracting enterprise DUTÉP. Piles had conical tips. The bottom perforated lengths of 1 m were made to become bulbous, with rubber membrane and supporting metal lamellae. Metal lamellae protected the "rubber hosiery" from injuries during lowering, at the same time, rubber and lamellar lining - thanks to its elasticity - could become bulged on the effect of concreting pressure (15 to $20 \cdot 10^2$ kPa) against the surrounding soil. Cross section increase may be assessed from the concrete volume pressed in. Piles were driven to a depth of 10 m, then pressed down. In pressing, counterweight was provided by water filling the tanks to 1/3 height, and by the existing steel structure. Pillar loads were transmitted to the piles by a r.c. slab uniting pile tops. Load capacities of single piles were determined in design according to the Caquot-Kerisel theory. For piles with 219 mm outer diameters, the load at failure was 906 kN, and the ultimate load 470 kN. Later in construction, correctness of computations has been checked by load tests. Load at failure of piles 15,6 m long, with 219 mm outer diameter, was 950 kN, and its ultimate value (at 6,7 mm subsidence) was 665 kN. In final account, foundations were strengthened by 196 micropiles $\varnothing$ 219 mm, and 32 ones $\varnothing$ 114 mm.

Subsidences of the twenty pillars were measured during water filling the tanks, and also thereafter. Maximum displacement (15 mm) was found for pillars E3 and E4 (Fig.4.)

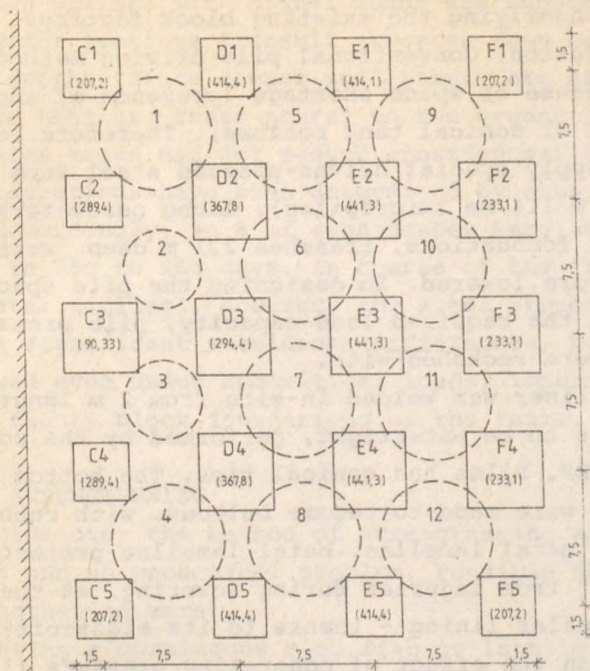


Figure 4. Stresses below foundations of pillars (kN/m^2)

Pillar E2 subsided by 14 mm; the others by less than 10 mm.

BEARING CAPACITY AND SETTLEMENTS OF BORED PILES WITH ENLARGED BASES IN COLLAPSIBLE SOILS

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The footings made of bored piles are greatly advantageous in urban areas because their making does not produce noise no vibrations that accompany pile driving activities. Enlarging of pile bases is broadly used to improve the bored pile bearing capacity.

It was assumed earlier that the pile tip resistance is directly proportional to the base area, and the specific resistance under the lower end get bigger as far as depth.

The test results and engineering practice show that the enlarged base using is far from the pile bearing capacity increasing always. The effectiveness of using enlargements bases depends on the soil physical-mechanical properties on the pile lower end level, the pile dimensions and relation between the pile length l and diameter d .

The bearing capacity and settlements of the piles with enlarged bases in collapsible soils were studied in Rostov region. Tested on the test sites were 22 long bored piles of 11 to 29.5 m long, 0.6 to 1.0 m dia with enlarged bases of diameters 1.6 and 2.2 m, and 10 piles without enlargement. Not less than two piles of the same dimensions were tested under static load at each test site. Taking into account the pile bearing capacity decreases considerably in collapsible soils by wetting, the piles were loaded with soil local wetting within the pile base.

The test sites are composed of loamy soil layers down to a depth of 35 m. Deposited under the pile tips is non-

collapsible loam with dry soil density of 1.52 to 1.68 t/m³, internal friction angle of 17 to 19 deg, cohesion of 15 to 30 kPa. The depth increase provokes only insignificant growth of the soil density. The strength properties of the soil depend on density and they grow little as far as deep.

The pile test results /1/, shown in Fig.1, are indicative of very small settlements even obtaining the ultimate loads and evident "disruptions" with an insignificant increase of the ultimate loads.

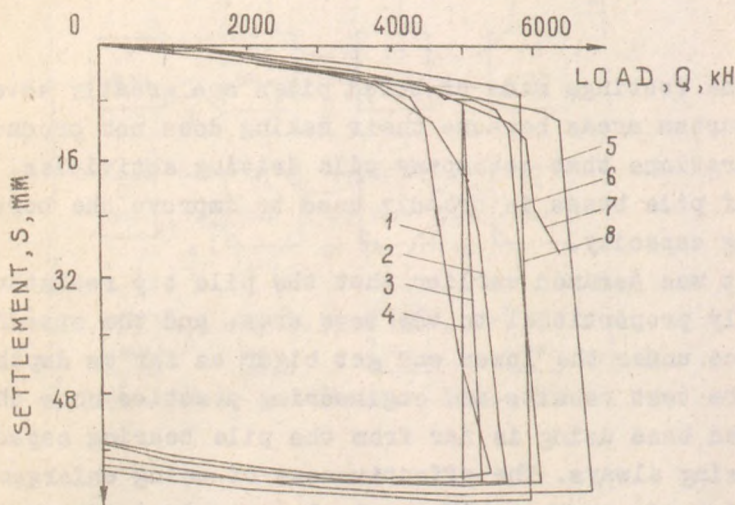


Fig.1. Graph of bored pile settlement S against loads Q .
 1,2- $d=1m$, $d_{eB}=2.2m$, $l=16m$; 3,4- $d=1m$, $D_{eB}=2.2m$, $l=18m$;
 5,6- $d=1m$, $l=25m$; 7,8- $d=1m$, $d_{eB}=2.2m$, $l=25m$.

The settlements of the piles with enlarged bases and those without enlarged ones before "disruption" were equal 3,5 to 10 mm. Such type of settlement-load curves demonstrates that in many cases the enlarged base of long piles practically is useless in many cases.

Given in Fig.2 are graph of ultimate load against the pile length. Sufficiently great spread of points is observed

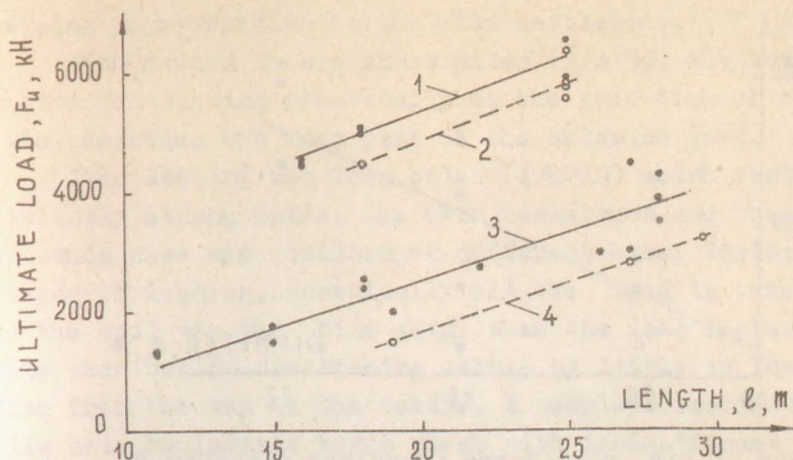


Fig.2. Graph of ultimate loads F_u applied to piles against their length l : 1- $d=1\text{m}$, $d_{eB}=2.2\text{m}$; 2- $d=1\text{m}$; 3- $d=0.6\text{m}$, $d_{eB}=1.6$; 4- $d=0.6\text{m}$.

due to the fact that the Figure unites the pile test results obtained at 7 test sites of Rostov region.

However, in spite of this circumstance, a relation between the ultimate load applied to the bored piles and their length is observed. The ultimate load applied to the pile increases in proportion to its length basically due to the skin growth. In this case, the pile tip resistance in homogeneous soils remains practically constant irrespective of the pile length therefore the part of the resistance under the enlarged base, being part of the total resistance, decreases as the pile length increases. As is shown in Fig.2, a bearing capacity increasing of pile 25 m long and 1 m dia due to installation of enlarged base of 2.2 m dia, is only 330 kN or 5.3% of the ultimate load applied to the pile.

Carried out side by side with the tests of bored piles with enlarged bases and without enlarged ones were tests of 7 tensometrical piles and piles with freed lower end under similar soil conditions, that permitted to specify the tip resistance and the skin one. As Fig.3 shows, the tip

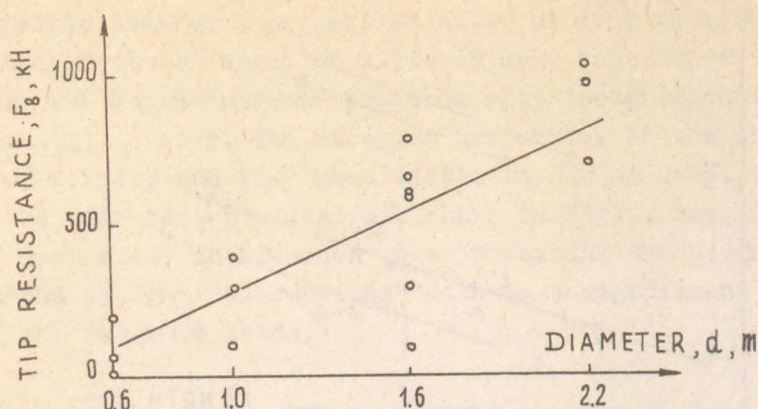


Fig. 3. Graph of tip resistance F_g against diameter d . resistance of a bored pile increases not in proportion to the base area, but in proportion to its diameter. This conclusion concerns both the usual long bored piles of 0.6 and 1.0 m dia and the piles with enlargements of 1.6 and 2.2 m dia.

In some cases the bearing capacity of bored piles with enlarged bases and of those without enlargements resulted similar, demonstrating that the widely spread opinion that the bored piles with enlarged bases are inefficient only due to the use of inadequate technology was not justified. In some piles, the hole bottoms were smoothed manually, but it did not influence on the test results. Due to the fact that the lower end of the long piles functions ineffectively, special tests with measurement of tensions under the lower end of piles of 18 m long and 1 m dia were carried out. The tests were performed under similar soil conditions. The lower end of the long bored piles of 18 m long, which cut through the low-density loams and rest on loams whose dry soil density is 1.68 t/m^3 , reacts right up to disruption not more than 300 to 320 kN that is less than 10% of the total load applied to the pile [2]. A sharp growth of resistance under the pile lower end up to 800-1000 kN occurred only after disruption, i.e. when the pile settlement had reached 70 mm. After the disruption, the efforts under the pile lower end were inc-

reasing in proportion to the pile settlement.

With regard to the short piles ($l/d < 5$), the lower end begins functioning practically at the same time as the pile skin, reacting the most part of the exterior load.

When loading the long piles ($l/d > 10$) which rest on insufficiently strong soils, the skin resistance and the tip resistance ones are realized at different time. During the first stages of loading, practically all the load is transmitted to the soil via the pile skin. When the load increases, the pile skin begins functioning little by little in the direction from the top to the bottom. A complete realization of the pile skin resistance takes place with insignificant (up to 10 mm) settlements which are basically elastic. Realization of the skin resistance over, the lower end begins functioning and subsequent growth of the pile resistance forces depends on the soil strength on the level of its lower end. If this strength is insufficient, the pile settling rate will increase sharply and the skin resistance will rather decrease. Owing to this assertion, the load part that is transferred to the lower end increases additionally. The longer the pile the more additional load is transmitted to the lower end when the skin resistance has exhausted. In case the soil strength under the lower end is insufficient, a compacted nucleus will be formed under the lower end when the pile disrupted. In this case, the resistance under the lower end increases, but the pile disruption does not stop, the pile settlement is increasing without stabilization and it reaches tens of cm /2/. Thus, ~~before~~ the disruption of pile, the resistance under the lower end is realized incompletely.

If deposited under the lower end of pile is strong and compact soil, the realization of the pile tip resistance forces takes place with small settlements and the tip part in the whole resistance increases.

In that way, the behaviour under a load and the bearing capacity of a long friction pile with enlarged base, which is installed in an insufficiently strong soil, will be determined

basically by the pile skin resistance. The part played by the lower end, including the enlarged base in the total bearing capacity of the long piles, that rest on insufficiently strong soil, results insignificant.

The results of the pile tests carried out in Zharinsk city (Fig.4) demonstrated that the soil strength at the pile base influences on the redistribution of efforts between the pile skin and the lower end, as well as on the bearing capacity of a pile with enlarged base.

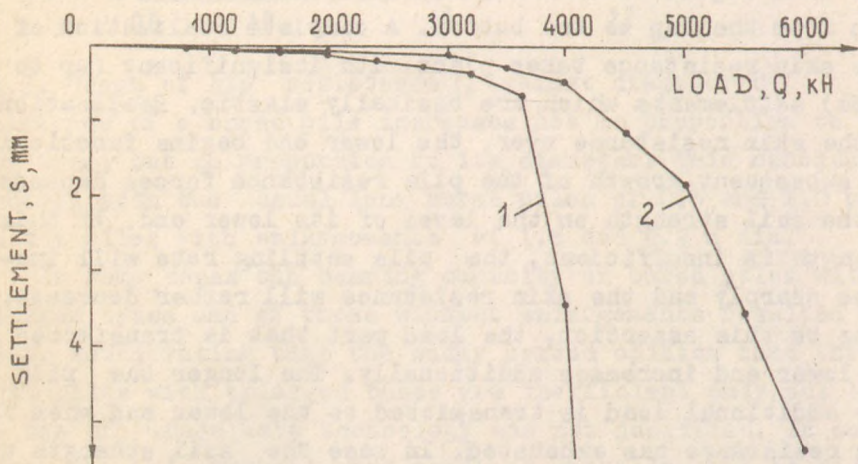


Fig.4. Graph of settlement S against load Q
1 - pile rests on loam; 2 - pile rests on sand.

Bored piles of similar dimensions, 18 m long and 1 m dia, with enlarged bases of 2.2 m dia, are installed inside the collapsible soil, represented by loess loam. The basic difference consists in conditions of these piles resting. Pile 1 does not cut through the loam layer and its base rests on the loams whose dry density is 1.55 t/m^3 , angle of internal friction is 22 deg and cohesion is 130 kPa. Pile 2 rests on sand whose angle of internal friction is 30 deg. The initial parts of the load-settlement graph for piles 1 and 2 practically coincide. Difference in pile functioning begins after the skin resistance has exhausted. If for pile 1 the abovementioned load

(3600 kN) practically corresponds to the beginning of the pile disruption, for pile 2 subsequent growth of the bearing capacity takes place up to 6000 kN owing to active inclusion of the enlarged base, that rest on the sand, into the total resistance.

The following formula, that is based on the theoretical grounds given in research work/3,4/, is proposed to be used for calculation of tip resistance, composed by loams or sand loams under the bored pile enlarged base when the latter begins functioning completely:

$$F_b = \pi \left[r^2 - (d_{eb}/2 \sin \varphi')^2 \right] C' \sin \varphi' \cos \varphi' + \pi r^2 \sin^2 \varphi' \bar{\sigma}_1',$$

where d_{eb} - pile enlarged base diameter, m;

C' - calculated value of soil cohesion under base, kPa;

φ' - calculated value of angle of internal friction of angle under the base, kPa;

r - radius of spherical surface, m

$$r = 0.5 d_{eb} (\operatorname{ctg} \varphi' + \operatorname{ctg} \alpha); \quad \alpha = 45^\circ - \varphi' - kC';$$

$k = 0.1$ - experimental coefficient, deg/kPa;

$\bar{\sigma}_1'$ - maximum principle stress, kPa;

$$\bar{\sigma}_1' = \frac{\bar{\sigma}_3 (1 + \sin \varphi') + 2 C' \cos \varphi'}{1 - \sin \varphi'}, \quad \bar{\sigma}_3' = \gamma h K_A,$$

γ - calculated value of unit weight of soil, kN/m³;

$h = 6$ m - for loams and sand loams;

K_A - coefficient of active earth pressure.

As a result of incomplete use of the resistance under enlarged base of the long bored piles, the performed full-scale investigation demonstrated a decrease of the limit resistance under the enlarged base down to 10-70% of F_b that was calculated according to the proposed formula.

The bases of four piles of 0.6 m dia, with enlarged bases of 1.6 m dia, 11 and 15 m long tested under the worst engineer-geological conditions among the test sites, practically did not give the resistance. These piles rested on loams whose dry density was of 1.45 to 1.52 t/m³. In regard to the most piles (16 among 22), the resistances under the enlarged

bases were realized only on 40 to 60%.

Conclusion: The inefficiency of making the enlarged bases of long piles that rest on insufficiently strong soil, is explained by their considerable skin resistance and small settlements right up to disruption, in consequence of which, the pile tip resistance is not realized.

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Behaviour of statically loaded piles embedded in a soil with cyclically varying effective stress

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It is well known that the behaviour of an axially loaded pile is influenced by the effective stress system in the surrounding soil mass. There are situations where the effective stress in the ground may change with time. In this study the pile applied axial load was constant throughout the loading test but the effective stress in the soil mass was cyclically repeated between an upper and a lower value and the behaviour of the statically loaded pile was recorded for both tension and compression tests.

The pile was 1000mm in length, 19mm diameter and was provided with five strain-gauged axial load cells to record the distribution of axial load in the pile. A special sand container, in the form of a pressure vessel, enabled a surcharge pressure to be applied to the sand surface via a special surface loading plate. This container was 380mm diameter by 960mm deep. The influence of the container size on the pile dimension was evaluated by Chan (1976) who kept the pile tip some 19 pile diameters from the apparatus base. The dry sand medium was placed by a pouring technique to give an average density of 1.56 Mg/m^3 with a void ratio of 0.17 and a relative density of 51%. The piles were slowly pushed into the loaded sand medium at 36mm/minute. A special pressure regulator enabled the surface surcharge pressure to be cycled between pre-set values, the duration of each square-wave loading cycle being 1 minute. Loads could be applied to the pile via a special double lever system to generate either compressive or tensile static forces in the pile and to carry out load tests on the pile to failure. A very high degree of test result repeatability was achieved to within 5%. The medium sand gave an angle of internal friction, ϕ' , of 39° in drained triaxial testing.

The variation of the load distribution in the pile, the pile top movements and the rate of movement with the number of surcharge cycles was studied. The pile depth of embedment was 30 diameters, the static load was 70% of the ultimate load for a surface surcharge of 100 kN/m^2 . The cyclic surcharge varied between 50 kN/m^2 and 100 kN/m^2 . Typical test data are presented in Fig. 1. At the initial stage of the test the pile moved "rapidly" but at a decreasing rate with increase in number of surcharge cycles. It is believed that the pile movement comprises two components, elastic and irreversible, the mechanics of movement being similar to those postulated by Ko and Scott (1967) for triaxial loading of sands.

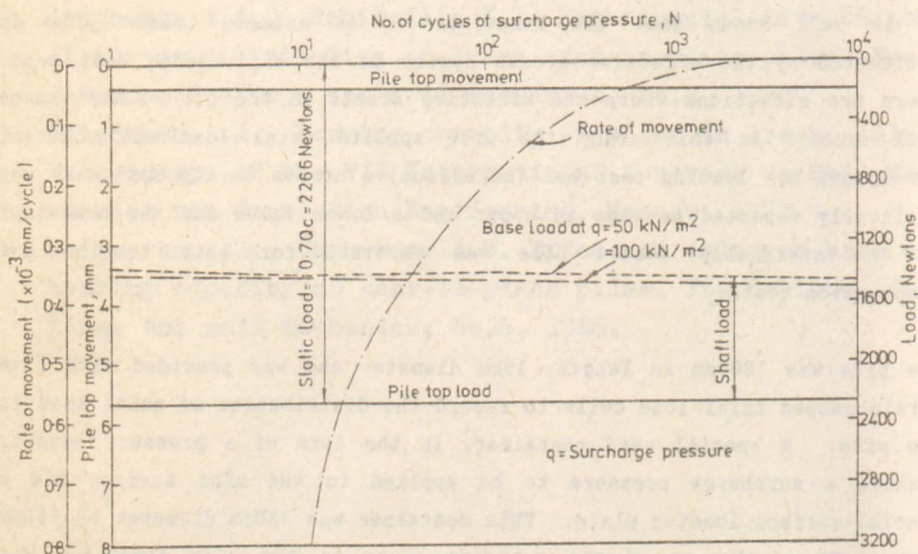


Figure 1. Pile under static compressive loading : variation of shaft and base load and movement of pile top with cycles of surcharge pressure application

The distribution of the axial loads for three of the load cycles are shown in Fig. 2. The axial load, at any location, increased with number of surcharge cycles increase. After a large number of cycles there is evidence of negative friction along the upper parts of the pile, the axial load in the upper third of the pile exceeding the applied load.

At the end of the test, a test to failure (statically applied) was performed with the surcharge equal to 100 kN/m^2 and repeated for static

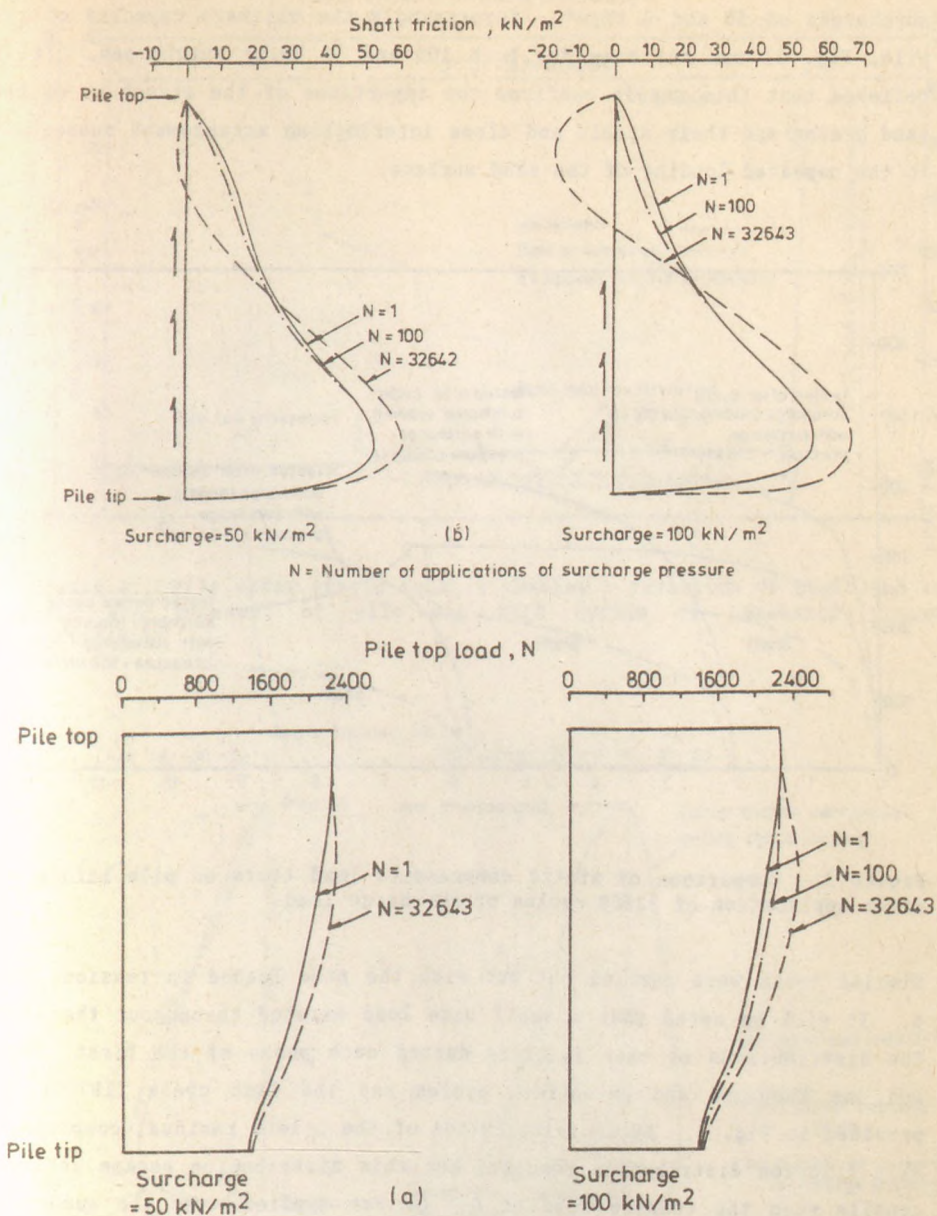


Figure 2. Distribution of (a) axial load and (b) shaft friction for pile under static compressive loading for different numbers of cyclic surcharge pressure application

surcharges of 50 and 0 kN/m². Surprisingly the ultimate capacity of the pile, Fig. 3, was the same for both 100 and 50 kN/m² surcharges. It is believed that this result confirms the importance of the structure of the sand grains and their stable and close interlocking arrangement subsequent to the repeated loading of the sand surface.

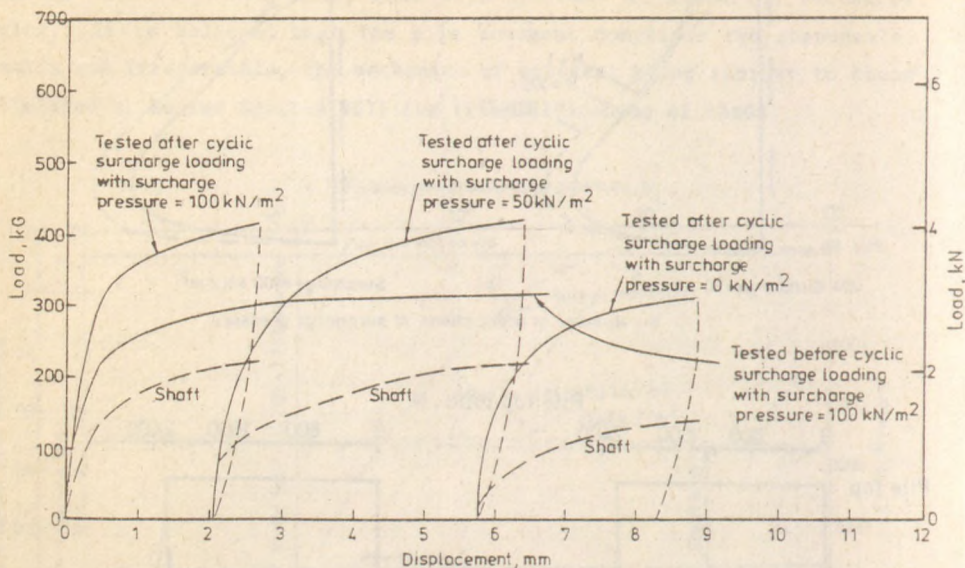


Figure 3. Comparison of static compressive load tests on pile before and after application of 32600 cycles of surcharge load.

Similar tests were carried out but with the pile loaded in tension, Fig. 4. It will be noted that a small base load existed throughout the test. The distributions of skin friction during each phase of the first, tenth and one thousand and seventieth cycles and the last cycle (18300) are provided in Fig. 5. After installation of the pile a residual compressive skin friction distribution resulted but this distribution became entirely tensile when the tensile load of $0.7 Q_t$ was applied. As the number of cycles of surcharge changed the negative friction along the upper part of the pile increased whilst along the lower part it decreased. The results of a static pulling test at the end of the cyclic surcharge test at 100 kN/m² and then at 50 kN/m² are compared with the virgin load test in Fig. 6. The pulling resistance of the pile had increased after it had been tested under cyclic loading but the increases were very small.

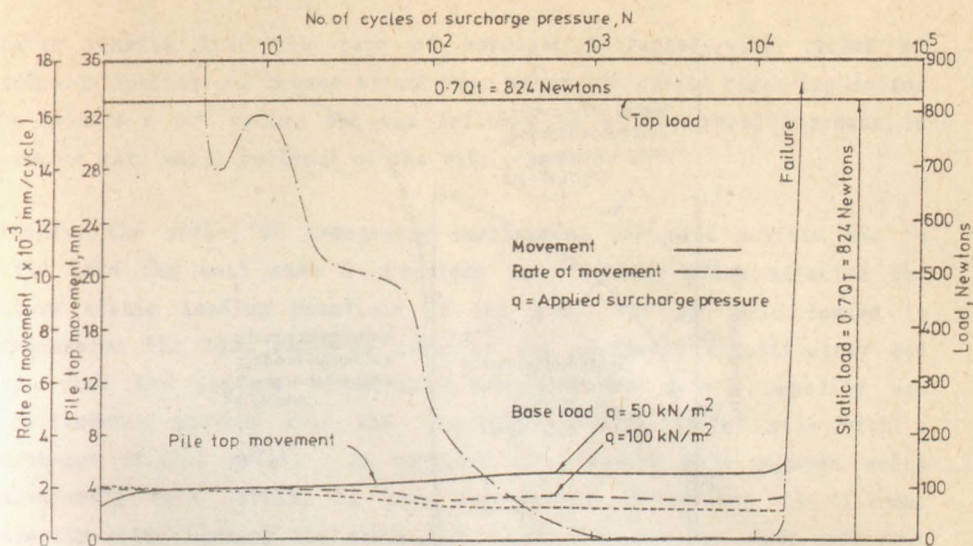


Figure 4. Pile under static tensile loading : variation of shaft and base load and movement of pile top with cycles of surcharge pressure application

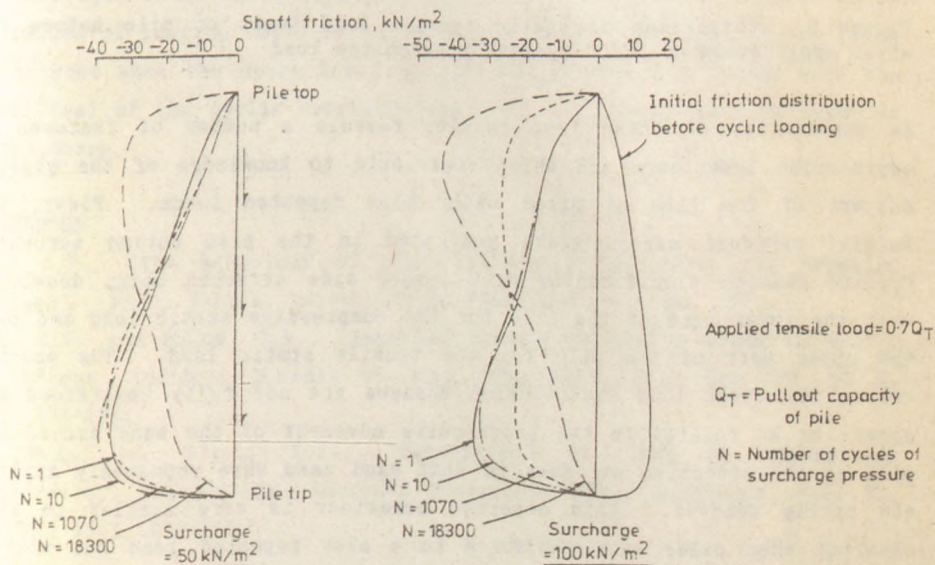


Figure 5. Distribution of shaft friction for pile under static tensile loading for different numbers of cyclic surcharge pressure application

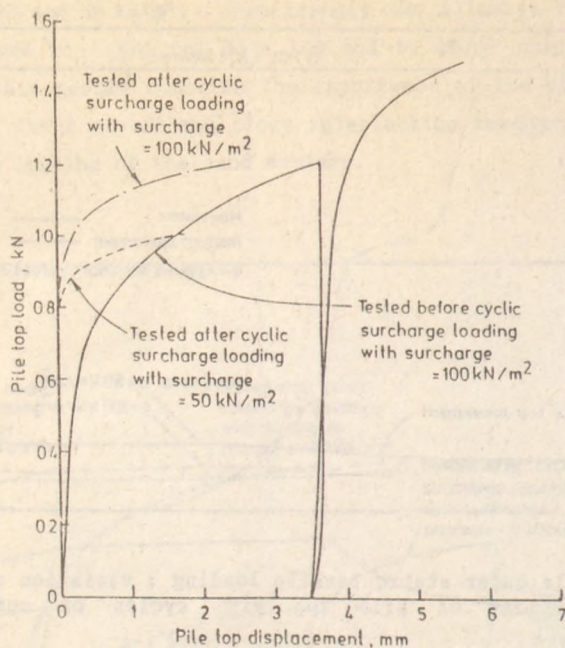


Figure 6. Comparison of static tensile load tests on pile before and after application of 18300 cycles of surcharge load

An examination of these test results reveals a number of features of engineering importance and which contribute to knowledge of the general subject of the life of piles under slow repeated loads. First, the initial residual stress state generated in the pile during surcharge cycling changed significantly with larger side stresses being developed over the lower part of the pile for the compressive static load and over the upper part of the pile for the tensile static load. The precise causes for these load distribution changes are not fully understood but appear to be related to the progressive movement of the sand around the pile as the effective stresses in that sand mass were repeatedly applied and partly removed. This observed behaviour is very similar to that observed when piles were subjected to a slow repeated load but with a static surcharge pressure, Chan and Hanna (1980). Secondly, for the pile under compression loading the pile top movement decreased with increase in the number of surcharge applications. In contrast with the pile under a

static tensile load the rate of movement decreased with cycles of surcharge loading and became almost zero after 10^3 cycles remaining so for the next 2×10^3 cycles but was followed by a very rapid increase in movement rate until pull-out of the pile occurred.

Thirdly, the effect of repeatedly surcharging the sand surface was to build into the soil mass a prestress or stiffness which affected the future static loading behaviour of the pile. For the pile loaded in compression its load carrying capacity was increased significantly and even when the surface surcharge was halved the pile's capacity was significantly greater than that for an initially tested pile with a surcharge of 100 kN/m^2 . In contrast the tension pile behaved quite differently with little, if any, significant improvement in pull-out capacity after cycling the surcharge. Quite clearly the very different patterns of load distribution along the piles has affected the capacity of the pile to resist applied load.

Other work, not reported here, has considered cases where the applied pile load was cycled and the surcharge pressure was also cycled. The peaks of the cycles of loading could be in or out of phase. The life of the piles was longest when the upper level of repeated load was in phase with the upper level of the cyclic surcharge pressure. This work will be reported on elsewhere.

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STRESS CHANGES IN SOIL AFTER PENETRATION

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1. INTRODUCTION

Joined model has been suggested for the description of the stress changes after penetration consisting of a one dimensional axysymmetrical coupled consolidation part-model and a phenomenological relaxation part model /Imre, 1989/.

Experimental results of rheological type CPT test¹ are compared with numerical solutions of the joined model produced by a parameter analysis herein.

The comparison is based on the following considerations.

/a/ The local side friction can be written as follows:

$$f_s(t) = \sigma'_r(t, r_0) k(t) + c_a(t) \quad /1/$$

where f_s : local side friction, σ'_r : radial effective stress, t : time, k : load transfer coefficient, c_a adhesion, r_0 : radius of CPT. Equation /1/ expresses that local side friction is /not uniquely/ related to the effective radial stress acting on the shaft.

/b/ Two dimensional consolidation part-model ought to be applied to describe the variation of the cone resistance /the total normal stress acting on the tip/. Qualitative similitude can be assumed between the radial total stress acting on the

¹Rheological CPT test comprises that local side friction and cone resistance are measured while the continuous pushing of the penetrometer is stopped and the rod is clamped.

shaft and cone resistance /i.e. solutions of one and two dimensional consolidation part-models/ since the physical model is the same in both cases.

2. STATISTICAL ANALYSIS OF RHEOLOGICAL TYPE CPT TEST DATA

Statistical evaluation of rheological type CPT test data presented earlier /Imre et al 1989/ is expanded herein into a form favourable for the comparison.

About 100 rheological CPT test records of ten sites were divided into groups considering layers, plasticity and horizontal inhomogeneity /Table 1/. Average normalized cone resistance curves and local side friction curves for groups in most cases show a fast immediate stress drop the duration of which is neglected in Figures 1 and 2 followed by a time dependent stress variation period.

Cone resistance-time curves can be characterized by a monotonic stress decrease irrespective of soil type.

Local shaft friction-time curves show different tendencies for different types of soils. Local side friction has increasing

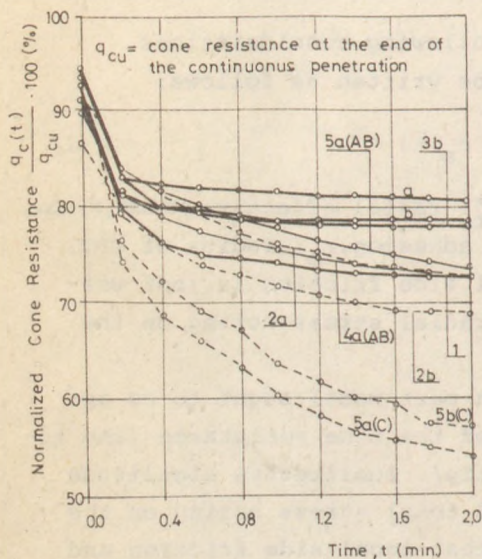


Figure 1.

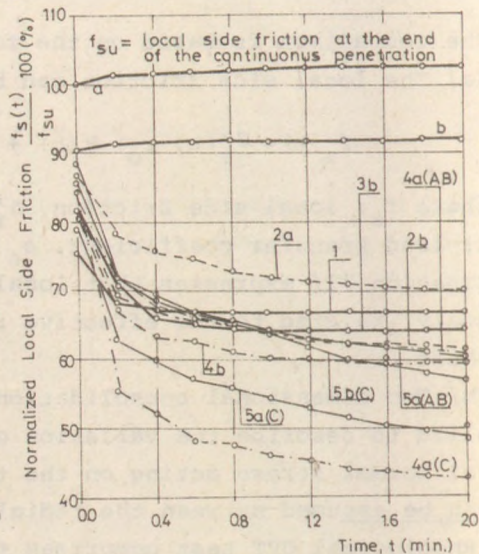


Figure 2.

tendency in sand -without immediate stress drop in the case of the coarser sand /group "a"/- and decreasing tendency in clay.

Relationships between sounding parameter $\Delta f_{s2}^2 - I_p$ and $\dot{\gamma}_c^3 - I_p$ are presented in Figures 3 and 4. Strong correlation can be observed if the plasticity index is less than a certain value between $I_p = 20\%$ and $I_p = 30\%$ in both cases. Deviation from the trend is encountered in the case of tests made at layer boundaries and in layer 2.

Table 1

Sign	Soil type	Category limit	OCR	Sign	Soil type	Category limit	OCR
5a/AB/	clay	$25 < I_p$	1.03	3b	silt	$10 < I_p < 20$	1.43
5a/C/	clay	$25 < I_p$	1.03	2a	clay	$25 < I_p$	1.77
5b/C/	clay	$20 < I_p < 25$	1.03	2b	clay	$20 < I_p < 25$	1.77
4a/AB/	clay	$25 < I_p$	1.31	1	Mo	$5 < I_p < 10$	3.32
4b/AB/	clay	$20 < I_p < 25$	1.31	a	sand	$0.1 \text{ mm} < d_{30}$	-
				b	sand	$0.1 \text{ mm} > d_{30}$	-

3. PARAMETER ANALYSIS, RESULTS

The aim of the parameter analysis is to produce radial effective and total stress-time curves for different types of soils. Solutions of the joined model depend on the initial condition coefficient of consolidation and coefficient of relaxation.

Only coefficient of consolidation $/c_v/$ -within a range of 10^{-1} - $10^{-10} \text{ m}^2/\text{s}$ - and coefficient of relaxation $/s/$ -within a range of 0.0-0.1- have been varied. Results are shown in Figure 5.

² The value of the Δf_{s2} parameter is equal to the decrement in local side friction from the beginning of the time dependent stress variation period until 120 s elapsed time.

³ $\dot{\gamma}_c = -(\sigma_o - \sigma_v)[1/\dot{\sigma}(t=0)]$; where $\sigma_o, \sigma_v, \dot{\sigma}$: the initial, final value and first time derivative of the fitted curve. The $\dot{\gamma}_c$ parameter has been determined by fitting of the Poynting-Thomson equation on data of time dependent stress variation period of cone resistance-time curves.

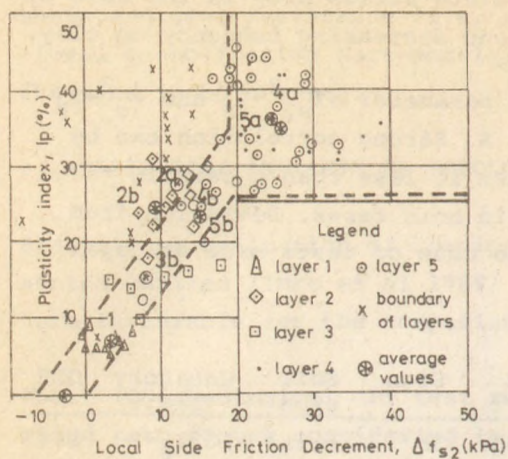


Figure 3.

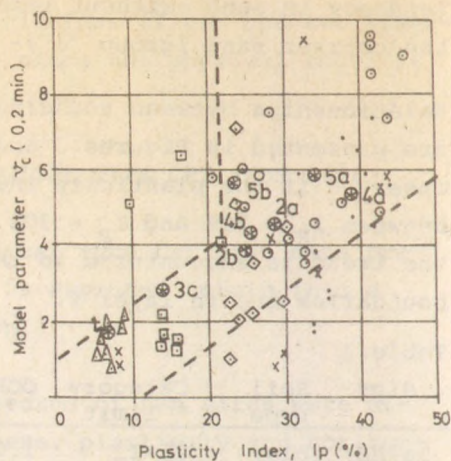


Figure 4.

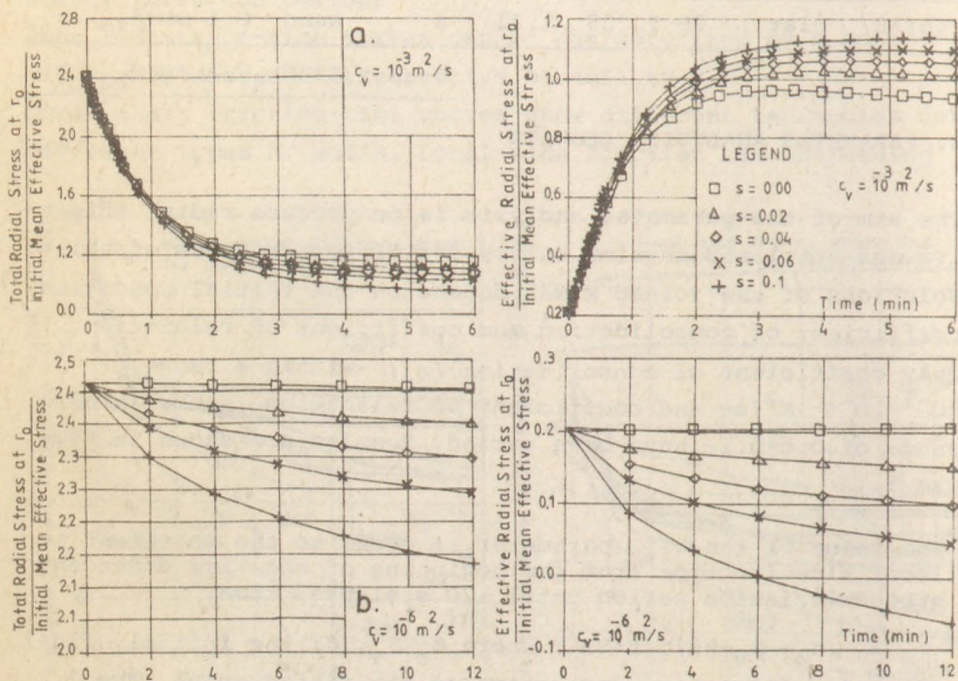


Figure 5.

Two parameters has been interpreted similarly to sounding parameters Δf_{s2} and γ_c and are represented in the function of c_v in Figures 6 and 7. The Δf_{s2} parameter has been approximated by the radial effective stress decrement after 120 s elapsed time. The γ_c parameter has been approximated by the $(-1/\dot{\epsilon}_r, t=0, r_0)$ term which was graphically determined on the radial total stress-time diagrams.

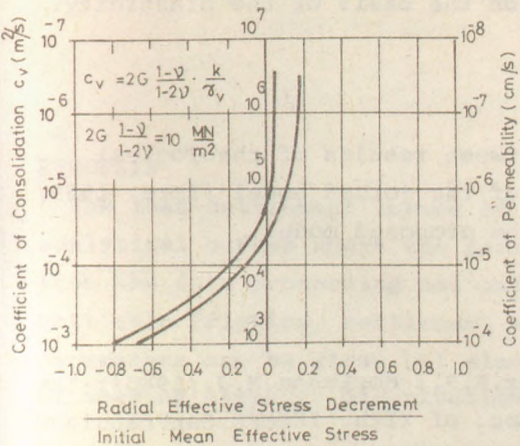


Figure 6.

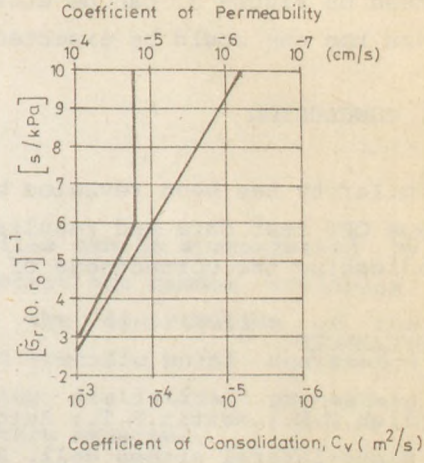


Figure 7.

4. DISCUSSION

It can be seen on Figure 5 that the radial total stress is with monotonic decreasing tendency irrespective of values of coefficient of consolidation and coefficient of relaxation. This result is in accordance with cone resistance curves shown in Figure 1.

It can be seen on Figure 5 that the radial effective stress is increasing if $c_v > 10^{-3} \text{ m}^2/\text{s}$ /sandy soils/ and decreasing if $c_v < 10^{-5} \text{ m}^2/\text{s}$ /clayey soils/, just like in the case of local side friction curves /Fig.2./.

Consolidation entails a very fast radial stress increase in the case of highly permeable soils. This fact explains the experience that no immediate stress drop occur in the case of rheological CPT curves for the coarser sand /group "a"/.

Diagrams $\sigma_c - I_p$ and $[-1/\sigma_r(t=0, r_0) - c_v] / \text{Figs. 4 and 7/}$ show good agreement. The scatter of values if $c_v < 8 \times 10^{-5} \text{ m}^2/\text{s}$ is caused by the variation range of the coefficient of relaxation $/s/$. The difference in the amount of stress decrease regarding diagrams $\Delta f_{s2} - I_p$ /Fig. 3/ and $[\sigma_r'(t=0, r_0) - \sigma_r'(t=120 \text{ s}, r_0)] - c_v$ /Fig. 6/ is caused by the error in the initial condition /Imre 1990/. The deviation of tests of layer 2 and at layer boundaries from the trend on Figure 3. can be attributed to a higher permeability than the one could be expected on the basis of the plasticity.⁴

5. CONCLUSION

Similarity has been revealed between results of rheological type CPT test data and results of the joined model /Imre, 1989/ indicating the correctness of the proposed model.

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⁴In the case of layer 2 the desiccation -which resulted in higher OCR /Table 1/- probably entailed the development of a secondary structure with fissures.

Statistical processing data of tests on driven piles

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SYNOPSIS

The load-settlement curves of piles can be approximated by analytical curves where the parameters are random variables. From the data processing not only the distribution of the unit skin friction, settlement and specific point resistance parameters can be given but also the statistical parameters of bearing capacity distribution were examined.

INTRODUCTION

Recently more and more attempts are made to take into account input parameters of civil engineering calculation as random variables. For example, in case of foundations, the changes of the soil parameters, the quality and bearing capacity of footings are all matters of stochastic phenomena. Thus making use of statistical methods for pile bearing capacity calculation seems to be quite obvious.

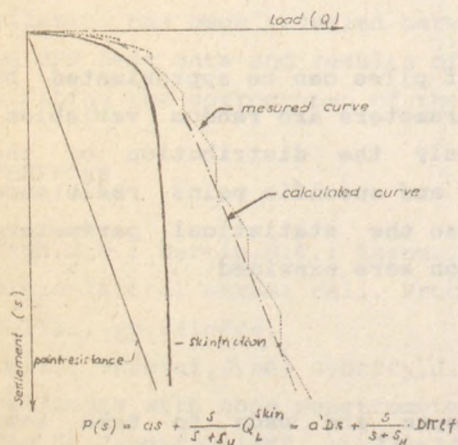
CALCULATION MODEL

The approximation of the curves derived from pile load tests was made by the help of a quite simple model. As it is the case usually for related theories in case of this model the skin friction was also calculated as point resistance.

Generally skin friction is approximated by curves that either have one limit value (as a hyperbola or a vertical

line) or by ones that characterize such circumstances when after a certain load the settlement increases very quickly (for example parabola, exponential function curve or a curve is composed from lines having increasing angles of inclination) [1].

The variation of point resistance is calculated in numerous ways for example using hyperbola, line or other methods [3]. Approaching by line has the advantage that the physical meaning is related to one of the traditional soil parameters --namely to the coefficient of compressibility [1],[2]. This of course --concluded from the definition-- presents real values only within determined settlement limits.



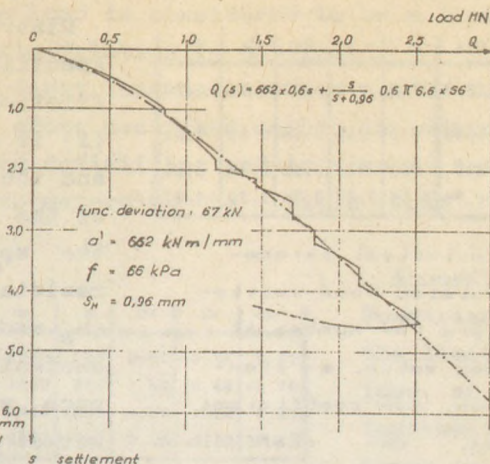
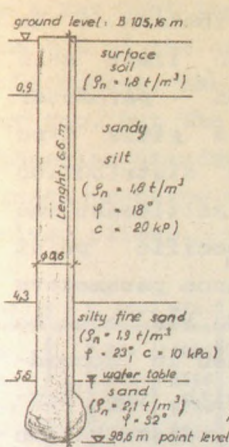
On the basis of the above mentioned reasons the load-settlement curve of pile was taken into account by analytical curves supposing that the skin friction increases according to a hyperbola but the increase of the point resistance is directly proportional to the load [5].

Fig. 1 - Measured and calculated curves of pile

LOAD-SETTLEMENT PARAMETERS

To get the necessary parameters 91 pile load test curves were approximated by the above mentioned formula. The average length of the examined Franki piles was 7.2 m the diameters ranged between 0.4 and 0.6 m. The soil below the pile point was sand or sandy gravel and different but generally silty layers surrounded along the skin surfaces.

Load-settlement curves of those tests were worked up only where the maximum settlement was between 5% and 2% of the pile diameter --that is 2.5 and 12 mm-- because realistic



results could be gained only in those cases. This situation was evidently due to the relationship between the point resistance and the modulus of compressibility.

Fig 2 - Boring profile, measured and calculated load-settlement curves of a typical test pile

The function deviation, it means average difference between the calculated and measured curves shows that the approximation is acceptable. Its distribution is normal distribution with a mean value of

37.4 kN and standard deviation: 21.0 kN.

The value of distribution equals the regarding accuracy of hydraulic jack manometers that were used for determining the

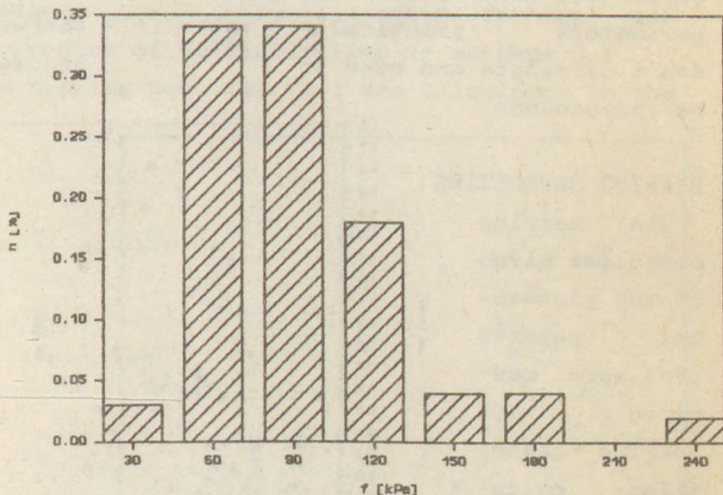


Fig 3 - Histogram of unit skin friction

load in the time of a lot of those tests, that was 10-20 years ago.

	n	min	max	m	s	Cv	Cs	Ck
l [m]		3.9	12.3	7.3	2.0	0.27	0.72	-0.08
a [kN/ /m/mm]	91	27	1583	570	249	0.44	0.63	1.91
f [kPa]		21	239	78	41	0.52	1.74	3.90
s _H [mm]		0.01	1.74	0.54	0.42	0.78	1.20	0.92

Table I - Statistical parameters of calculated load-settlement parameters (n- number of tests, m- mean value, s- standard deviation, Cv- coefficient of variation, Cs- coefficient of skewness, Ck- kurtosis)

system [4]. (The specific point resistance parameter is equal to the total point resistance parameter which is divided by the diameter as it can be used on the basis of some theories.) The received parameters were real. The parameters practically don't correlate and seem to be independent.

$$R = \begin{vmatrix} 1 & -0.205 & -0.169 \\ -0.205 & 1 & 0.181 \\ -0.169 & 0.181 & 1 \end{vmatrix} \begin{matrix} a' \\ f \\ s_H \end{matrix}$$

Table II - Correlation matrix of used parameters

BEARING CAPACITIES

The bearing capacities given in the geotechnical reports (Ph) were compared with bearing capacities calculated different way. From these curves the limit loads can be calculated

Distribution of the specific limit skin friction parameter (f) is a gamma type and the distribution of the settlement and the specific point resistance parameters (s_H and a') can be estimated in accordance with the log-normal distribution by the help of Pearson's co-ordinate

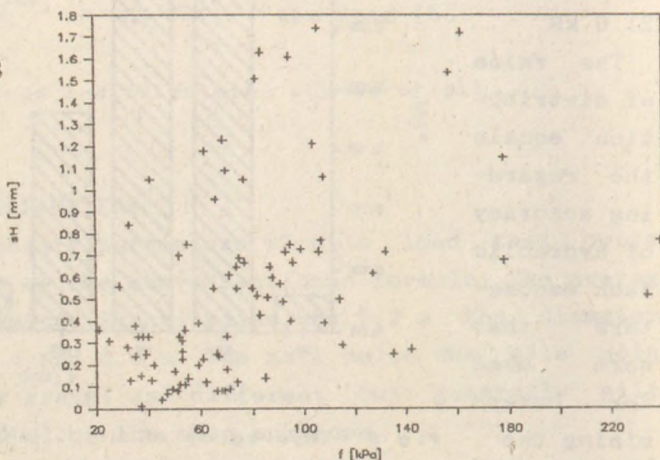


Fig. 4 - Specific settlement parameter versus the limit skin friction (n=91)

on a way that the load is considered to be a limit load where the 90% of the skin friction is mobilized.

Of course, the limit load calculation can be done on other way too, but the above described method is adauquate to the Hungarian Standard definitions and is almost the same with French practice to determine the "yield load" in case of driven piles.

[kN]	n	min	max	m	s	Cv	Cs	Ck
Ph	61	793	2030	1568	283	0.17	-0.75	0.81
Qa		348	3368	1508	790	0.52	0.48	-0.78
Pck		320	9860	2443	1966	0.80	1.78	3.48
Qs		720	3250	1877	492	0.26	-0.13	0.18

Table III - Statistical parameters of load bearing capacities <bc> determined on different way (Ph- bc in geotechnical reports, Qa- bc by Hungarian Standard, Pck- bc by static formula, QS- load when $s(Q)=0.005D$)

the value of its product of multiplication is maximum 0,7.

In our case the bearing capacity (Qa) was calculated in the above way. In geotechnical reports the bearing capacity was calculated by "Caquot-Kérisel" static formula, too (Pck). In Table III there are also the statistical parameters of another load, when the

According to the Hungarian Standard the limit load is the load at which the load-settlement curve keeps to a vertical or to an inclined tangent. The bearing capacity to be calculated by such method that the limit load has to multiplied by three decreasing safety factors and

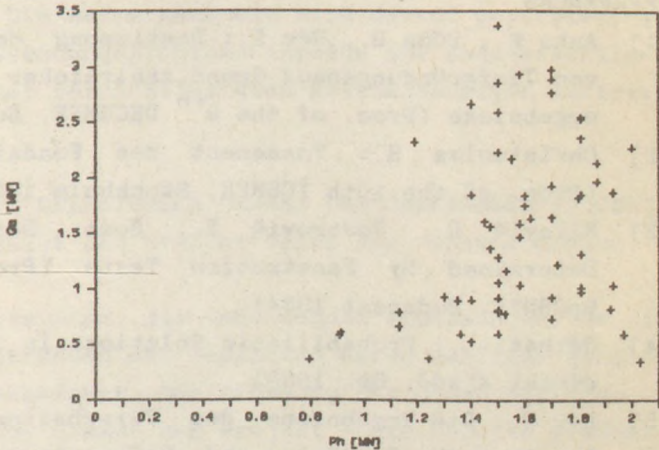


Fig. 5 - Calculated bc versus bc given in geotechnical reports

settlement is equal to the 0.5% of the pile diameter (Q_s).

It can be seen that though the mean value of given and calculated bearing capacities are quite similar and the coefficient of variation of Q_a corresponds with the C_v values of load-settlement parameters but the standard deviation of P_h is too small. It shows that the bearing capacities in the examined geotechnical reports were "quite independent" of the real load-settlement curves.

CONCLUSIONS

- Average difference between the calculated and measured curves is less than the accuracy of the load measurement.
- The curves calculated by the help of the used model fitted well to the measured load-settlement curves.
- The received parameters were real, and practically don't correlate.
- The calculated bearing capacities (Q_a) correspond with the C_v of load-settlement parameters but the bearing capacities given in geotechnical reports were uniformed.

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VERSTÄRKUNG DER GRÜNDUNG EINER RAHMENKONSTRUKTION IM LAUFE DER AUSFÜHRUNG

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EINLEITUNG

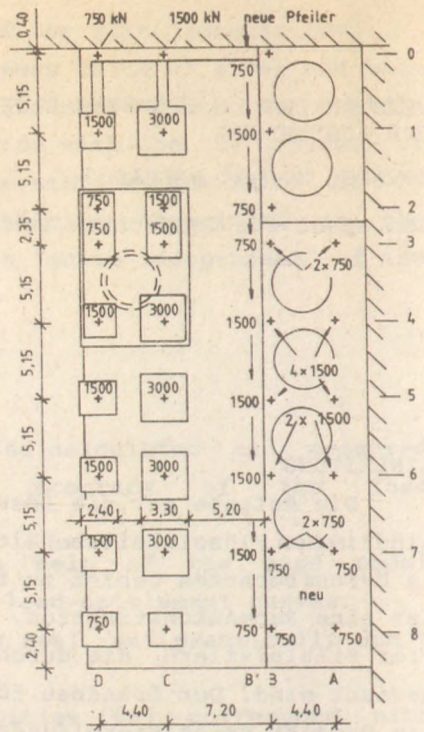
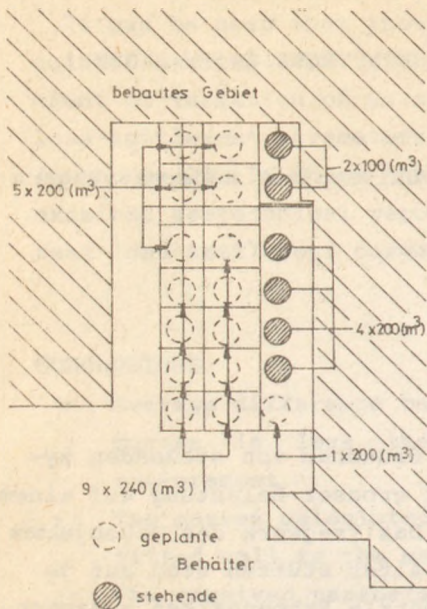
Die Aufgabe war, die Lösung für Grundbau von stehenden zylindrischen Flüssigkeitsbehältern mit grosser Belastung auf einem in U-Form bebauten Gebiet zu finden. Das Tragwerk des Bauobjektes ist eine Rahmenkonstruktion, die Behälter stützen sich auf je vier Stahlpfeilern, die durch horizontale Rahmenbalken zusammengefasst sind. Der Grundbau für Pfeiler wurde gemäss dem originalen Projekt durch Einzelfundamente aus Stahlbeton ausgeführt, bzw. ein Teil der Belastungen der Träger sollte der Grundplatte der bereits vorhandenen alten Behälter übergeben werden.

In der Abhandlung werden Fragen der Tragfähigkeit, besonders der Verformung und Setzung der bereits ausgeführten Fundamente behandelt. Die Aufmerksamkeit wird darauf gerichtet, was für unangenehme Gründungsprobleme infolge der Bodenerschliessungsmangel und der vernachlässigten Setzungsanalyse auftreten können.

GRUNDBAU UND SEINE BELASTUNGEN GEMÄSS DEM ORIGINALEN PROJEKT

Das Bild 1 zeigt den Standortsplan der vorgenommenen Behälter.

Das Gründungssystem, die umfassenden Abmessungen und die massgebenden Belastungen der geplanten neuen Behälter sind im Bild 2 zusammengefasst. Die Gründung der neuen äusseren Behälterreihe wurde gemäss dem Projekt - wie bereits erwähnt - durch Einzelfundamente gelöst. Die neuen Behälter in der mittleren Reihe stützen sich - gemäss dem Projekt - einerseits auf den dazwischen befindlichen Einzelfundamenten, andererseits -



durch Vermittlung der Stahlsäulen der bereits stehenden Behälter - auf ihren Grundplatten. Die Einzelfundamente aus Stahlbeton mit den projektierten Abmessungen wurden gebaut. Bei der Ausführung wurde ein alter, aus Ziegel gemauerter, aufgefüllter Brunnen mit einem grossen Durchmesser gefunden. Die Mauer des aufgefüllten Brunnens wurde bis zu einer Tiefe von 70 cm unter der Gründungsebene abgerissen. Unter den durch den Brunnen betroffenen 6 Pfeilern wurde eine 70 cm dicke Stahlbetonplatte mit einer Grundfläche von cca. 10,55 - 7,45 m gefertigt, die das Loch des aufgefüllten Brunnens deckt und überbrückt. Die Blockfundamente der Pfeiler wurden auf dieser Platte gebaut.

Dem Obigen ist es zu entnehmen, dass das zu erwartende Verhalten des ausgeführten Gründungssystems sowohl bezüglich Tragfähigkeit als auch Senkungen besonders heterogen ist.

BODENSCHICHTEN, FESTIGKEITS- UND DEFORMATIONSKENNWERTE

Unterhalb des geregelten Terrainniveaus ist es eine lockere, gemischte, künstliche Auffüllung von einer Dicke von 1,70 -

2,35 m zu finden. In einer Tiefe von cca. 5 Metern unter der originalen Geländeoberfläche befinden sich gehängeschüttige Ton- und Schluff-Schichten. Darunter kommen körnige Schichten, Sand und Kiessand (mit Schluff) in einer Dicke von etwa 3 m. Die untere erschlossene Schicht war grauer fetter Ton von einer grösseren Dicke. Aufgrund von Bohrungen und laboratorischen Untersuchungen wurde ein Idealprofil zu Tragfähigkeits- und Setzungsprüfungen zusammengestellt, in dem die zu dem Berechnungen nötigen bodenphysikalischen Kennwerte angegeben sind (Bild 3).

PRÜFUNG DES AUSGEFÜHRTEN GRÜNDUNGSSYSTEMS

Bei der Prüfung war es festzustellen, dass die von dem Entwerfer durchgeführten Erschliessungen die Bodenverhältnisse unter der Gründung - infolge des bebauten Geländes - nur annähernd klarlegen konnten. In der vorgeschlagenen Gründungsebene war bei der Ausführung immer noch Auffüllung gefunden. Die Auffüllung wurde unter den geplanten Blockfundamenten herausgenommen und eine Tauschschicht aus magerem Beton eingebaut. Bei dem Grundbauprojekt war Senkungssensibilität des räumlichen Rahmen-Tragwerks ausser Acht gelassen, keine Setzungsanalyse wurde durchgeführt. Deshalb konnte es vorkommen, dass für das gegen Setzungs-differenz und Schiefwerden empfindliche Komplex mit einem hoch liegenden Gewichtspunkt ein Grundbausystem errichtet wurde, das Setzungs-differenzen frei ermöglicht.

Die Tragfähigkeitssicherheit der angefertigten Einzelfundamente entsprach dem Normwert.

Nach der Bestimmung der voraussichtlichen Setzungen kamen wir aber zu einer grundlegend anderen Folgerung. Im Laufe der Untersuchung wurden die Spannungen, die unterhalb der Grundkörper aus ihrer eigenen Belastung bzw. - aufgrund des Prinzips der Superposition - aus der Belastung der benachbarten Grundkörper stammen, mit Hilfe von zwei verschiedenen Spannung berechnenden Methoden (Steinbrenner - Newmark) bestimmt. Die Senkungen wurden mit den im Bild 3 dargestellten Zusammendrückungszahlen berechnet. Die Ergebnisse der Setzungsanalyse sind im Bild 4 enthalten.

Die berechneten Ergebnisse ergaben Setzungswerte, die für Oberkonstruktion eines statisch unbestimmten räumlichen Rahmens

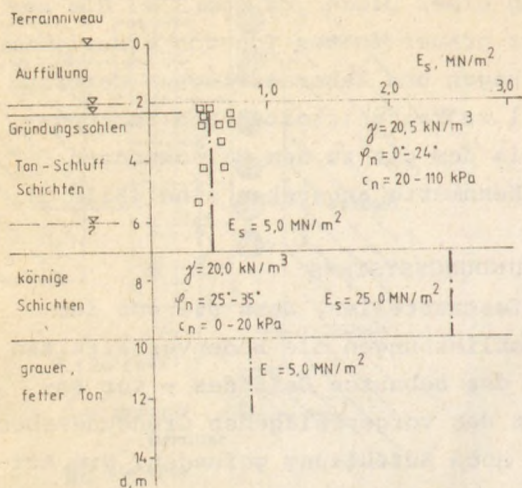
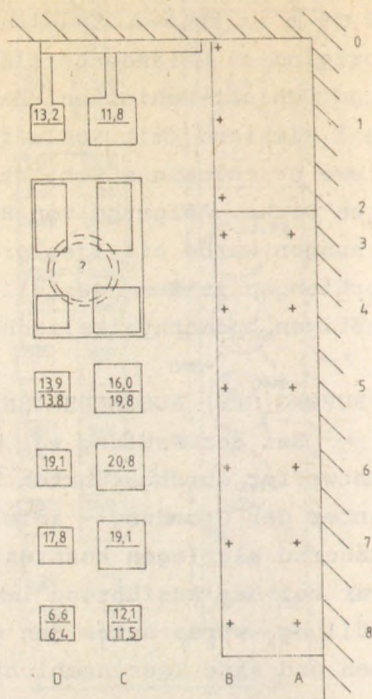


Bild 3



Berechnete Senkungen:
Steinbrenner (cm)
Newmark (cm)

Bild 4

unzulässig sind. Besonders grosse Setzungsdifferenzen waren im Falle der sich auf die Flachgründung der bereits stehenden Behälter stützenden Träger zwischen der Grundplatte und der Einzelfundamente der mittleren Reihe (c), sowie zwischen der sich auf dem alten gemauerten Brunnen stützenden kleineren Grundplatte und den nachbarlichen Einzelfundamenten zu erwarten. In diesem Falle sind also die sonst materialsparsamen, einfach ausführbare Einzelfundamente in Hinsicht der Verformung nicht genügend und die Kombination mit der Platte verdirbt die ungünstigen Setzungsverhältnisse noch weiter.

Die sichere Gründung der Oberkonstruktion war also nur durch eine komplexe Gründungstechnik möglich, die die Senkungsdifferenzen ausschliesst oder aufs Minimum vermindert.

ÄNDERUNG DES GRÜNDUNGSSYSTEMS

Die geänderte Gründung soll den folgenden Anforderungen entsprechen.

- Sie soll die der Oberkonstruktion schädlichen Senkungsdifferenzen beseitigen.
- Die bereits vorhandenen Gründungskörper sollten nicht oder nur teilweise abgerissen werden.
- Die Gründung der neuen Behälter und ihre Tragwerke sollen von diesen Konstruktionen der bereits stehenden Behälter unabhängig sein.
- Der Gründungsvorschlag soll die technischen Möglichkeiten, den Personalbestand und die technische Ausrüstung der Ausführer-Firma an Ort und Stelle in Kauf nehmen.

Zur Änderung des Gründungssystems waren zwei Vorschläge vorgelegt. Der erste Vorschlag, der später realisiert wurde, enthielt eine Flachgründung, eine Grundplatte, während der andere eine Lösung mit im Schlitzwandverfahren hergestellten Pfeilern anbot. In der Abhandlung wird nur das ausgeführte System dargestellt. Die gebaute Grundplatte sollte den Tragfähigkeits- und Verformungsanforderungen entsprechen und sollte mit den bereits fertiggebrachten Einzelfundamenten im Verbund arbeiten.

Die Bodentragfähigkeit musste nicht eingehend geprüft werden, da in dieser Hinsicht auch die Einzelfundamente ausreichend waren.

Nicht einmal an den Plattenkanten und - ecken war keine kritische Lage, weil die Grundplatte - infolge der nötigen Verbundarbeit und der Gegebenheiten des Gebietes - mit einer erheblichen Dicke zu entwerfen war.

Zur Beurteilung der Verformungen sollten die Spannungen bestimmt werden, die auf Wirkung der Plattenbelastung in dem Baugrund auftreten. Die Berechnung wurde mit Hilfe der Steinbrenner-Methode durchgeführt. Die Verteilung der Spannungswerte unterhalb der einzelnen Plattenpunkte und die infolge der Belastung entstehenden Zusammendrückungen der Bodenschichten sind im Bild 5 zu sehen.

Da die Kompressionskennwerte der tiefer liegenden Schichten günstiger sind, vermindern sich die Absolutgrößen der Senkungen - auch im Falle der Berechnung für eine flexible Platte. Die tatsächliche Starre der Platte gleicht die für flexible Platte berechneten kleineren Setzungen aus, die schädlichen Setzungsdifferenzen werden verhindert.

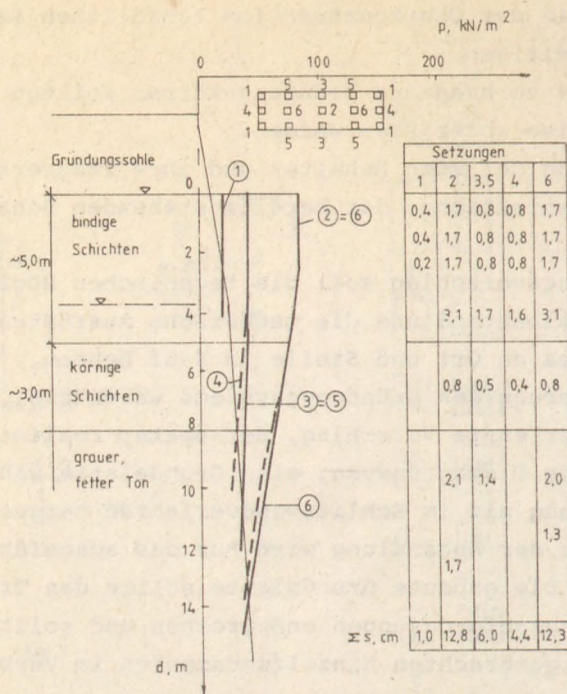


Bild 5

Das Bild 6 zeigt die Zusammensetzung der gebauten neuen Flachgründung im Querschnitt. Auf der Grundplatte sind bereits 4 Behälter gestellt und aufgefüllt. Die fortlaufende Plattendicke von min. 0,5 m, die Auffüllung aus magerem Beton unterhalb der Platte sowie die erhaltenen Stahlbeton- Einzelfundamente waren fähig, ausreichend im Verbund zu arbeiten und die schadhafte Senkungsdifferenzen beseitigen. Bei der Auffüllung der Behälter war eine Plattensenkung von 1,5-4,5 mm in den Pfeilerreihen O-1-2 gemessen. Wird die Tatsache in Kauf genommen, dass die 4 Behälter weniger als 30% der Belastung der bei der Projektierung kalkulierten 14 Behälter ausmachen und dass die gemessenen Senkungen mit grösster Wahrscheinlichkeit zu dem Konsolidationsgrad kleiner als 50% im Falle von bindigen Schichten gehören, ist es festzustellen, dass das Verhältnis der gemessenen Senkungen den für die volle Belastung berechneten Plattensenkungen gut entspricht.

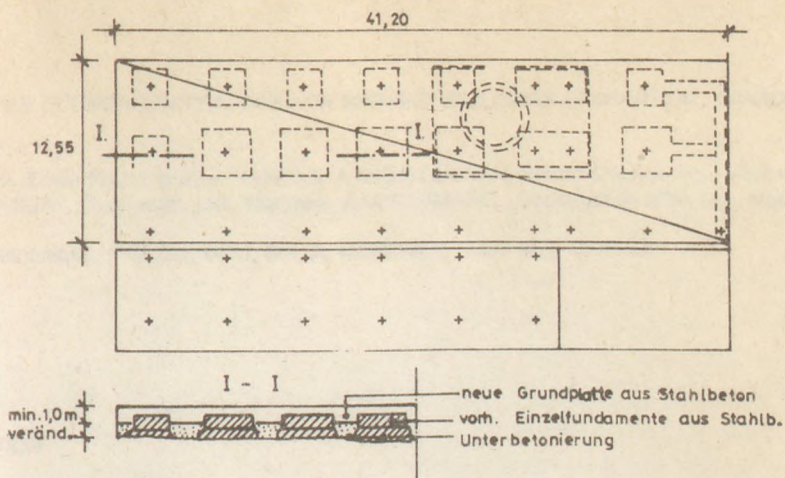


Bild 6.

Das günstige Verhalten der auch die originalen Einzelfundamente beinhaltenden Grundplatte bestätigte also die Annahme der Vorprüfungen: Die Mängel der Blockgründung der Rahmenkonstruktion können durch nachträglichen Einbau einer entsprechend starren Grundplatte behoben werden.

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DRILLED PIER FOUNDATION PRACTICE FOR HIGHRISE BUILDINGS IN MONTREAL, CANADA

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INTRODUCTION

Because of particular geological conditions in the Montreal urban area, involving thick glacial tills, underlain by mainly sub-horizontally bedded shale and limestone deposits at a depth varying from 13 to 25 m, the majority of highrise buildings, requiring deep underground space for either parking garages or shopping malls, are usually founded on drilled piers embedded in bedrock.

The design principles for drilled piers being presently well established, it was considered of interest to the profession to find out how these principles are being used in practice for the design of such piers. For this purpose, a survey of two recent sites was undertaken (1), with a special reference to the methods used for geotechnical and structural design of the piers. These methods have been subsequently evaluated in the light of the present knowledge on the behaviour of drilled piers embedded in rock.

GEOLOGY OF THE MONTREAL REGION

The Montreal region is characterized by a variety of rocks and sediments, with widely differing properties, such as strong and weak igneous rocks, sandstones and limestones, as well as strong and weak fissile shales. The bedrock stratigraphy is locally disturbed by faults and glacial overthrusts. Among the sediments, glacial tills are common. They contain beach sands, gravels, lacustrine silts, and both normally and overconsolidated clays.

Although dense glacial tills and bedrock are found at shallow depths in most of the Montreal region, some areas and in particular the city centre contain relatively deep deposits of clays and other less competent materials, unable to carry very heavy structures. Thus, a great number of highrise buildings are founded on large diameter piers socketed into rock. Because of generally high shear resistance between concrete and the rock socket, a single pier can carry very large loads, replacing

several single piles. Two typical construction sites of this kind will be described in the following.

ROCK SOCKET DESIGN METHODS

According to local geological conditions, city building codes and customs, current socket design methods are normally based on one of the following assumptions :

- (I) Load is carried by end-bearing only.
- (II) Load is carried only by shear resistance (bond) along the shaft.
- (III) Load is carried by allowable values of bond and end-bearing.
- (IV) Elastic behaviour with no slip is assumed. An allowable bond value is adopted, and end-bearing is expressed as a fraction of the applied load. (V) Mobilized end-bearing for an allowable settlement, the rest carried by the ultimate bond resistance (slip allowed).

Among these five methods, (I) is used when upper rock layers are very weak or weathered, (II) when end-bearing is uncertain because of construction or socket inspection problems, (III) is used in some cities, but it is known that it usually does not reflect the true load distribution in rock sockets, (IV) is an acceptable method, provided the bond value and end-bearing fraction of the load are properly selected, and, finally, (V), proposed in (14), is the most advanced and economical method, but it is more difficult to apply in practice, and is therefore still rarely used.

At the two sites to be described in the following, the method (IV) was used for rock socket design. In this method, the load P is assumed to be carried simultaneously by the shear stress component along the socket wall, P_s , and the end-bearing at the pier base, P_b , the latter expressed as a fraction, n , of P . Thus,

$$P = P_s + P_b \quad [1]$$

where

$$n = P_b/P \quad [2]$$

Assuming a uniform distribution of stress along the socket,

$$P_s = \pi D L s_a \quad [3]$$

where D and L are the socket diameter and length, respectively, and s_a is the allowable shear stress at the rock-concrete interface. For a given socket diameter and recommended values of n and s_a , the required socket length is then:

$$L = P(1 - n)/\pi D s_a \quad [4]$$

This method gives reasonably correct results, provided the values of n and s_a have been properly selected. In several cities in North America, the allowable values of the bond resistance s_a are given in the local building codes. In SI units, these values are, e.g., for Boston: 0.69 MPa, for New York: 1.38 MPa, and for Toronto, Ottawa and Montreal: 1.03 MPa. Some cities give also values of a

maximum or allowable end-bearing pressure, but it is usually high and is normally not attained for properly constructed piers and well inspected rock sockets.

As for the fraction n of the load transferred to the base, it is now well known from both theoretical and experimental investigations in the last 15 years (2,4,7,9,10,13,15,18,19), that, for an elastic behaviour of the rock- pier system its value depends on the socket aspect ratio, L/D , and on the ratio E_p/E_R of the moduli of elasticity of the pier (E_p) and the rock mass (E_R), respectively. In other words, the real $n = n_{e1}$ is given by

$$n_{e1} = P_b/P = f[(L/a), (E_p/E_R)] \quad [5]$$

where $a = D/2$ is the socket radius. Some values of n_{e1} according to [5] were first shown in (9), based on data from (13). More detailed n_{e1} values, published later by Pells and Turner(15), and based on a numerical method proposed in (16), are shown in Fig.1. When these curves are available, in order to find n_{e1} , one must satisfy simultaneously Eqs.[4] and [5]. From the former,

$$P_b = 1 - (L/a)(2 \pi a^2 s_a/P) \quad [6]$$

which, for each pier of radius a and loaded by P , represents in Fig.1 a straight line descending from $P_b = 1$ at $L/a = 0$, to the point on the abscissa where $P_b = 0$, given by

$$(L/a)_{max} = P/2 \pi a^2 s_a \quad [7]$$

$(L/a)_{max}$ is the aspect ratio of the socket for which all the load is carried by the shaft, as long as s_a has been properly selected. If $L/a > (L/a)_{max}$, the portion of the pier exceeding $(L/a)_{max}$ is considered to be unloaded and therefore needless. If $L/a < (L/a)_{max}$, the true load transfer to the base can be evaluated by combining Eqs.[5] and [6]. As [5] is not given analytically, this can be done either by trial and error (9), or graphically, as shown in (15) and in Fig.2. What it really means is that the value of n cannot be arbitrarily selected, but is determined by the socket shape and the modulus ratio. Figure 2 shows a case in which $P = 4$ MN, $s_a = 0.35$ MPa, and $E_p/E_R = 10$. Intersection of any given line (for a given pier diameter) with the curve, gives the value of n_{e1} on the ordinate, and the corresponding ratio L/a on the abscissa. Generally, for usual aspect ratios of rock sockets, $L/a > 4$, it is found that less than 20% of the load will be transferred to the base at service loads.

If, on the other hand, L/a has been selected arbitrarily for an assumed n value, and the modulus ratio is known, the elastic load transfer fraction can be found directly from Fig.1. The result is that, if $n > n_{e1}$, more load will be carried by the shaft, so that the value s_a will be exceeded. This is usually not a serious problem because, as shown by laboratory and field tests (11,14,18), for normal socket roughnesses, lateral shear behaviour is not brittle but pseudo-plastic, without much loss of strength after the slip. This is thought to be due to the gradual replacement of the adhesive bond resistance by the frictional one, mobilized by pier displacement along a rough socket wall, which because of dilatancy effects at the concrete/rock interface, increases the radial stress acting on the socket (2,10,20).

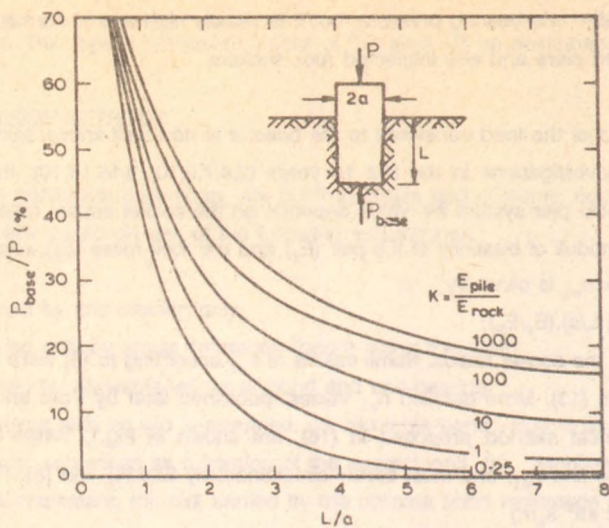


Fig. 1. Rock Socket Design Curves (After Pells and Turner, 1979)

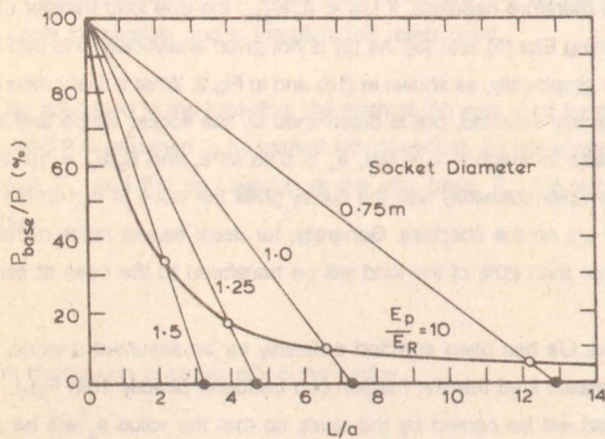


Fig. 2. Graphical Method for Rock Socket Design (After Pells and Turner, 1979)

In other words, even if s_a is exceeded, the socket will generally still be able to carry the load without excessive settlement, provided the rock is sound below the base and there is a good concrete-rock contact at the base. This is in fact the principle of Method (V) mentioned previously. However, in that case, the settlement of the pier should be checked by using the method described in (15), valid for an elastic pier/rock system behaviour.

In order to facilitate the use of the method described above, an attempt has been made here to express analytically some of the numerically determined curves in Fig.1. It was found, e.g., that the curve for $E_p/E_R = 10$ in that figure can, for $L/a > 3$, be expressed approximately by

$$\eta_{e1} \approx (1 + 1.25 L/a)^{-1} \tag{8}$$

which, combined with Eqs.[6] and [7], yields the corresponding value of L/a ,

$$L/a \approx (L/a)_{max} - 0.8 \tag{9}$$

In the same manner, the curve for $E_p/E_R = 100$ in Fig.1 can, for $L/a > 3$, be expressed by

$$\eta_{e1} \approx (1 + 0.77 L/a)^{-1} \tag{10}$$

giving the corresponding L/a ,

$$L/a \approx (L/a)_{max} - 1.3 \tag{11}$$

DETERMINATION OF ALLOWABLE BOND

As mentioned in the foregoing, the allowable bond stress for piers socketed into rock is given in certain city building codes. However, it is known from systematic field and laboratory investigations that its value depends on the strength of the weaker material at the socket/rock interface and on the socket roughness (4,6,7,10,11,13,18,20). For standard drilled sockets (with amplitude of asperities > 3 mm, and their average deviation angle $> 14^\circ$, according to (20)), brittle failure along the socket will not occur, and the value of shear bond resistance can be related to the uniaxial compression strength of the rock substance, q_u . Although for weak rocks values of shear bond of 5% to 10% of q_u have been used in the past, for relatively hard rocks, a relation giving values close to the measured ones is that given by Horvath et al.(4,6)

$$s_{max} = 0.25 \sqrt{q_u} \tag{12}$$

from which, $s_a = s_{max}/FS$, with $FS = 2.5$. According to (17) and (19), Eq.(12) gives generally conservative values, and may be replaced by (19):

$$s_{max} = 0.45 \sqrt{q_u} \tag{13}$$

For comparison, it is interesting to note that for the allowable bond for rock anchors, Littlejohn and Bruce (12) recommend $q_u/30$, with a maximum value of 1.4 MPa.

DESIGN OF ROCK SOCKETS AT THE SITE A

The Site A, located in downtown Montreal, was originally occupied by the Christ Church Cathedral, a parking lot and a couple of 2-4 storey buildings. On the site, a full underground shopping mall

had to be built, partly under the church, and partly under a new 34-storey office tower. The church had to be left intact by carrying out a complete underpinning of its 100-year old shallow foundations. Geotechnical investigations carried out at the site in 1985 by Inspec-Sol Inc., revealed a ground profile composed of about 3 m of loose fill, underlain by loose to dense glacial till, containing clays, clayey silts, silts, and sandy silts with gravel, cobbles and boulders. The bedrock was found at a depth of 13 to 16 m below the original ground surface. It was described as "Dark grey shaley limestone with subvertical thin calcite veins, unweathered but very fractured, with typical fractures at every 5 to 7 cm (RQD = 0%). As the construction plan required all soil deposits to be excavated because of the shopping mall and garages, drilled piers socketed into bedrock were selected as foundations.

For the design of piers, the geotechnical consultant (Inspec-Sol Inc.) recommended the values of n and s_a to be taken as: $n = 15\%$, and $s_a = 1.03$ MPa. It is noted that the recommended value of s_a corresponds well with Eq.[12], because this rock had q_u varying from 102 to 117 MPa. The modulus ratio for this rock is considered to be about 10, because according to (19), the rock modulus is related to q_u approximately by: $E_R = 215 \sqrt{q_u}$, giving for E_R values of about 2200 to 2300 MPa, which is close to one tenth of E_{concrete} .

All 23 outside piers serving for the church underpinning, had diameters of 91.44 cm, and they carried loads varying from 2 MN to 11.56 MN. The socket lengths calculated from Eq.[4], varied from 1.52 to 3.35 m, giving L/a between 3.33 and 7.33. When checked according to the elastic theory predictions described in the foregoing, it was found that actual n values were $0.193 < n < 0.098$, with an average of $n = 0.145$, which is close to the assumed value of $n = 0.15$. As a result, actual s_a values varied slightly from socket to socket between 0.98 and 1.10 MPa, which is close to the value assumed in the design. The settlement of the most highly loaded pier with $L/a = 7.33$ under $P = 11.56$ MN would, from (15), amount to about 2 mm.

DESIGN OF ROCK SOCKETS AT THE SITE B

The Site B is located in Longueuil, a community on the opposite side of the St. Lawrence River, south of Montreal. The structure that had to be built was a 22-storey condominium complex with a basement, called L'Estuaire Phase III. At the site, the ground profile is composed of about 7 m thick deposit of silt, sand and schiste debris. The bedrock at the depth of 7.26 m was described as a black fissile shale, horizontally bedded, with RQD of 25 to 55%. The 21 rock-socketed piers selected as foundations for this structure, had diameters of 40.6 cm, 61 cm, 76 cm and 91.4 cm, respectively. They carried loads from 1.1 to 2.0 MN for the smallest, up to 7 to 10 MN for the largest diameter piers. The required lengths of the sockets were calculated from Eq.[4], by adopting the values of $n = 10\%$ and $s_a = 1.00$ MPa. The resulting L/a ratios varied between about 3.9 to 7.8, with most

frequent values of about 5.0 to 6.6. Although no uniaxial strength information for this rock is available, local experience and field data (18) justify well the selected value of $s_a = 1.00$ MPa, while $n = 10\%$ comes still closer to the elastic theory predictions.

DISCUSSION AND CONCLUSIONS

This analysis shows that the socket design method (IV), when correctly applied, furnishes acceptable results, close to theoretical predictions. It is noted that this method implies no-slip condition at the rock-concrete contact, which is satisfied when s_a remains below the ultimate bond strength. However, s_a is only an average value, while the true distribution of shear stresses along the socket in non-uniform, with a maximum at the top of the socket (8). This may produce a progressive slip, even under allowable loads, leading with time to a small increase of the base load (9) and a creep settlement of the pier (3,5). Nevertheless, the total settlement is still expected to remain within tolerable limits of 1 to 2% of the pier diameter. Another interesting aspect of rock socket design, worth mentioning, is that, even a very fractured rock with RQD of 0% may still be acceptable as bearing medium, provided it is unweathered and mechanically undisturbed with closed interlocked joints. A method for determining the maximum shear bond for a socket in such a rock, where it is difficult to take a proper core for q_u determination, was shown in (11).

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NEW FOUNDATION METHOD OF MOTORWAY BRIDGE ON THE RIVER DANUBE

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To release the downtown of Budapest from heavy through-traffic a ring-shaped motorway trace around the city has been proposed. The first southern section of that Motorway M0, just under construction, crosses the river Danube. The crossing is made up of a main 315 m bridge over the river itself with three openings of 105 m each and access bridges over the flood areas on both banks symmetrically with a total length of 444 m. Three piers of the main bridge are situated in the riverbed.

It was about 45 years ago when a similar job of constructing bridge piers had been completed on the Hungarian reach of the Danube. Methods in use that time, involving mainly caissons, are rather outworn in our days and practices developed and used recently on other rivers countrywide have also been valued as inapplicable ones because of differences in hydraulics. Furthermore, some possible alternatives using steel sheetpiles - double-wall cofferdams, man-made islands of any kind for instance - showed during their recent applications their time-consumptions longer than acceptable in the case discussed. Both reasonable cost level and tight construction timetable urged the contractor during the bidding and designing period to find a new construction method which, in comparison with the analyzed earlier ones, was carrying advantages in several details of it.

Principles to be taken into consideration by that problem-solving task were:

- to minimize auxiliary/temporary structures used for watertight enclosure during the construction period
- to reduce considerably the works to be done in the riverbed by using precast reinforced concrete members

- to utilize the furthestmost possibilities of the available floating crane with 1200 kN lifting capacity, which must have been used anyway by assembling the superstructure
- to involve recent developments of associate technologies, mainly dredging and industrial diving
- as a result of the enumerated and other factors the shortest possible construction period of the pier has to be achieved and thus a significant saving in construction costs has also to be targeted.

All the mentioned factors resulted in the final construction method chosen during the whole possible period of design, i.e. from the early competitive alternatives of bidding to the last hours of drawing before commencement of construction.

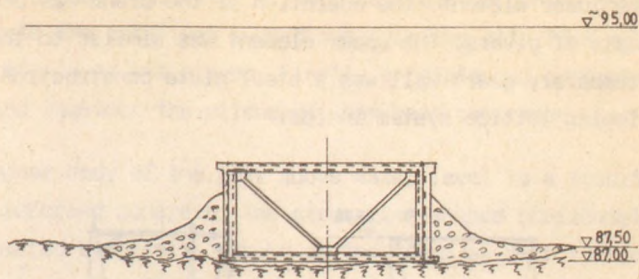
Stages of the construction and their characteristics are the followings:

Stage 1: Dredging with a floating ladder-bucket excavator in two phases.

First the expected bottom level of the regulated riverbed is to be reached in a widely spread environmental area, the dredged quantity of which in our case makes 117,000 m³ of soil, mainly gravelly sands or silty sands. The second phase, to be done immediately before placing precast bottom member, means a deepening of the bottom at the close area of the pier by 0.5 m but with an accuracy of the requested final level by only ± 2 cm. Such a correctness has never been achieved before with ladder-bucket type excavators.

Stage 2: Placing the lower prefabricated reinforced concrete crust-element (immediately after second-phase dredging). The element itself, weighing 1050 kN, has been prefabricated at the riverbank on a location to be reached easily at any water-level by the floating crane "Clark Ádám". The hoisting into definitive place needs a sophisticated land-survey work of live instruments; theodolites, distance-meters and pointing lasers. The element was equipped with two steel pipes of diam. 400 mm holding reference marks on their top edges, so the actual position of the element could be monitored even during its underwater motion. Evaluation of measurements from different survey instruments has taken place in the wheel-house of the floating crane continuously. So the crane, approaching the location from upstream, could be manoeuvred quickly and accurately. The whole placing has taken 4-5 hours and the accuracy

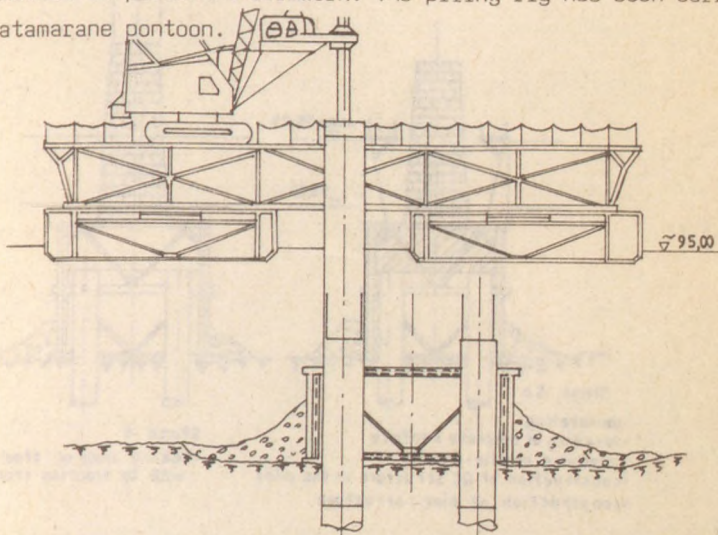
of the definite position has been measured as being within 30 per cent of the expected margins of error. The placement was followed within one hour with protection works against erosion by using stones with appropriate shape, size and quality.



Stage 1-2

- dredging in the river bed (100m×300m) for level 87,50 m
- dredging under the pier for level 87,00 m
- placement of bottom RC unit by floating crane

Stage 3: Pile foundations. Bored cast-in-situ piles were made with diam. 1500 mm. Accurate positioning was given by guide steel-rings built into the lower prefab r/c unit, so divers have helped to start piling instead of surveyors. Retracted casing has been used as borehole protection and at the upper part of the pile, protruding from the bottom into the water by a length of 4.5 m, this casing has remained as permanent formwork. The piling rig has been carried by a catamarane pontoon.

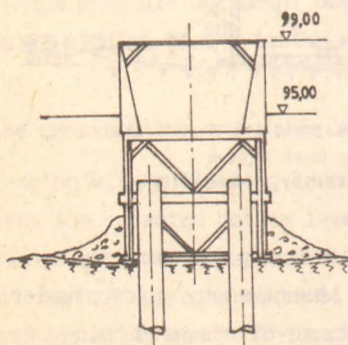


Stage 3

construction of diam 1500 mm bored piles (8-10 pcs)

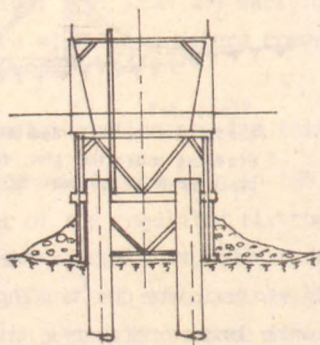
Stage 4: The upper prefab r/c member and the temporary steel guard-wall, joined together watertightly on the bank, have been lifted onto the lower r/c element.

Proper positioning has been helped by built-in centering devices of the lower element. The operation of the crane has been based on the data of divers. The upper element was similar to the lower one. The temporary guard-wall was a steel-plate construction with a stiffening lattice system inside.



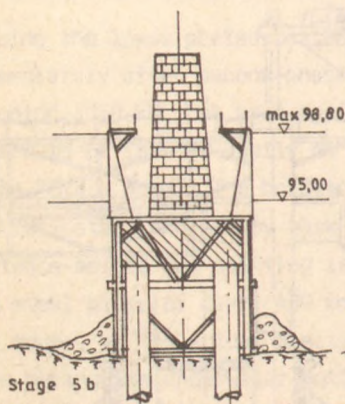
Stage 4

- placement of upper RC unit and steel guard wall by floating crane



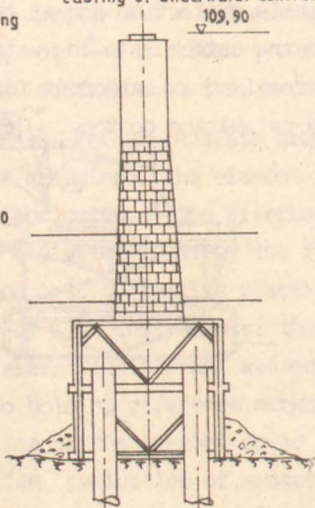
Stage 5a

casting of underwater concrete



Stage 5b

-dewatering
-levelling of concrete surface
- cleaning of pile-heads
-construction of RC structure on the piles
-construction of pier structure



Stage 6

-taking away of steel guard wall by floating crane

Stage 5: Underwater concrete. Instead of traditional movable tremie pipes the pouring was done by using four permanent steel tubes fixed to the inner stiffening structure. Thanks to the good watertightness there was no need of slurry pumping or other preparatory works before concreting. After a seven day long hardening period of concrete the water was pumped out from the pit. Then a levelling concrete layer was placed. Finally a reinforced concrete head-slab, holding together the pileheads, has been concreted.

Stage 6: The upper body of the pier above water level is a traditional one of reinforced concrete. The streamline-shaped horizontal section is covered by granite blocks at the edges as a protection against ice motion. The final action of constructing a pier was the removing of the steel guard-wall, which was to be used at the following piers, too.

The total construction time of a pier took 4-4.5 months, which could be judged as a good comparable result among other up-to-date bridge foundation technologies. Pier construction was carried out by the Hídépítő Vállalat (Bridge Construction Company) as general contractor. Main subcontractors were: the FOKA Company for underwater dredging and UVATERV Consulting Engineers for design and survey.

THE ANALYSIS OF DRILLED PIERS SETTLEMENTS WITH NON-LINEAR DEFORMABILITY OF THE SOILS

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One of the main differences of deep foundations from shallow foundations is that the deep once transfer the load to the base not only by their footing but also by their side surface. At first let us consider a pier compression under load without tangential forces along the side surface and with their action (Fig.1,a and 1,b). The compression value S in the first case is

$$S = \frac{Q_A L}{E A} \quad (1)$$

where E - Young's modulus of the pier material, A - its cross-section area. If tangential forces Q_S act uniformly along the side surface at tangential stress value τ_0 , we have

$$Q_S = 2\pi R L \tau_0 \quad (2)$$

From the equilibrium condition we can obtain (Fig.1,b)

$$dG_z = -2\tau_0 \frac{dz}{R} = -2\tau_0 \frac{\Delta L}{R} \quad (3)$$

If the tangential stress values are equal within each section $\tau_{0,i}$ and the whole pier is divided into n sections L_i high

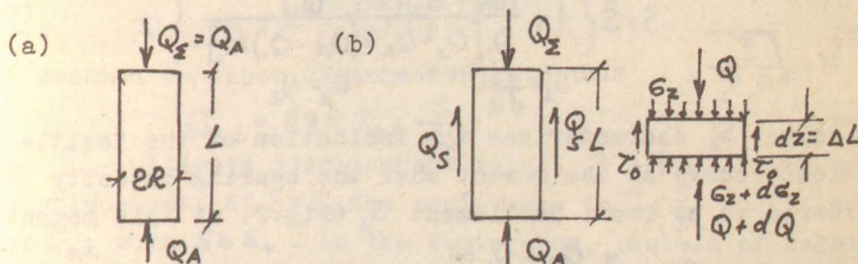


Fig. 1. The scheme. (a) - without tangential forces, (b) - with them

each, then upon summation we have the compression value

$$S = \frac{Q_1 L}{EA} - \frac{\pi RL}{EA} \sum_{i=1}^n \tau_{0,i} L_i \quad (4)$$

The suggested analysis of the settlement value due to the load transferred by the pier end is carried out with account of non-linear deformability of the foundation soil according to the earlier proposed method (Malyshev and Nikitina, 1982). For this purpose it is necessary to know the ultimate load values for foundation soil Q_1 - when the plastic zone arise, and Q_2 - corresponding to the state when the bearing capacity is up. At loads $Q < Q_1$, we have the settlement value varying according to the linear law

$$S = \frac{\bar{S}}{Q_1} Q_A \quad (5)$$

where $\bar{S}$ - settlement corresponding to the beginning of the plastic zone arise under the edge and being

$$\bar{S} = \frac{1.58 (1 - \nu^2) R Q_A}{E} \quad (6)$$

Here ν - Poisson's ratio of the soil. Value Q_1 is defined by formula

$$Q_1 = \left[\frac{\pi (\gamma L + c \cot \varphi)}{\cot \varphi + \varphi - \pi/2} + \gamma L \right] A \quad (7)$$

where γ - average unit weight of the soil located above the footing, c - cohesion and φ - angle of internal friction. Ultimate load Q_2 can be found from formula (Wintekorn and Hsai-Yang Fang, 1975)

$$Q_2 = [N_\gamma \gamma R + N_q \gamma L + N_c c] A \quad (8)$$

where N_γ , N_q and N_c are the well-known bearing capacity factors. Within load variation $Q_1 < Q_A < Q_2$ we have settlement

$$S = \bar{S} \left\{ 1 + \frac{(Q_2 - Q_1)(Q_A - Q_1)}{Q_1 [Q_2 - Q_A + (Q_A - Q_1)M]} \right\} \quad (9)$$

Here $M = \sqrt{\frac{\bar{S}}{Q_1 S'_2}}$

and $S'_2 = \frac{dS}{dQ}$ when $Q_A = Q_2$

This value S'_2 characterizes the inclination of the "settlement-load" curve at the moment when the bearing capacity is considered to be lost. Settlement S_2 (Fig.2) at this moment is

$$S_2 = \bar{S} \frac{Q_2 + Q_1 M}{Q_1 (1 + M)} \quad (10)$$

Formula (9) is simplified if a vertical tangent is assumed at the moment of the failure ($S_2 \rightarrow \infty$; $S'_2 \rightarrow \infty$) and takes the form

$$S = \bar{S} \frac{Q_A(Q_2 - Q_1) - (Q_A - Q_1)Q_1}{Q_1(Q_2 - Q_A)} \quad (11)$$

The curve of settlement $S(Q_A)$ at point $Q_A = Q_1$ acquires a certain fracture side on the left-hand side at $Q_A < Q_1$, we have

$\frac{dS}{dQ} = \bar{S}/Q$, while on the right-hand side at $Q_A = Q_1$, we obtain $\frac{dS}{dQ} = \bar{S} \cdot \frac{Q_1}{Q_1(Q_2 - Q_1)}$. At sufficiently high values Q_2 in relation to Q_1 the curve fracture will be small.

The analyses of settlements for the drilled piers were earlier described by Hakimov (1967) and especially by Berezantsev (1970). Berezantsev noted that the term with N_f in formula (8) for deep foundations is less than other once.

Now let us consider the forces acting on the side surface. We use the values determined for the bearing capacity of bored piles and piers (Gosstroy USSR, 1986; Kezdi and Rethati, 1988). They depend on the physical state of the soil and its properties as well as on the depth where the reference section is located increasing with the depth. The bearing capacity of piers is determined by the mobilized value of the skin friction. The mobilization of the friction depends on the pier displacement. This question was considered schematically by Ellison (Franke, 1985). A linear relationship between the pier displacement and mobilized friction was assumed in that paper up to the ultimate value with subsequent separation. The value of settlement with respect to the pier diameter was assumed being the same at the moment of separation = 0.02.

If the ultimate value of specific resistance is denoted as $\tau_{0,i}$ then the ultimate value Q_s is

$$Q_{s,u} = 2\pi R \sum_i \tau_{0,i} L_i \quad (12)$$

It is assumed in paper (Franke, 1985) that

$$Q_{s,i} = 2\pi R \tau_{0,i} \frac{S}{S_u} \quad (13)$$

where S_u - ultimate displacement value, S - real value of the displacement. At $S < S_u$ the resistance is $\tau_{0,i} \frac{S}{S_u}$ and it is always $\tau_{0,i}$ when $S \geq S_u$. As the tests show, instead of relationship (13) it is better to use a linear-fractional function

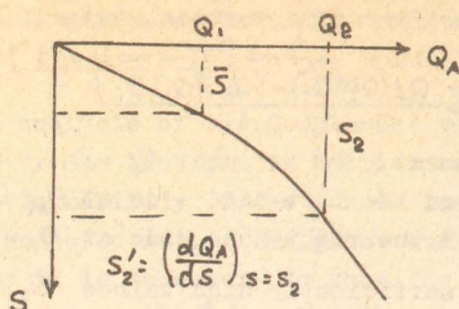


Fig. 2. Relationship "Load - settlement"

for displacements. Then we obtain

$$Q_S = 2\pi R \tau_0 \frac{2S/S_u}{1 + S/S_u} \quad (14)$$

Here as in paper (Franke, 1985) $S \leq S_u$, and at $S > S_u$ we assume $S = S_u$ to calculate Q_S . The relation between the force and the load is performed as follows. For the vertical load we have formula (9). The side force is defined by formulas (2) and (3). The force required to calculate settlement S is related to the force applied from the top on pier Q_Σ by the equilibrium condition

$$Q_\Sigma = Q_A + Q_S \quad (15)$$

To clarify the influence of individual factors Fig. 3 presents a diagram of constituent relations between the displacement and the force acting on the pier as linear-discrete dependences. The displacement of the pier top 1 is assumed as displacement S . This figure presents similar plots for the pier - its footing 2 and section 5 being on a half of the height. The reference 4 characterizes the pier compression. Fig. 4 shows summary plot 3 and its components required to find the "settlement-force" curve for the pier. If these dependences are represented not a linear discrete plots but as the plots described by formulas (9) and (14), we obtain essentially similar plots given in Fig. 3 but with non-linear sections 1, 2 and 5.

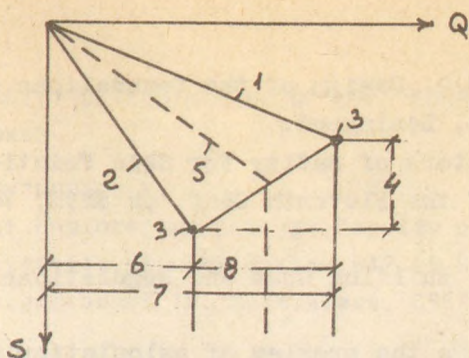


Fig. 3. Plot of the side friction component. (1 - pier top displacement; 2 - pier footing displacement; 3 - a point characterizing pier break down along the side surface; 4 - displacement due to pier material compressibility; 5 - displacement of the section at the pier mid-length; 6 - force transferred by the footing of the pier; 7 - force transferred upon the pier from the structure; 8 - force taken by the pier side surface).

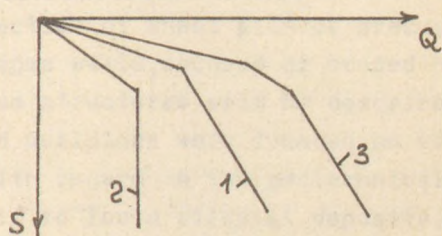


Fig. 4. Plot of the pier top settlement. (1 - plot of "settlement - load" relationship for the footing; 2 - plot of the relationship of the pier top settlement due to side friction mobilization and compression of the pier itself; 3 - the top settlement).

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BEARING CAPACITY DETERMINATION OF DEEP FOUNDATIONS IN BRATISLAVA AND OTHER TOWNS

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INTRODUCTION

Bratislava, the capital of Slovakia lies near the frontiers of Austria and Hungary. During the last 15 years the number of inhabitants has increased of about 150 thousands and has grown to 400 000. On account of this increase the housing and several other buildings were concentrated in this town (the National Theatre, the National Gallery, department stores, press-centre, hotels etc.).

The mentioned buildings required deep foundation pits constructed under the protection of sheet pile or precast or monolithic underground diaphragm walls, anchored or braced by various stiffening systems. These structures will be described in another paper. The mentioned buildings were founded on various types of deep foundations with regard to the geotechnical conditions (from medium compact to loose alluvial deposits, neogene sediments-sandy clay and silt) and to the acceleration of foundation construction. In order to determine the required bearing capacity of deep foundations it has been developed a new simple and for the application in towns ecologically correct loading system, producing the load of 1100 kN. For the determination of the pile bearing capacity it will be described a computation using the pressuremeter tests results, which are advantageous from the standpoint of their speed and ecological fitness in town agglomerations as well.

TESTING EQUIPMENT FOR THE BEARING CAPACITY OF DEEP FOUNDATIONS

A steel cylindrical tank with the diameter of 3,1 m and length

of 12 m was used as the testing equipment. The load test is performed by placing the tank on the pile head by a crane. The tank is anchored by three steel anchor cables. The panels, piles or motor vehicle can be used as anchoring blocks. (Fig.1)

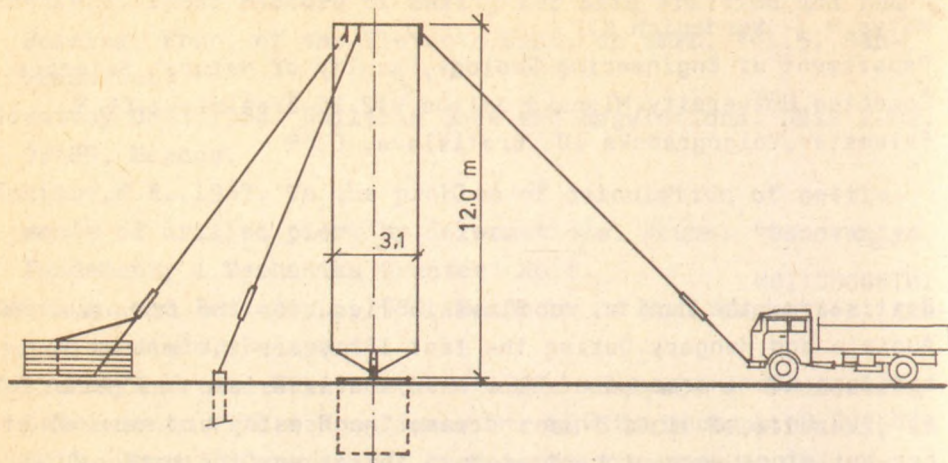


Fig.1 Scheme of the loading system

Each cable is attached to the anchoring block by means of the turnbuckle. The cables have to close in the plane projection an angle of 120° . The minimum distance among the anchors and the axis of testing has to be about 15 m. The filling of tank with water through the valve placed on the tank bottom can begin after the field check of the anchors, the vertical position of equipment, the scale of water level, feeding hoses and measuring instruments. The loading of the tank placed on the tested foundation is increased by the successive filling of tank. The load is given by the gravity of tank (130 kN) and its water. The loading force acting on the foundation is regulated by filling or discharge of water. The length of loading steps is chosen according to the usual methodics of pile loading tests. After the execution of the load test, water will be discharged and the tank will be transported to another locality by a flat track (Fig.2). The whole load test of deep foundation with the assembly and dismanting can be performed in one prolonged shift (circa 10 to 12 hours).

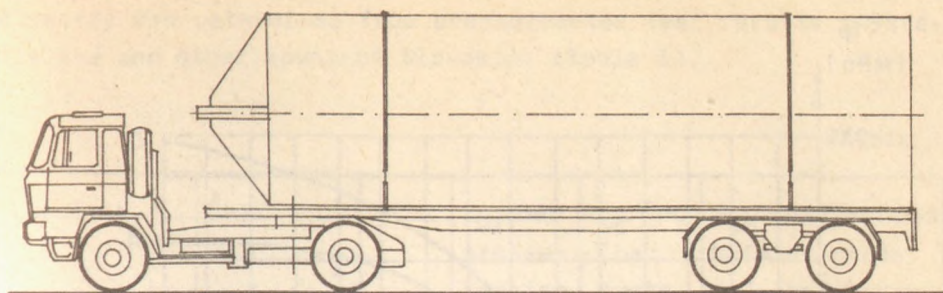


Fig.2 Load tank prepared to transport with flat track

BEARING CAPACITY DETERMINATION OF DEEP FOUNDATIONS FROM PRESSU- REMETER TEST RESULTS

It is not always possible to transport the described equipment to the construction site and to perform the loading tests with regard to the land coverage in towns. In addition to other methods, an in-situ load test, carried out within the borehole - the pressuremeter test - can be used in these cases.

For the calculation of the design bearing capacity of deep foundation, the following relationships, quoted in /1/, have been established by Ménard (1975).

$$U_{vd} = R_{dp} \cdot A + \frac{k}{2} \cdot U \cdot \sum_{i=1}^n h_i \cdot R_{fp_i}$$

where:

- U_{vd} - is the design bearing capacity of deep foundation,
- R_{dp} - is the design bearing capacity of soil at the level of pile foot (calculated according to the relationships quoted in /1,2/,
- A - is the pile foot area,
- k - is the coefficient of deep foundation, see in /1,2/,
- U - is the pile perimeter,
- h_i - is the tickness of layer at depth i ,
- R_{fp_i} - is the friction between the pile skin and the soil in the layer at depth i .

The value of skin friction R_{fp} is determined from the pressuremeter tests results given by the plot on the figure 3 /1/.

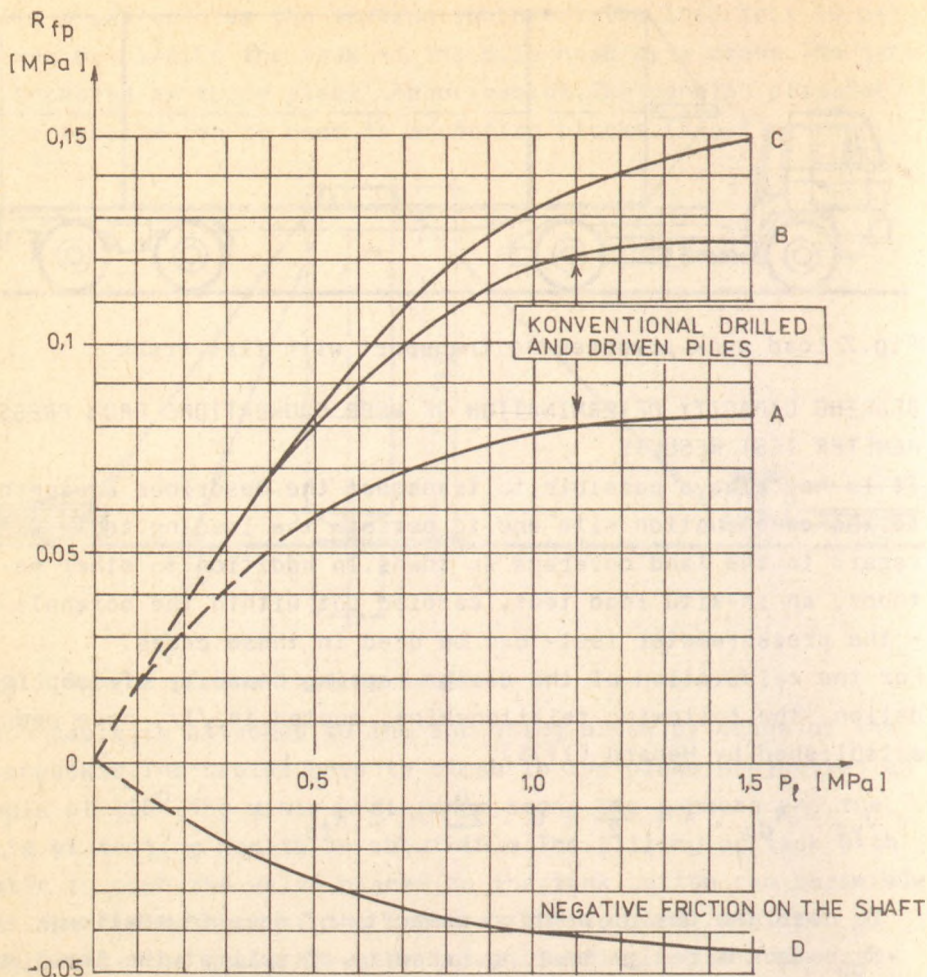


Fig.3 Skin friction resistance of a pile as a function of the limit pressure

The value of R_{fp} depends on the technologic process of pile construction, on the petrographic soil type (sand, clay, etc.), on the origin of the soil (alluvial, eolic soil) and on the mineral composition of the soil. The value of the skin friction R_{fp} is therefore regionally variable and it has to be determined for individual regions and for the used technologic procedure of pile construction. The vibro-driven piles VUIS are used for the deep foundations in CSFR.

The skin friction value was tested by several series of the comparative bearing capacity tests of deep foundations. This bearing

capacity was determined from pressuremeter test results in Bratislava and other towns of Slovakia. (Table 1).

Table 1

Locality	Soil	Pile	U_{vd} (kN) pressu- remeter tests	U_{vd} (kN) Load tests	U_{vd} differences between press. and load tests (%)
Bratislava	slury	V 1	325,5	350,0	+0,71
	sand,	V 2	349,0	350,0	-0,29
	clay	V 3	267,0	350,0	-23,70
	loam,	V 4	313,0	350,0	-10,57
	sand	V 5	342,5	350,0	-2,14
	clay	V 7	281,0	350,0	-19,71
Trnava-	loess	TP 1	662,8	690,0	-3,94
Prednádražie	loess like sed.	TP 2	510,0	515,0 490,0	+28,70 +4,08
Michalovce	sand clay	SP 1	219,0	182,0	+20,33
	clay loam	taper		210,0	+4,29
Vrbové	clay loam	V 1	585,5	600,0	-2,42
	sandy loam, sandy-clay loam	V 3	335,2	330,0	+1,58
Trnava- Hlboká	loess,	P 9	426,2	455,0	-6,33
		P 10	487,45	620,0	-21,38
	loess like sediment	taper P 11 taper	487,45	610,0	-20,09
Nová Dubnica	clay loam sandy loam gravel	ND 3	708,0	720	-1,67

The maximum differences are in the ranges of - 23,5 % to +28,7%
The results will be in the range of $\pm 5\%$ /2/ on condition of fulfilment of following instructions for the performance and interpretation of pressuremeter tests:

- the pressuremeter tests has to be carried out after each one meter pass of the drill,
- the limit pressure p_1 should be determined at least to the level of pile foot,
- the computational value determination for the pile skin of the pile VUIS should be performed by means of the curve C for all soils with the exception of clay loams and clays with a very soft consistence. For these soils it should be used the curve A (Fig.3),
- for the computation of the pile foot area the diameter should be increased in sand and gravel soils to the value of 1,28 multiple of the diameter.

CONCLUSION

In order to determine the required bearing capacity of deep foundation applied in deep foundation pits as well, two methods have been described in the paper. They should be recommended also for other conditions with regard to the economical and ecological feasibility (elimination of the contamination of the soil and water, noiseless test performance) and to the obtainable speed of results.

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RESULTS OF FIELD LOAD TESTS OF PILES

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INTRODUCTION

In the past decades a number of methods have been developed for analysing axially loaded piles. The determination of the ultimate bearing capacity of piles, based on the laboratory investigation of soil parameters, is accompanied by a considerable dispersion of results. Therefore, the efforts are oriented towards the field investigations, such as penetration tests and field load tests.

The ultimate bearing capacity of pile can be determined by the expression (Mohan et al., 1963):

$$P_f = P_p + P_{sk} = R_p A_p + \frac{R_{pav}}{50} A_{sk} \quad (1)$$

where: A_p = area of pile base; A_{sk} = area of pile shaft; R_p = cone resistance of soil, obtained by static penetration test.

Bustamante and Gianeselli (1982) suggest the following expression:

$$P_f = R_p A_p K_p + \sum_i \frac{R_p}{K_{sk}} D \pi \ell_i \quad (2)$$

where: K_p and K_{sk} are dimensionless coefficients which depend on the type of soil and its penetration resistance.

The unit skin friction f_s , given in eq. (1) is equal to:

$$f_s = \frac{R_{pav}}{50} \quad (3)$$

whereas Meyerhof (1956) recommends the expression:

$$f_s = \frac{R_{pav}}{100} \quad (4)$$

For cohesive soils Broms and Hellman (1968) suggest the expression:

$$f_s = \alpha C_u \quad (5)$$

in which C_u is the undrained shear strength of soil and for reinforced concrete piles $\alpha=0.80$.

Semple and Rigden (1984) use $\alpha=1.0$ for $C_u \leq 35$ kN/m² and $\alpha=0.50$ for $C_u \geq 80$ kN/m². Fleming et al. (1985) recommend the following expressions for α :

$$\alpha = \left(\frac{C_u}{\sigma_v'}\right)^{0.5} \left(\frac{C_u}{\sigma_v'}\right)^{-0.5} \quad \text{for} \quad \frac{C_u}{\sigma_v'} \leq 1 \quad (6)$$

$$\alpha = \left(\frac{C_u}{\sigma_v'}\right)^{0.5} \left(\frac{C_u}{\sigma_v'}\right)^{-0.5} \quad \text{for} \quad \frac{C_u}{\sigma_v'} \geq 1 \quad (7)$$

According to Burland (1973), the unit skin friction can be determined by:

$$f_s = (1 - \sin \phi') \operatorname{tg} \phi' (\text{OCR})^{0.5} \quad (8)$$

where OCR is the overconsolidation ratio. Lyons (1974) gives the expression

$$f_s = \beta \sigma_v' \quad (9)$$

where $\beta = F \operatorname{tg}(\phi' - 5^\circ)$. For piles in compression the coefficient F is suggested to be $F=0.7$ and for piles in tension $F=0.5$. Poulos (1988) recommends $\beta=0.5-0.8$.

Deformation modulus E_s is very often correlated with cone resistance R_p in cone penetration test. So, Christoulas (1985) gives for piles in clay the following expression:

$$E_s = 21 R_p^{1.09} \quad (10)$$

For piles in sand Imai and Tonouchi (1982) use the expression:

$$E_s = 53 R_p^{0.61} \quad (11)$$

where E_s and R_p are in MN/m².

For over consolidated sands Poulos (1988) gives the expression:

$$E_s = 7.5 R_p \quad (12)$$

and for normally consolidated sands:

$$E_s = 5.0 R_p \quad (13)$$

whereas Milović and Stevanović (1982) give higher values for E_s :

shaft is determined by:

$$P_{sk} = P(1-\beta) \tag{18}$$

and the corresponding settlement:

$$\rho = \frac{IP_{sk}}{E_s D(1-\beta)} \tag{19}$$

The load carried by the base is given by:

$$P_p = P\beta \tag{20}$$

The base load versus settlement relationship is thus:

$$\rho = \frac{IP_b}{E_s D\beta} + (P_b - \frac{P_{sk}\beta}{1-\beta}) \frac{L}{A_p E_p} \tag{21}$$

Table 2 The unit skin friction f_s

pile No	$f_s, \text{ kN/m}^2$				
	eq.(2)	eq.(3)	eq.(4)	eq.(17)	field test
1	76	116	58	52	30
2	47	126	63	46	29
3	48	140	70	20	48
4	33	20	10	14	14
5	40	46	23	15	50
6	42	50	25	28	88
7	42	50	25	28	80

The predicted load-settlement curves for tested piles are also shown in Figs. 1-7 (dash lines). As shown in Fig. 3, the agreement between the predicted and measured load settlement curve depends to a large extent on the ratio E_p/E_{sk} , where E_p and E_{sk} are moduli of soil at pile tip and along shaft, respectively.

Using the expression for settlement of a pile, given by Davis and Poulos (1980):

$$\rho = \frac{PI}{E_s D} \tag{22}$$

where $I=I_o R_k R_h R_v$, the values of the deformation modulus E_s were deduced from field load tests of piles, and shown in Table 3.

$$E_s = (20 - 40) R_p$$

(14)

For further analyses of the above mentioned parameters, the results of field load tests will be used.

RESULTS OF FIELD LOAD TESTS OF PILES

On several sites field load tests of piles and cone or standard penetration tests were performed. In Figs. 1-7 the load-settlement curves are shown.

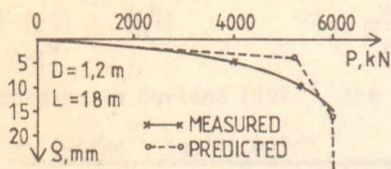


Fig. 1

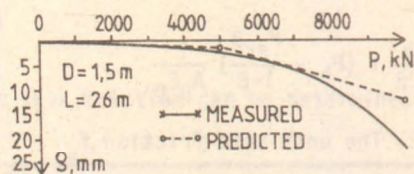


Fig. 2

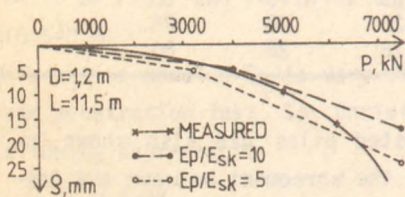


Fig. 3

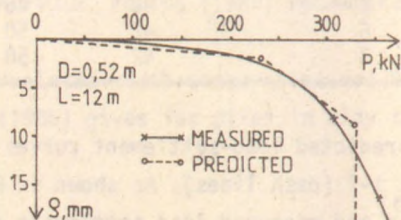


Fig. 4

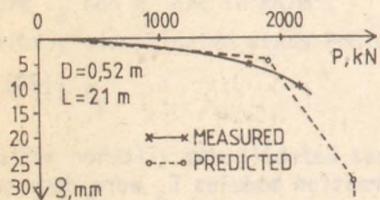


Fig. 5

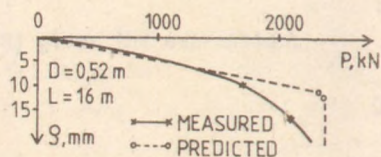


Fig. 6

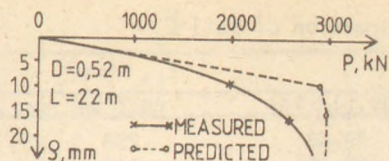


Fig. 7

Duncan and Chang (1970) proposed bilinear approximation of settlement ρ and load P on the pile:

$$P = \frac{\rho}{a+b\rho} \quad (15)$$

Using the same assumption, Maksimović (1981) gives the expression for the ultimate bearing capacity of pile:

$$P_f = \frac{1}{b} \quad (16)$$

and the ultimate load carried by pile shaft:

$$P_{sk} = \frac{\rho_c^2 b_1}{(a_1 + b_1 \rho_c)^2} \quad (17)$$

In Table 1 are given the values of the ultimate bearing capacity of tested piles according to the eqs. (1), (2) and (16), and compared with the values obtained from field load tests.

Table 1 The ultimate load of piles P_f

pille No	P_f , MN			
	eq.(1)	eq.(2)	eq.(16)	field test
1	19.17	10.84	8.30	6.00
2	36.68	15.30	12.70	9.87
3	19.60	7.50	6.90	7.50
4	0.61	0.74	0.44	0.33
5	3.28	2.20	3.13	2.60
6	2.15	1.42	3.68	2.40
7	2.64	1.84	4.00	3.00

In Table 2 are given the values of the unit skin friction f_s , determined by means of the equations (2), (3), (4) and (17) and compared with the values deduced from field load tests.

According to the method proposed by Poulos (1972), for seven tested piles load-settlement were constructed and the amount of load carried by the

Table 3 Modulus of deformation of soil E_s

pile No	E_s , MN/m ²		
	eqs.(12-13)	eq.(10)	field test
1	29-43	258	190
2	60-90	315	270
3	60-90		60
4	5-7,5	21	16
5	30-45	148	72
6	12.5-18.7	57	50
7	12.5-18.7	57	50

CONCLUSIONS

For the tested piles the values of the deformation modulus of soil E_s could be determined using $\alpha=12-22$ in the expression $E_s=\alpha R_p$;

Satisfactory agreement between predicted and measured load-settlement relationship, according to the method proposed by Poulos, was obtained.

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UPLIFT TEST RESULTS OF PILES

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INTRODUCTION

Pile foundations are generally used to support compressive loads. However, very often some structures are constructed on piles which have to resist uplift forces. The uplift resistance of pile is assumed to be dependent on the skin friction between the pile shaft and the soil. Generally, the uplift capacity of a pile is given by:

$$P_{sk} = \frac{1}{2} \gamma' L^2 \pi D K_s \operatorname{tg} \delta + G \quad (1)$$

where: P_{sk} = ultimate uplift load; γ' = effective unit weight of soil; L = embedded depth; D = diameter of pile; K_s = coefficient of earth pressure; δ = angle of pile friction, and G = weight of pile.

In several analytical approaches (Ireland, 1957; Meyerhof, 1973; Levacher and Siefert, 1984) attempts have been made to determine the earth pressure coefficients K_s to be used in the above expressions. Meyerhof (1973) introduced an uplift coefficient K_u . Vesic (1970) and Ismael and Klym (1979) suggested the use of the same value of K_u in compression and tension. Poulos and Davis (1980) recommended evaluating the values of K_u in tension by reducing to 2/3 of the values in compression. Chattopadhyay and Pise (1986) proposed the analytical model, depending on the slenderness ratio, the angle of shearing resistance ϕ of the soil, and pile friction angle δ .

FIELD LOAD TEST RESULTS

On three locations field load tests have been carried out with piles subjected to a vertical uplift load.

Pile N^o 1 was of diameter $D = 0.52$ m and of length $L = 18$ m. The average cone resistance in static penetration test was $R_{pav} = 3200$ kN/m². The results of

this test are shown in Fig. 1.

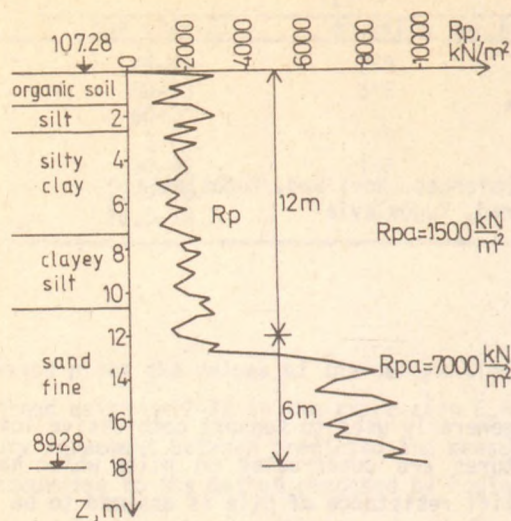


Fig. 1

During the field load test a maximum value of the applied uplift force was $Z_{\max}=350$ kN. The results of this test are shown in Fig. 2.

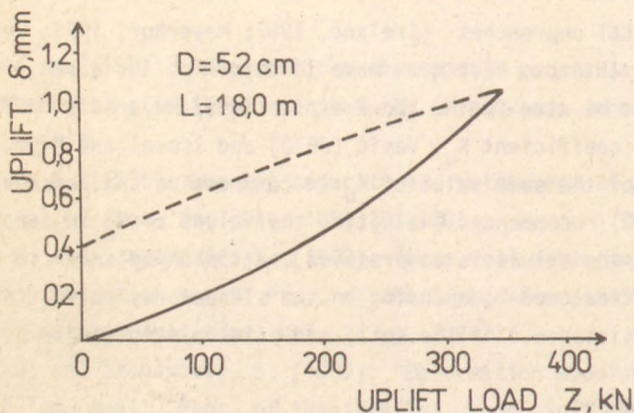


Fig. 2

Pile No 2 was a steel pipe fullfilled with concrete. The diameter of a pile was $D=80$ cm and its length $L=29.5$ m. In the vicinity of this tested pile a

standard penetration test was carried out. Using the relationship $R_p = 2N$, where R_p is the cone resistance and N is the number of blows, an average value of $R_{pav} = 7500 \text{ kN/m}^2$ was obtained. The results of standard penetration test are presented in Fig. 3.

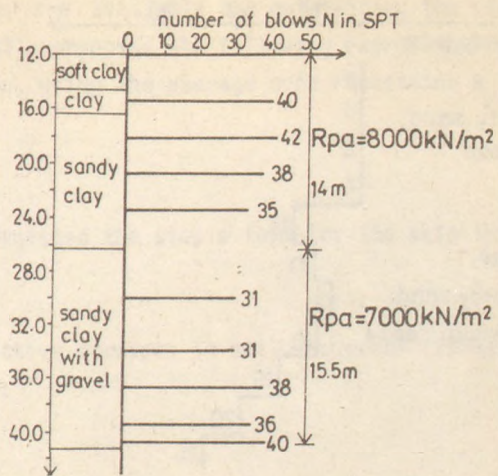


Fig. 3

The maximum value of the applied uplift load in this test was $Z_{max} = 300 \text{ kN}$. The obtained results are shown in Fig. 4.

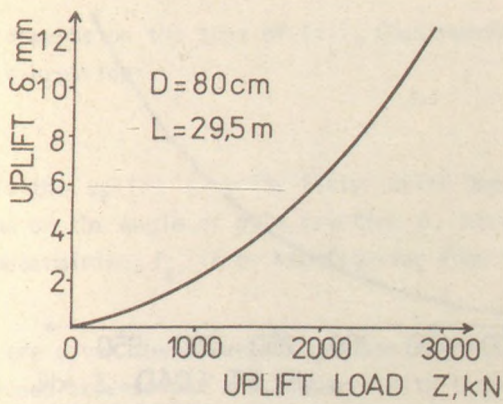


Fig. 4.

In the vicinity of pile No 3 of width $B = 0.285$ m and of length $L = 11.5$ m an average value of $R_p = 400$ kN/m² was obtained. The results of standard penetration test are shown in Fig. 5.

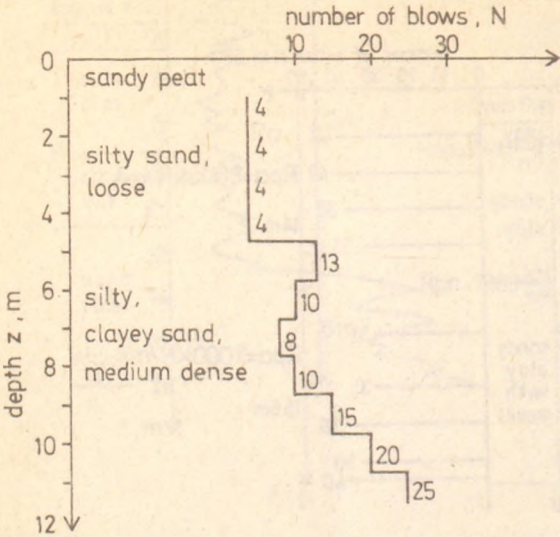


Fig. 5

The results of field load test for pile No 3 are shown in Fig. 6.

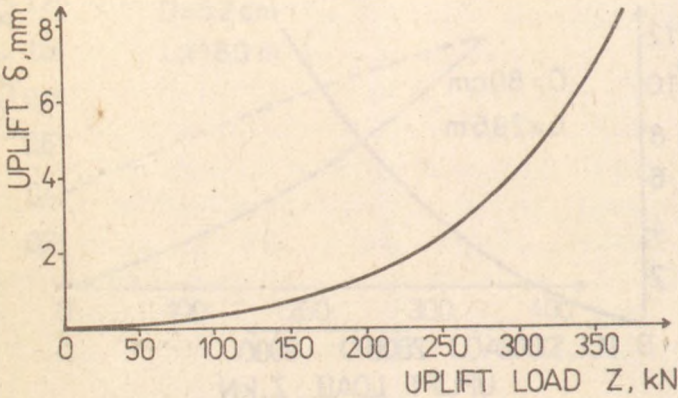


Fig. 6

ANALYSIS OF FIELD LOAD TEST RESULTS

The ultimate uplift load for each tested pile was determined by Van der Veen's procedure, using the results registered during the field load tests.

Several approaches are available for determining the shaft resistance f_s . Mohan et al. (1963) proposed the following expression for determining the unit skin friction, using the average cone resistance R_{pav} in CPT test:

$$f_s = \frac{R_{pav}}{50} \quad (2)$$

Burland (1973) suggested the simple form for the skin friction:

$$f_s = \alpha p'_v \quad (3)$$

based on the effective stresses in soil. Meyerhof (1976) proposed the following expression:

$$f_s = \frac{R_{pav}}{100} \quad (4)$$

whereas Bustamante and Gianeselli (1982) correlated the values of f_s with static cone resistance R_p , taking into account the type of soil:

$$f_s = \frac{R_p}{K_{sk}} \quad (5)$$

where K_{sk} depends on the type of soil. Chattopadhyay and Pise (1986) suggested the expression:

$$f_s = \gamma D A_1 \lambda \quad (6)$$

where A_1 is the uplift capacity factor which depends on the slenderness ratio λ and on the angle of pile friction δ . However, the most reliable method of determining f_s is by backfiguring from the results of pile load tests.

In Table 1 are given the values of unit skin friction f_s , determined by the above mentioned expressions and compared with those deduced from field load tests.

Table 1 Values of unit skin friction; f_s kN/m²

pile number	1	2	3
eq. (2)	64	150	80
eq. (3)	45-72	74-118	29-46
eq. (4)	32	75	40
eq. (5)	40-80	37.5-75	27-50
eq. (6)	13-49	30-77	7-30
load test	15	50	22

CONCLUSIONS

- The precision level of methods for determining the unit skin friction f_s from laboratory strength data is strongly dependent of the quality of the undisturbed samples;
- The expressions (2) to (6) give, in general, the overestimated values of f_s ;
- The values of f_s are very dependent on the angle δ of pile friction (eq. 6);
- In the presented cases of field load tests, the values of K_{sk} in eq. (5) are probably too low. These values obtained by backfiguring from the results of pile load tests are of the order of $K_{sk}=180-200$.
- Further investigations in this field are needed.

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The pile foundation inspection and strengthening.

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SUMMARY

The report deals with the experimental research results of the pile carrying capacities in the reconstructed building footings, including complex inspection of the foundation constructions and footing soils; obtained dependencies of pile carrying capacity variation by the data of their test in construction are given; the factors, influencing upon the pile carrying capacity values during the building-in-use period are systematized; the design procedure of the pile foundation reinforcement necessity is given; possible methods of the pile foundation reinforcement conformably to the soft grounds and water-deposited soils are analyzed.

INTRODUCTION

When reconstructing the city historical centers and reequipping the old factory buildings, defining the suburb architecture, it is often necessary to determine the reserves and real pile foundation carrying capacity.

It is known by the experience, that many uncertain moments occur when reconstructing the buildings on pile foundations under soft soil conditions. It is known, that under the long-period foundation loading, the soil changes its physical-mechanical properties in interpile space, including due to the thixotropic processes. The soil compresses in the pile foundation footing. At the same time, for a long period of the building operation some unfavourable factors would appear, deteriorating pile performance and the foundation soil quality, and in a number of cases, leading to the pile destruction (ground water level change can lead to the pile wood rotting, corrosive medium influence, the new foundation erections near the pile bed, additional loading, distributed on the area, etc.).

The pile carrying capacity utilization in revised parameter calculations provides rational reconstruction problem-solutions and gives good reasons for foundation loading increase.

RESEARCH IN THE RECONSTRUCTION SITE

The authors carried out the experimental research complex in the building reconstruction sites in Leningrad.

The inspection complex in sites contained of: pit arrangement for the state determination, degree of the grating physical wear, pile material; the foundation construction peculiarity revelation; specification of the site engineering-geological conditions by the soil samples, the static sounding performing; archives material study, the existing pile test by the static loading in the grating.

Taking into account the importance of pile carrying capacity revised values of pile test conducting in the existing building foundations without their unloading. The special equipment application allowed to eliminate the foundation carrying capacity decrease in the period of the test preparation and conducting.

The given method provides the pile test conducting without unloading. For this purpose, a metal frame, connected with the gating bottom half, is rigidly fastened on the pile, and, in such a way, the pile conservation is provided in the construction. Then, one carries out the pile section cutting off, the reinforcement bar trimming and the jack installation with the distributing packings. The jack system pressure is provided with the help of an ailstation. The pile transposition is fixed by the deflectometers. The test procedure stipulates the transposition control of the gating construction, serving as a stop. The loading steps and every step endurance time corresponded to the standard pile test requirements. On the termination of the test the metal frame fixed in a new position, providing connection with the gating. In the cut off pile plane the metal sheet was located, through its holes the free length of reinforcement bars was performed. The metal branch pipe, connected with the free length of reinforcement bars, was installed on the sheet and the monolith has been made of the pile section, including it into the combined operation with the pile gating construction.

The test sites description and the description of each of the sites are given below.

Site N 1. The industrial building with a fertilizer storage, the dimensions are of 192×60 m, the height is of 20 m. The long-period observations, carried out by LICI at the installation for the period of 1976—1989 made it possible to observe the settling development of the pile foundation building storage section, depending on the additional loading, distributed on the footing. The foundations under the building columns are made in the form of packing pile clusters (6—12 piles per a cluster), the piles are of reinforced concrete with prestressed reinforcement, dipping depth is of 9 m, the section is of 35×35 cm. The piles cut through the layers of soft peated sandy loams, clayey soils and strip clays, mainly of soft plastic consistency. Usually, a friction pile point is located in consistent sands, or in moraine tight-plastic clayey soils.

When operating from 1977 up to 1979, the additional loading, distributed on the building floors, as a result of long-period fertilizer storage in the "technological afterripening" process, constituted more than 62 kN/m^2 . Being onesided for the edge axis foundations and two-sided for the inner central axis foundations, the additional loading, distributed on the area, has led to occurrence and development of negative friction and the carrying construction irregular deformations, respectively.

It was noted, that after the footing unloading, some surface and pile foundation raising took place. Apparently, it was connected with elastic deformations of the soil mass seal failure. The observation data after 1986 demonstrate relative stabilization of the foundation settlements. Such unusual processes, noted in the period, previous to the reconstruction, complicated the task of further building usage after its reconstruction for a new technology. The main question of the possibility of additional load transfer to the existing piles remained obscure too.

To elucidate the real pile carrying capacity, the static test of a working pile, being a part of the gating, consisted of 12 piles, was carried out. By the test moment the tested pile has been under loading for 26 years.

The data of the pile tests, carried out immediately after their driving (1961) showed the carrying capacity of $F_d = 474 \text{ kN}$. The pile test in the given site, carried out 60 days later their driving, gave a little bit larger carrying capacity values (560 kN).

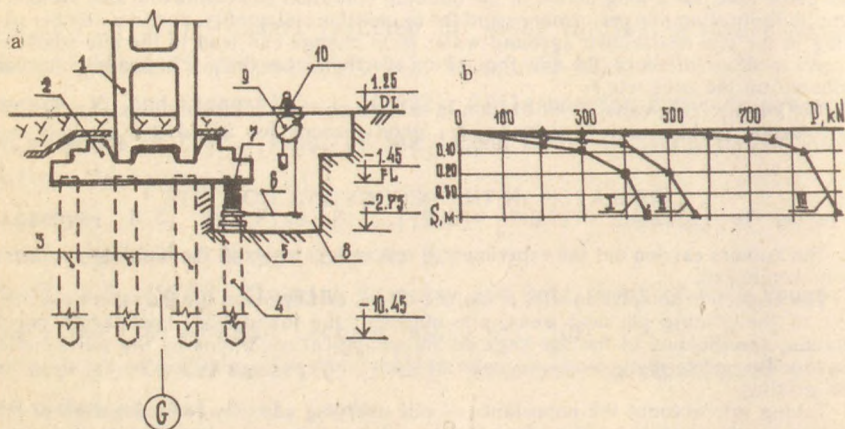


Fig. 1. — The construction test in the site N 1: a) the carried out test diagram; b) the pile

carrying capacity time variation: I — column, 2 — gating, 3 — pile = 9,0 m with the cross section of $0,35 \times 0,35$ m, 4 — tested pile, 5 — collar, 6 — thallium bars, 7 — jack, 8 — pit, 9 — deflectometers, 10 — bench mark system bearing beam; I — pile, tested in 30 days after driving; II — in 60 days after driving; III — after 26 years of operation as a component of the foundation.

The pile test without unloading, using standard test requirements, carried out by the authors directly in the building construction, demonstrated that the pile carrying capacity has been of 850—900 kN. (Fig. 1).

Site N 2. The four-storied brick dwelling building in the central part of Leningrad, in Poltavskaya street, was built in the middle of the last century. At the beginning of the XX-th century the building was superstructured by one storey. The building foundation is performed on wooden piles with the diameter of 26—30 cm, the depth of a pile driving is of 6 m in the form of a single-row strip; the gating is made of rubble with lime-sand mortar.

The engineering-geological conditions are characterized by successive interchange of contemporary and lake-glacial deposits: filled up ground to the depth of 1—2 m, a layer of brown, low-decomposed turf, with the capacity of 1,6—2,3 m; dusty plastic sandy loam with plant residues; Moraine deposits in the form of thick dusty sandy loam with inclusion of fine ships and grass up to 30% are fished from the depth of 7,5—8,0 m. Practically, the wooden piles did not achieve solid moraine deposits.

The inspection showed, that the building deformation took place because of the following reasons: the total head rotting of wooden piles and sleepers; local soil removal from under the gating with loosening of material carrying capacity of some part of the piles.

When inspecting pitting was performed for the purpose of technical state fixing of the rubble strip gating wooden piles. To specify the soil states in the building footing the well dwelling together with the soil sampling were carried out near the foundation; static and dynamic soundings were carried out too.

Designed by the results of the direct tests in the construction, the pile carrying capacity (connected with soil) has formed up to 270 kN on the moment of the inspection, which is twice as much as the calculated value — 140 kN (Fig. 2).

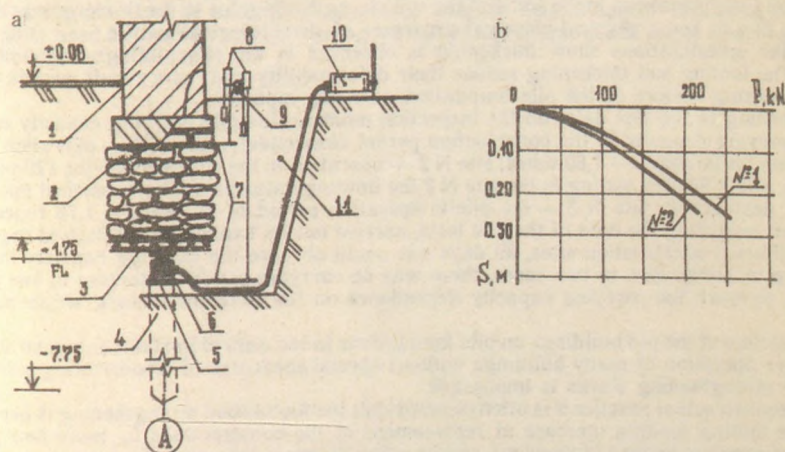


Fig. 2. The pile test in the strip pile foundation construction.

a — test diagram; b — test results in the form of the plots of "loading versus settling"; 1 — brick wall; 2 — rubble gating; 3 — distributing beam; 4 — tested pile; 5 — collar; 6 — jack; 7 — metal bars; 8 — deflectometers; 9 — bench mark beam; 10 — oil station; 11 — oil supply branch pipes.

Site N 3. The brick office building with the dimensions of 11×60 m, the height is of 8,85 m, was built as the beginning of the 60-th. For the period of operation the building received irregular settlements along the longitudinal axis, which became the cause of crack net formation. While solving a problem of the building destination change and the further operation expediency, the complex inspection of the constructions and the building footing soils was carried out, the pile tests in the foundation construction were carried out too.

The building foundations are made in form of clusters of four, eight piles. The dipping depth is of 6,0 m.

In the geological respect the section represents technogenetic deposits with the great capacity up to 3,2 m; biogenetic deposits — turf with the capacity of 3,6 m; which are laid under by the plastic consistency strip loams with the capacity of up to 8,6—10,8 m. The piles did not reach moraine deposits, represented by the dusty sandy loams with gravel and pebble.

Pitting provided the possibility to specify the pile foundation state and construction. The gating is of monolithic reinforced concrete, the piles are wooden with the diameter of 220—250 m. On the moment of the inspection their state was satisfactory.

Taking into account the inspection results and the necessity to solve the problem of the foundation strengthening, the pile test in the construction has been carried out. According to the test results the following pile loading limiting values were received: pile N 1 — 150 kN with the settling of 53,9 mm; pile N 2 — 125 kN with the settling of 40,24 mm.

Thus, one came to the reasonable conclusion, that according to the new increased loadings it was necessary to strengthen the pile foundation for the sake of normal building operation.

DISCUSSION OF THE INVESTIGATION RESULTS

The investigators, studying the soil thickening round the piles, offer different basing ways of the driving pile carrying capacity time variations in the dusty-clayish soils. The offered dependencies of the pile carrying capacity increasing descriptions provide similarity, necessary for practical purposes in the period of up to 60 days, which is not enough for the building reconstructions after their being-in-use for a long time.

The investigations in Leningrad reconstruction sites, carried out by the authors, give the opportunity to conclude, that the carrying capacity increase is the irreversible process, depending upon a number of factors, such as: quantity of load, taken by the foundation piles; loading character (cyclic, dynamic loading effect), hydrological regime change.

While analysing the forming mechanism of the pile carrying capacities under reconstruction (as a rule, after a long period (10—15 years) of the pile operation in construction), it can be noted that the pile carrying capacity increasing takes place as a result of numerous water-coagulation link formations in the soil and the soil strengthening due to the thixotropic processes and, also, due to some physical-chemical processes of the lithological nature.

As the investigations show thickening is observed in the pile footings and under the gatings. The footing soil thickening reduce their deformability and, as a result, serves as one of the increasing factors of the pile foundation carrying capacity.

According to the test data, on the inspection moment, the pile carrying capacity exceeds the pile carrying capacity in the construction period respectively: site N 1 — operation in the construction for 26 years — 1,89 times; site N 2 — operation in the construction for 120 years — 1,92 times, while the pile testing in the site N 2 the limiting value hasn't been reached due to the pile wood destruction; site N 3 — the pile-in-operation period of 28 years — 1,79 times.

When analyzing the data of the pile tests, carried out by Leningrad specialized organizations in different construction sites, 39 days one could observe the carrying capacity increase from 25 up to 110%. But, in two cases, there was no carrying capacity increase in one site. It allows us to mark the carrying capacity dependence on the different factors, which must be studied.

Inspection of the old buildings on pile foundations in the central part of Leningrad showed, that further operation of many buildings without special measures of wood rotting protection or special strengthening works is impossible.

In reconstruction practice it is often occurred that the foundation strengthening is necessary due to the footing loading increase at replacement of the constructions by more heavy ones (reinforced concrete instead of wooden), construction diagram change, equipment arrangement, when the footing carrying capacity becomes insufficient.

Strengthening can proceed from the established carrying capacity deficiency, which can be determined by the formula:

$$F_d^D = F_d^N - F_d^f$$

where F_d^D — carrying capacity deficiency, kN;

F_d^N — required carrying capacity, necessary for loading perception after reconstruction;

F_d^f — real pile foundation carrying capacity on the moment of inspection;

on condition $F_d^N = F_d^f$ — strengthening is not required.

To evaluate the possibility of additional loading transfer to the pile foundations it is also necessary to carry out the deformation calculations

$$S_a \leq k_s \cdot S_u$$

where S_a — additional absolute settling from the loading increase under reconstruction, m;
 S_u — the given building type limiting deformation in form of absolute settling (by norm);
 k_s — limiting deformation quantity reduction coefficient, depends on the construction states and the building operation period. k_s definition is carried out by analogy with the method, developed for the foundations on natural bed.

Depending on the calculations, the possible strengthening variants can be traditional: replacement of the rotted wooden pile sections by reinforced concrete spikes; foundation construction strengthening by the additional piles, performed according to different technologies, and new ones: city borer-injecting piles, recently developed according to new technology.

Using of one or the other method of strengthening depends on the concrete reconstruction conditions, the gating and pile state, the type and state of the soils in the building footing, technical-economical indexes.

CONCLUSIONS

1. The pile tests in the gating construction, intended for their carrying capacity determination, showed, that in dusty-clayish soils and sands the piles had great real carrying capacity in comparison with the calculation and test data in the process of construction in the sites, where such tests had been carried out.

2. Received carrying capacity reserve can be realized while the reconstructing buildings — and also, for the guarantee of architecture memorial conservation.

3. As extreme measure, the method of strengthening by the increased carrying capacity borer-injecting piles can be used. It is performed with the help of special technology, which is successfully used for conservation of Leningrad center buildings, which are of great value from the point of view of History.

PILE FOUNDATIONS OF LARGE DIAMETER WITH PRECAST DRIVING POINT

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1. INTRODUCTION

The pile foundations are generally an efficient solution for constructions where soil in the shallow-foundation zone is of poor heaving capacity, or settlement problems are anticipated. The piles may be built by several procedures. The piles of large diameter are usually the cast-in-place piles, formed by making a hole in the ground and filling it with concrete. The hole may be drilled, but more often is formed by driving a shell or casing into the ground. The casing may be filled with a mandrel, after which withdrawal of the mandrel empties the casing. The casing may also be driven with a driving tip on the point, providing a shell that is ready for filling with concrete immediately, or the casing may be driven open-end, the soil entraped in the casing being jettied out after the driving is completed.

In this paper a technology of the cast-in-place piles execution is described, used for the founding of the multi-storey-buildings. The pile load test data are presented too, for a compression test.

2. SOIL DESCRIPTION

The soil is characterized by the layers of poor bearing capacity (unconsolidated fill, peat and organic silt) on a variable depth of 6,50 m to 8,50 m. At this level a gravelsand layer is located which is a firm soil. The ground water is located at different depths from 0,40 m to 3,50 m. The water is intensely corrosive to pile concrete

3. FOUNDATION SOLUTION

Taking into account the given site conditions, an economical study has been carried out having in view two foundation solutions on caissons with diameter between 1,70 m and 2,70 m made up of precast concrete elements of 0,50 m length each, and on large diameter piles. The study results lead to the conclusion that later solution is economically advantageous.

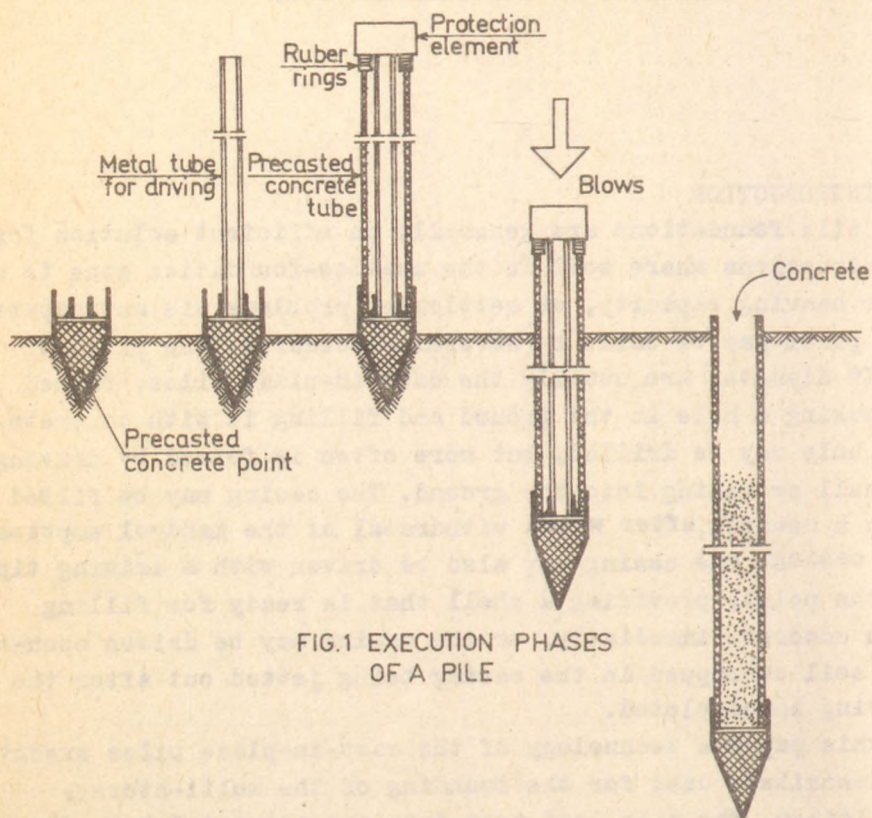


FIG.1 EXECUTION PHASES
OF A PILE

The technology of the chosen solution, though a classical one, has several original features and consists of following (Fig.1): a conical hole is dug on the pile location and a precast reinforced concrete point is put in place. On the concrete point a driving metallic tube is mounted (having a smaller diameter) and another centrifugated reinforced concrete tube of the same diameter as the concrete point is mounted on the point containing the smaller diameter tube. The tubes are vertically checked and fixed to pile driver.

A protection device is fixed on the upper end of metallic tube which is 0,50 m longer than the concrete external tube. The difference in height of the two tubes is filled up with rubber rings that protect the concrete tube during the driving.

The precast point is progressively introduced in the ground by blows applied to the metallic tube.

In the same time the external reinforced concrete tube enters the ground.

When the first tube-element levels the ground, another element is mounted over it, the metallic tube is extended and the process is continued down to the foundation level. The new mounted concrete tubes are progressively shorter to avoid the tube losses when the final level is reached. At this stage, the metallic tube is pulled out, the inside of the concrete tube is checked using searchlight, if reinforcement is found necessary an extra reinforcement is introduced and the space is filled up with concrete.

Using this procedure, pile-foundation has been built having 0,70 m diameter and up to 10,00 m height.

In view of corrosion protection against the underground waters, the concrete tubes have been beforehand insulated and made waterproof by using an adequate material. This water-proof insulation and the relatively short time of pile-driving (approximately 30 min.) ensured against the water leakage inside the tube before the concrete pouring.

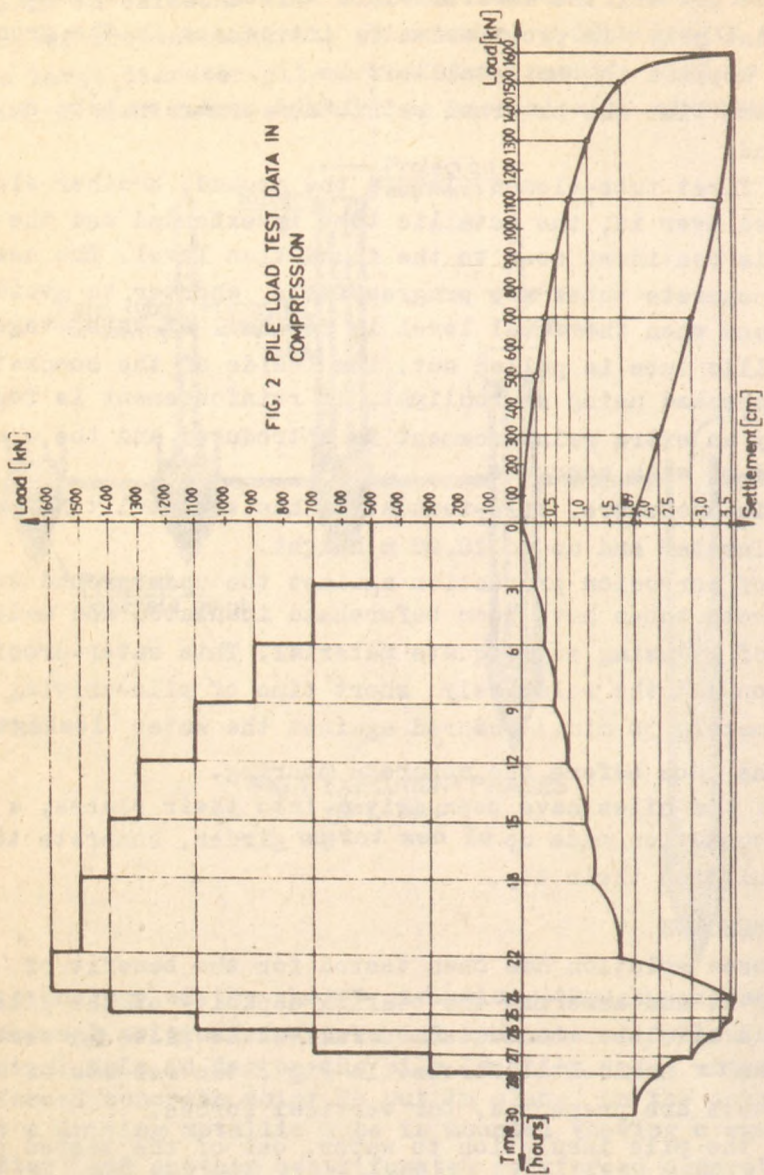
After all the piles have been driven into their places, a proper foundation made up of new torque girder, concrete top, etc is built on their top.

4. TEST RESULTS

The proposed solution has been tested for the benefit of improving it and asserting the bearing capacity of the piles. More tests have been conducted on full-scale piles for vertical as well as for horizontal forces. In Fig.2 the results of one of the tests are presented, for vertical forces.

To check the pile insulation to water, one of the tested piles has been left without pouring concrete inside it for three days, leading to the conclusion that the used water-proof solution is entirely reliable.

The testing diagram from Fig.2 shows a favourable behaviour



behaviour of the pile: a high bearing capacity and relatively small settlement.

The following technical and economical advantages are offered by the used solution:

- reduced concrete volume;
- the diggings and the pumping of the underground water are eliminated;
- improving the soil around the piles;
- the work-volume is significantly decreased;
- the possibility of pile-protection against the corrosive water;
- increased labour efficiency.

The buildings constructed on the above described foundation solution show a good behaviour with time.

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ON THE INTERACTION BETWEEN DEEP FOUNDATIONS AND A SUBSOIL ON THE MINING AREA

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INTRODUCTION

The aim of this contribution is to propose and discuss an idealized soil-foundation interaction model which takes into account the depth effect due to the mining subsidence and an unelastic behaviour of a subsoil under the loading-unloading processes. The model is applied to the interaction analysis for the embedded rigid foundations on the mining area.

Throughout the paper subscripts i, j, k, l run over $1, 2$; summation convention holds. The dot over a letter denotes time differentiation, ∇^2 is Laplace's differential operator in the cartesian coordinates $x=(x_1, x_2)$ and $e(\cdot)$ is a function given by: $e(a)=0$ if $a \leq 0$, $e(a)=1$ if $a > 0$.

INTERACTION MODEL

Let Ox_1x_2z be a cartesian coordinate system such that the half-space $z \geq 0$ represents the subsoil in its reference (original) state. The subsidence surface at a time instant t let be given by $z=s(x, t)$, $x=(x_1, x_2)$; we assume that $s(x, t)=0$ for every $t \leq t_0$. By $z=H_0$ we denote the foundation level, F stands for the orthogonal projection of a foundation footing region on a coordinate plane $z=0$ and $q(x, t)$, $x \in F$, be a normal stress distribution acting from the foundation on the subsoil (contact stresses). By $u(x, z, t)$ and $u_1(x, z, t)$ we denote vertical and horizontal displacements in the subsoil, respectively. Following [3, 4] we introduce the displacement constraints in a foundation layer (H_0, H) , setting $u(x, z, t)=w(x, t)h(z)+s(x, t)$, $u_1(x, z, t)=-g(z)$.

$s_1(x,t)$, $z \in (H_0, H)$, where $g(z)$, $h(z)$ determine postulated variations of displacements in the vertical direction, such that $h(H_0)=1$, $h(H)=0$. The relative deflection $w(x,t)=u(x, H_0, t)-s(x,t)$ on the foundation level will be treated as a basic unknown. By M_0, M we denote the primary and secondary soil compression modulae, respectively, and by G the shear modulus of a soil. We also introduce the parameters

$$c_1 = \frac{G}{2} \int_{H_0}^H h(z)(1-g'(z))dz, \quad c_2 = \frac{G}{2} \int_{H_0}^H h(z)^2 dz, \quad c = c_1 - c_2,$$

$$C = M \int_{H_0}^H (h'(z))^2 dz, \quad C_0 = M_0 \int_{H_0}^H (h'(z))^2 dz$$

and the function

$$d(x,t) = -q(x,t) - C_0 w(x,t), \quad d(x,t) \geq 0, \quad (1)$$

which will be used below. On the basis of the principle of virtual work and introducing the simplified constitutive equation for the normal stresses, after some algebras we obtain the following incremental form of the idealized soil response to the contact stresses

$$\dot{q}(x,t) = [C - (C_0 - C)e(d(x,t))e(\dot{w}(x,t))] \dot{w}(x,t) - c_2 \nabla^2 \dot{w}(x,t) - c_1 \nabla^2 \dot{s}(x,t). \quad (2)$$

Eq.(2) has to be considered together with the pertinent boundary and initial conditions. It has to be emphasized that the model proposed can be applied only if $0 \leq q(x,t) < q_L$, where q_L is a limit contact stress.

For the surface foundations, where $H_0=0$, the shear stresses in a foundation layer due to the subsidence, equal to $0.5G(1 - g'(z))\text{grad}s(x,t)$, can be neglected; it follows that $g'(z)=1$ for $z \in (H_0, H)$ and hence $c_1=0$. It means that for the surface foundations the term in Eq.(2) depending on the subsidence $s(x,t)$ has to be concealed.

For the embedded (deep) foundations the horizontal displacements $u_1(x,z,t)=g(z)s_1(x,t)$, $z \in (H_0, H)$ in a foundation layer can be treated approximatively as independent of the z -coordi-

nate; hence $g(z)=\text{const}$ for every $z \in (H_0, H)$. It means that for the deep foundations in the definition of a parameter c_1 we have to substitute $g'(z)=0$ and the function $g(z)$ does not enter the model proposed.

For the shallow foundations we obtain a certain intermediate case which will be not discussed here.

APPLICATION TO DEEP RIGID FOUNDATIONS

In most cases the subsidence surface under the footing F can be approximated by $s(x, t) = 0.5K_{ij}(t)x_i x_j + k_i(t)x_i + k(t)$, $x \in F$; hence $\nabla^2 s(x, t) = K_{ii}(t)$ for every $x \in F$ and $\nabla^2 s(x, t)$ coincides in F with the mean curvature $K(t) \equiv K_{ii}(t)$ of the subsidence surface at $x=0$. At the same time for the rigid foundations we have $u(x, H_0, t) = v_i(t)x_i + v(t)$, $x \in F$. Hence the relative deflections on the foundation level $z=H_0$ are

$$w(x, t) = v_0(t) + v_i(t)x_i - k_i(t)x_i - \frac{1}{2}K_{ij}(t)x_i x_j \quad (3)$$

under the extra denotation

$$v_0(t) \equiv v(t) - k(t).$$

The basic unknowns are $v_0(t)$, $v_i(t)$ and $k_i(t)$, $K_{ij}(t)$ are assumed to be known in every problem. In order to simplify the considerations we shall also assume that the region F , representing a foundation footing, has two symmetry axes coinciding with the coordinate axes $x_i=0$. Let us define

$$A = \int_F dx_1 dx_2, \quad J_{ij} = \int_F x_i x_j dx_1 dx_2,$$

$$\tilde{A}(a) = \int_F e(d(x, t)) e(a) dx_1 dx_2,$$

$$\tilde{J}_{ij}(a) = \int_F e(d(x, t)) e(a) x_i x_j dx_1 dx_2$$

for an arbitrary real a ; we have to remember that $\tilde{A}(a)$ and $\tilde{J}_{ij}(a)$ also depend on the right-hand side of Eq.(1). For the sake of simplicity let us also assume that the resultant verti-

calforce and resultant moments acting from the structure on the rigid foundation are constant in the time interval (t_0, t_f) and let us neglect the inertia forces due to the motion of the subsidence basin. Hence

$$\int_F \ddot{q}(x, t) dx_1 dx_2 = 0, \quad \int_F x_i \ddot{q}(x, t) dx_1 dx_2 = 0$$

for every $t \in (t_0, t_f)$. Inserting the RHS of Eq.(2) into the above equilibrium conditions, after rather lengthy calculations we obtain $v_i(t) = v_i(t_0) + k_i(t)$, $t \in (t_0, t_f)$ and the following governing equation for the unknown deflection $v_0(t)$:

$$(CA + (C_0 - C)\tilde{A}(\dot{v}_0(t)))\dot{v}_0(t) = c\dot{A}K(t) + \frac{1}{2}(CJ_{ij} - (C_0 - C)\tilde{J}_{ij}(\dot{v}_0(t)))\dot{K}_{ij}(t), \quad t \in (t_0, t_f), \quad (4)$$

where $\tilde{A}(\dot{v}(t))$ and $\tilde{J}_{ij}(\dot{v}(t))$ also depend on the distribution of $e(d(x, t))$ in F for an arbitrary but fixed $t \in (t_0, t_f)$. Eq.(3) now yields

$$w(x, t) = v_0(t) - v_i(t_0)x_i - \frac{1}{2}K_{ij}(t)x_ix_j. \quad (5)$$

The initial values of $v_0(t)$, $v_i(t)$ at $t=t_0$ have to be calculated from the equilibrium conditions for $d(x, t_0)=0$. The solution $v_0(t)$, $t \in (t_0, t_f)$ to the nonlinear differential equation (4) can be obtained via the step-by-step procedure; on every step we have to calculate the contact stress distribution and then to determine the distribution of $e(d(x, t))$ in F . To this aid we use Eqs.(5) and (2) and introduce the discretization of the time interval (t_0, t_f) .

If the foundation layer can be treated as elastic then $C=C_0$ and Eq.(4) leads to

$$\dot{v}_0(t) = \dot{v}_0(t_0) + \frac{c}{C}K(t) + \frac{1}{2A}J_{ij}K_{ij}(t), \quad t \in (t_0, t_f);$$

from Eqs.(5) and (2), under the aforementioned assumptions, we obtain

$$q(x, t) = C(v_0(t_0) - v_i(t_0)x_i) + \frac{C}{2A}(J_{ij} - Ax_ix_j)K_{ij}(t).$$

This result is valid for deep as well as shallow foundations.

CONCLUDING REMARKS

The interaction analysis for the rigid surface foundations, based on Eqs.(1)-(4) (under assumption $c_1=0$ but taking into account the unelastic properties of the subsoil) was carried out in [5], where the solutions to some special problems can be found. Inserting the RHS of Eq.(2) into the known beam or plate equations (rewritten in the incremental form) we formulate the interaction problems also for deformable flexible footings on the mining areas; these problems are now under consideration. Neglecting the nonelastic properties of a subsoil we have $C=C_0$ and from Eq.(2) we obtain

$$q(x,t) = Cw(x,t) - c_2 \nabla^2 w(x,t) - c_1 \nabla^2 s(x,t), \quad (6)$$

where the term depending on the subsidence $s(x,t)$ has to be taken into account for the deep foundations and neglected for the surface foundations. Thus we conclude that for an elastic foundation layer and surface foundations we arrive at the known Vlazov model of an elastic layer, [3,1]. On the other hand, neglecting in Eq.(2) the effect of shear strains (i.e. setting $c_1=0$ and $c_2=0$) we obtain the model of a partly elastic subsoil proposed in [2] but with all parameters related to the intrinsic material properties of the soil.

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SECTION 3
SURFACE SUBSIDENCE

Numerical Estimation of the Effectiveness of Measures to Stabilize the Ground Massive when Tunneling in the Vicinity of Buildings.

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When constructing subway tunnels in the regions of urban development severe requirements are made to preserve the buildings and structures which are found within a zone of tunnel driving. In particular, these requirements consist in allotting maximum allowable settlements a building may have.

The present paper considers the tunnel driving effect on settlements of neighbouring buildings. In addition, it describes the calculation methods from the viewpoint of reducing the settlements and measures to stabilize the ground massive.

The above measures to reduce additional settlements may, for example, consist in preliminary chemical grouting of soil massive as well as in choosing the best way of tunnelling and in making various kinds of barrier structures in the massive near a building. The estimation of the effectiveness of such measures is evidently possible by means of solving comprehensive mechanical problems [3].

The estimated settlements of soil base surface as well as predicted sizes of ground massive destruction by which is possible to judge the progress of settlements can serve as the quantitative characteristic engineering oriented estimation of the effect of tunneling on the neighbouring buildings. The analysis of these solutions allows to determine some rational scales of the chemical grouting regarding other measures undertaken as well as to estimate the expediency of using other know-how practices proposed by the contractors. Due to substantial inhomogeneity of the foundation resulted from the

intrusion of various reinforcing elements into it to solve the mechanical problems is possible exclusively numerically, for example, by the finite elements method (FEM) [2].

Consider now the calculations to estimate the sufficiency of measures of soil massive stabilization in subway tunneling near historically precious building.

According to the design at the construction of the subway in a soviet city the tunnels were to be driven at a small depth (from 12m to 17m) and at a small distance (about 2,5m) from the corner of a building for which a quantity of the ultimate additional settlements was designed equal to 5 mm. To prevent subsidences in the neighbourhood of buildings the soil was stabilized with calcium silicate solution. In addition to the chemical grouting 2 lines of bored-injected piles were placed in a chess order at a distance of 0,5m from each other along the route of the subway. These piles were made by means of boring up to the tunnel bottom mark and by a subsequent cement grouting mortar injected through the whole depth of the bore-hole. Furthermore, it was suggested that a retaining wall made of cast-in-situ reinforced concrete piles having a diameter of 0,7m and a pitch of 1m should be installed to provide some additional strengthening for the building foundation.

To estimate the sufficiency of measures to stabilize the building soil base as well as the practicability of the retaining wall construction some analyses of the settlements and the zone of failure within the soil massive in the vicinity of the tunnel were carried out for different stages of the work and the stabilization alternatives.

The quantity of additional settlements caused either by works accomplished or by technological measures can be determined as a difference of elastic displacements of the soil base surface beneath the structure, which were computed for the conditions existing before the outset of tunnelling and after completion of the work.

The outcome of the settlement observations carried out from the outset of dewatering and silicification of the soil base revealed no additional displacement of the building as a result of the work. Hence, it is not necessary to determine a

stress-strained state of the soil massive before the work on stabilization and to confine yourself to the determination of displacements for three soil base computational schemes which are as follows:

1. The stabilized ground massive without a working.
2. The stabilized ground massive with a working.
3. The stabilized ground massive with the working and the retaining wall made of cast-in-situ concrete piles.

The additional displacement of a building as a result of tunnel driving are defined as the difference of displacements calculated according to the 2-nd or the 3-d computational scheme (depending on the stabilization scheme) in comparison with the displacements according to the 1-st initial computation scheme.

Because of the lack of some reliable data regarding the depth of the silicified zone, the analysis of an effect of the grouting depth H_x was based on the computation accomplished for the case when the grouting was carried out up to the subway tunnel bottom mark (17m) and to the depth of 15 m at which the presence of silicate was proved by the control drilling of all check boreholes.

For estimation of dimensions of zones of failures it is necessary in every element of the FEM scheme to compute a quantity of a "nonfailure parameter" ψ , which depends on the acting stresses and the material specification presented in the element.

As the subway tunnel has a great extent, the problem under consideration can be approximately solved for conditions of plane deformations. Since the longitudinal axes of the building and the subway tunnel are not in parallels, the adopted idealization will yield somewhat overstated values for the stress acting into the soil massive which also counts in a margin of safety. The computation based on FEM can be practically realized only assuming that the soil displacements do not occur at the distance of more 5 radii of the workings from the centre. This is necessary to specify dimensions of the computational region. The upper boundary of the region is at the building foundation bottom level. Since a structure settlement is determined as a difference between the displacements of

the surface points computed for different stages of the work, so the errors caused by use of a plane deformation scheme and a limitation of the computational region dimensions turn out to be less essential.

The foundation effect on the soil base was simulated by an equally distributed load $P = 0,2$ MPa. The pressure of the soil mass above the bottom level of the cathedral foundation was simulated by equally distributed load $q = \gamma H$ ($H = 3$ m, $\gamma = 18$ kN/m³). In the computations there were used the rectangular Cartesian coordinates which centre coincides with the centre of the working. The low boundary of the computed region is at the distance of 16,5 m from centre of the workings (the radius is 2,8 m), the upper boundary of the region has coordinate $z = 11$ m. Both the right and the left boundaries of the region are at the distance of 15,5 m from the axis of symmetry. Mechanical characteristics of the elements were specified according to the data obtained from the survey, pressuremeter tests as well as to the tests of the grouted soil samples.

For a non-grouted soil: the modulus of deformation $E = 30$ MPa, the Poisson's ratio $\nu = 0,3$, weight by volume $\gamma = 18$ kN/m³, the internal friction angle $\varphi = 35^\circ$, the specific cohesion $C = 1,82$ MPa. For silicified soil: $E = 100$ MPa, $\nu = 0,25$, $\gamma = 22$ kN/m³, $\varphi = 35^\circ$, $C = 1,82$ MPa. For a zone of a sheet bored-injected pile screen ($\delta = 0,65$ m) the mechanical characteristics were evaluated in relative volumes of pile infillings spaced by cement mortar and soil and were found to be as follows: $E = 61$ MPa, $\nu = 0,2$, $\gamma = 22$ kN/m³, $\varphi = 35^\circ$, $C = 2,5$ MPa.

A stress-strained state of the soil base is formed by an external load (a pressure beneath the foundation bottom and a superimposed load created by the soil layer above), as well as volumetric forces defined by the dead load of the soil.

To check up the correctness of an algorithm for FEM, a test problem concerning a determination of stresses arising in homogeneous ground mass as a result of a dead load was solved. The error was less than 1% in comparison with the known precise solution.

The results of computations on the basis of FEM were horizontal and vertical displacements of every computational scheme

element as well as the components of stresses $\sigma_x, \sigma_y, \tau_{xy}$, in a point corresponding to this element. Besides, in every element a quantity Ψ was computed in order that it could be then used in defining dimensions of failure zones:

$$\Psi = 2c \cos \varphi + (\sigma_x + \sigma_y) \sin \varphi - \sqrt{(\sigma_x - \sigma_y)^2 + 4\tau_{xy}^2}$$

There is rather effective and widely used method to define approximately the failure zone from the conditions that within their boundaries a quantity of the parameter Ψ is negative [1].

As mentioned above, primarily a stress-strained state of the building soil base was determined before the workings had been started. The computations revealed that in this case (as it should be expected) the failure zones were not formed. At this stage of the computation vertical displacements of the massive points in contact with the building foundation were obtained too. These quantities were then used to define relative displacements of the building resulting from the workings.

Then the version with the workings was considered when the grouting was performed both up to the subway tunnel chute bottom and up to the depth of 15 m. The analyses revealed that at non-complete massive grouting the failure zones were formed beneath the subway tunnel bottom. The dimensions of these zones were determined for the conditions when the installation of the face support lagged behind the working digging. To be sure, the formation of failure zones can be prevented owing to the employment of shield driving and a rigid support. However, to provide a reliable prevention of a progressive development of these zones at all stages of the work and the resultant uncontrolled settlements of the building it is necessary to carry out the chemical grouting of the ground mass at least up to the subway tunnel chute bottom.

There was analysed a question: how a stress-strained state of the soil base was changed when an additional sheet screen of cast-in-situ concrete piles was erected. It turned out that such an expensive construction did not practically improve the situation, but on the contrary it increased, though slightly, the dimensions of the failure zones beneath the tunnel. Further-

more, in the know-how aspect, the erection of such constructions in the vicinity of a structure involves great difficulties and may have an unfavourable effect on the building.

Thus the results of the analyses make it possible to estimate a quantity of additional building settlements caused by the tunnel driving. Here the vertical displacement of the ground mass points in contact with the foundation base is assumed as a reference point. The displacement of a building point located on the boundary of the computational region without working was 283mm with the vertical displacement of its corner being 208mm. In case of the complete chemical grouting zone and the presence of the working these conditions are equal to 285 mm and 212 mm respectively, and in case of grouting up to the depth of 15 m these quantities were 289 mm and 217 mm respectively. Hence, in case of non-complete grouting of the ground mass and the installation of the face support fulfilled behind the working face digging, the additional settlements may be as much as 6-9mm. The quantity is slightly more than an allowable one and this circumstance puts some special restrictions on the driving know-how practice. Besides, in chemical grouting of the ground mass up to the depth of 15 m, the release of soil into the workings is possible and this might lead to some progressing settlements of the soil base and the building failure.

In conclusion it may be pointed that when choosing a know-how practice to stabilize the soil base it should be taken into account specific factors which make it difficult its accomplishment under the urban conditions, for example the existing underground service lines, the impossibility of using large urban territories, etc.

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SUBWAY TUNNEL WITH MULTIPLE FUNCTIONS ON THE BALCESCU BOULEVARD, BUCHAREST

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The construction of the Bucharest subway started in 1975 and up to date (May 1990) three thoroughfares summing 60 km of double line and 38 stations were put into exploitation (fig. 1).

With two connections in progress and a fourth thoroughfare in the design stage, another 20 km of double line will be added.

Alluvium-type soils, consisting of alternate, discontinuous layers of sands, clays and silty clays, determined by the two main rivers crossing the town in the N.W.-S.E. direction, are characteristics for the lithology along the all subway lines.

The level of the phreatic water is located between -3.50 and -7.00m.

Taking into account both the soil conditions and the capabilities of the contractor, the solution to build a subway of small depth was adopted.

Stations and tunnels built in open excavation have a minimum cover of 1.50m, while the shield-type tunnels have a minimum cover of one diameter or even less.

The main advantages of this solution are: smaller costs, reduction of material consumption, decrease of the construction time and most convenient passenger excess to the subway stations.

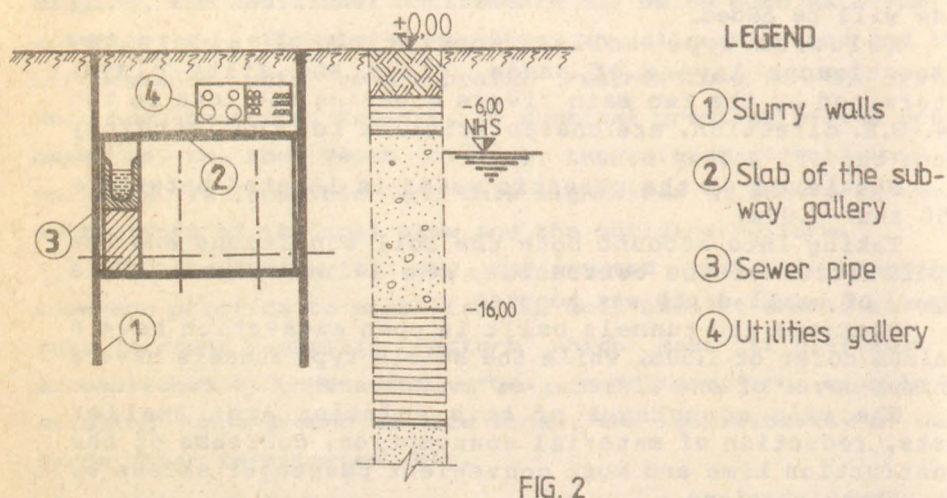
In choosing the construction procedures, the function of the urban traffic (cars, tramcars, trolleybuses), the uninterrupted exploitation of underground networks (water supply, sewage, gas, etc.), as well as the stability of the buildings located in the vicinity of subway works, have been carefully considered.

About 50% of the 60km of subway have been built in underground excavation by using 6.50m diameter romanian made shields.

The rest of the tunnel was a rectangular gallery for two lanes, built either in open excavation or in covered excavation under a subsurface slab (in order to allow short restoring of the street traffic).

Regardless of the tunneling procedure, the water table was lowered by means of deep wells. Extraction of solid material exceeding 10mg/l during the pumping was not allowed.

Special problems were put by the construction of the subway tunnel on the Balcescu Boulevard, in the very centre of the Capital town. Buildings of 7 or more storeys, most of whom were erected before 1940, had to be protected, in order to the architectural feature of this beautiful avenue. The twin tunnel solution was rejected, due to ground subsidence risk by deep wells functioning too close to the buildings. Preference was given to a multipurpose rectangular tunnel, including the two subway lanes and a large sewer. Above the upper slab of the tunnel, another gallery was placed, housing various pipes and cables (fig. 2).



The work started by placing slurry walls, followed by the excavation required in order to build the slab connected at both ends in the walls. Hollows were provided in the slab, to evacuate the excavated soil and to allow the access for the materials used in the reinforced concrete underground structure. Most of the excavation and concrete works were performed during night time, when access points were open. Traffic on the 900m long boulevard was normally

resumed after five month work.

The joints between the slurry wall panels represented a source of major concern since infiltrations could "steal" sand from below the neighboring buildings and cause damaging settlements. Defective joints were closed using wood wedges. One defective panel had to be doubled by five adjacent Benotto piles.

A very particular case was the presence of a large discontinuity in the siding wall due to a large sewer that could not be broken by the grab-bucket. An aggravating factor was also the presence of predominant sandy layers on site. The gap has been finally filled from the inside by jacking sheet piles as the excavation proceeded, using the upper slab to support the hydraulic jacks.

A comprehensive program in order to measure and record the settlement of every building along the boulevard was established. No settlement exceeding 20mm has been recorded during or after completion of the metro works. One has to mention that most buildings under observation were affected by the two major earthquakes of 1940 and 1977.

The tunnel was put into exploitation in 1987. Water leaks were insignificant and needed no special proofing measures. Wooden sleepers on a ballast blanket insured a fairly good damping for vibrations and noise caused by the underground traffic and no one living in the buildings along the boulevard claimed the subway was a source of any discomfort.

ELEMENT TYPES IN FEM ANALYSIS OF TORCRETE TUNNEL WALLS

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1. CONTINUUM ELEMENTS

1.1 Triangular Elements

Displacements inside the element are obtained by linear interpolation from vertex displacements. Therefore specific deformations and stresses are constant all over the element area. Stiffness matrix of the element and vertex forces due to initial stresses have been determined by integration, based on the extreme value theorem of potential energy (Fig. 1).

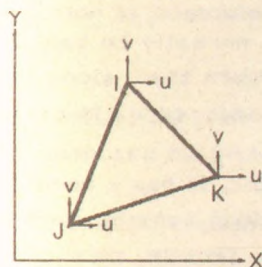


Figure 1

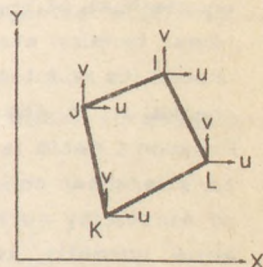


Figure 2

1.2 Tetragonal Elements

Displacements inside the element can be determined by a quadratic interpolation function from vertex displacements. Specific deformations and stresses inside the element vary according to a linear function. Having to do with general tetragons, stiffness matrix of the element and vertex loads from initial stresses have to be determined by numerical integration. Because of higher-order interpolation, tetragonal elements

yield exacter results than do triangular ones, at half the error with reference to the mathematically exact value (Fig. 2).

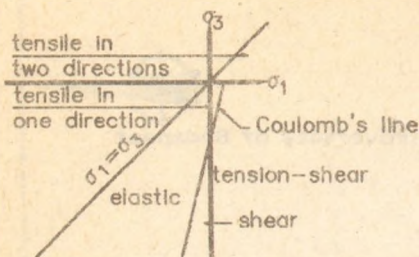


Figure 3

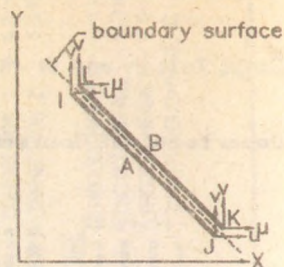


Figure 4

1.3 Possible Element States

Possible states of continuum elements are approximated by elastic models, but elasticity parameters and initial stresses are modified according to results in the previous phase of computations.

Possible states (Fig. 3):

- elastic state: the material is isotropic, its modulus of elasticity and Poisson's ratio are program inputs (test results);
- shear state: the material is isotropic, its modulus of elasticity is program input (test result), value of the Poisson's ratio is computed so that compression (volumetric deformation) modulus of the material equals that of the material in elastic condition;
- linear tension state: the material is orthotropic, normally to tensile cracks its modulus of elasticity is less by five orders than, along the cracks it is the same as the modulus for elastic condition; the Poisson's ratio is zero;
- tensile/shear condition: the material is orthotropic, it has a modulus of elasticity corresponding to shear condition along tensile cracks, while normally to them, corresponding to the tensile condition; Poisson's ratio is zero;
- two-way tensile condition: the material is isotropic, with a modulus of elasticity corresponding to tensile condition; Poisson's ratio is zero.

Provided there is a state change in some elements, element stresses are modified to agree both with elasticity parameters and with element deformations. Thereby elements may undergo several state changes.

2 CONTACT ELEMENTS

2.1 Description

Contact elements help simulating connections between continua of different materials. In fact, the element is bi-dimensional, it has no thickness. It has four vertices at two sides of a surface - pairwise opposite (Fig. 4).

Displacements can be computed by linear interpolation from vertex displacements at each two sides of the surface. Specific deformations of the element result from relative displacements between the two sides, normal to, and/or in, the element plane (tangential). Relative displacements multiplied by normal or tangential stiffness of the element yield normal and/or shear stresses.

These stiffnesses are in kN/m units, values have to be assumed so as not to allow excessive relative displacements between two sides of the surface, at the same time, stiffness of the contact element must not be by orders higher than that of the surrounding contact elements, since it would significantly impair the reliability of results. If no surface friction is possible (e. g. due to dampproofing), tangential stiffness is zero.

Stiffness matrix of the element and nodal loads from initial stresses may be computed by integration.

2.2 Possible Element Conditions

Contact elements may assume three conditions:

- elastic condition: normal and tangential stiffnesses are program inputs;
- shear condition: normal stiffness is the same as the elastic one, tangential stiffness is by five orders lower;
- tensile condition: both normal and tangential stiffnesses are by five orders lower. Also here, stresses are modified as for continuum elements.

2.3 Gap Elements

Gap element are, in fact, a special application of contact elements. Such an element has also a thickness, considered in the program as initial deformation, and until this gap gets closed, the element is in tensile condition, hence, practically, without stiffness. After gap closure, element acts as a perfect contact element.

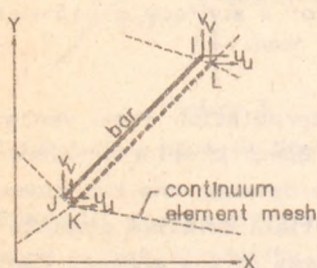


Figure 5

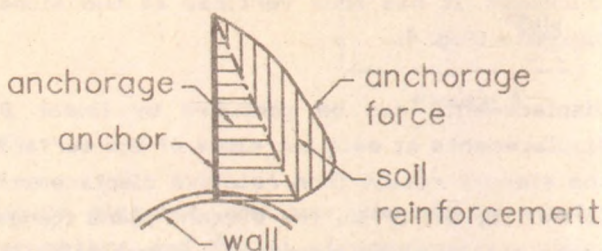


Figure 6

3 ANCHOR ELEMENTS

3.1 Description

An anchor element is essentially a contact element combined with a bar. Thereby the bar can be made to other than rigidly join nodes of the finite element mesh. A rigid connection could not describe the soil peculiarity of not to transmit axial forces to the embedded anchor else than at a relative displacement between soil and anchor bar. Of course, if needed (e. g. for a connection between anchor and tunnel wall) the bar may rigidly join the finite element mesh (Fig. 5).

Connection between the bar and the mesh of continuum elements is modelled by a contact element. Shear stresses in the contact element are due to relative axial displacements between anchor bar and soil. anchor bar displacements have only an indirect effect on normal stresses, namely in the program they are averaged from stresses normal to the bar, developing in continuum elements joining the contact element, rather than from contact element deformations.

The anchor element does not disjoin continuum elements, it does not cut the finite element mesh, but about emerges from its plane. Thereby the anchor not only serves for fastening, suspending the tunnel wall to the

ANCHORAGE FORCES

$M=1:50$

100 kN

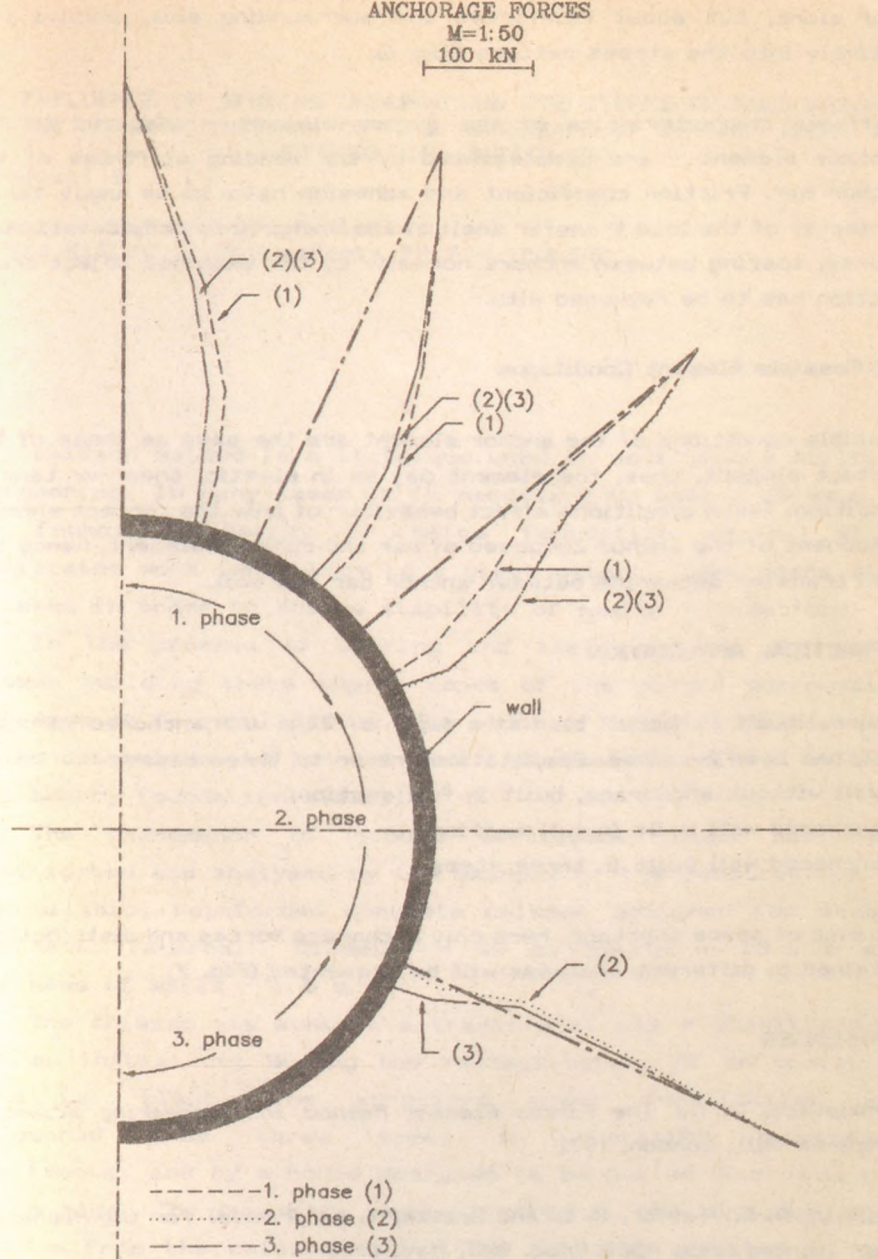


Figure 7

soil alone, but about reinforces the surrounding soil, involving it actively into the stress pattern (Fig. 6).

Stiffness characteristics of the anchor element - compared to the contact element - are complemented by the bending stiffness of the anchor bar. Friction coefficient and adhesion have to be input taking perimeter of the load transfer shell of the anchor into consideration. Of course, spacing between anchors normally to the examined object cross section has to be reckoned with.

3.2 Possible Element Conditions

Possible conditions of the anchor element are the same as those of the contact element, thus, the element may be in elastic, shear or tensile condition. These conditions affect behaviour of only the contact element component of the anchor composed of bar and contact element, hence the load transfer mechanism between anchor bar and soil.

4 PRACTICAL APPLICATION

A tunnel built in Kiscell clay at a depth of 22 m with anchored torcrete wall, has been examined. Computations refer to three cases:

- wall without anchorage, built in full section;
- anchored wall built in full section;
- anchored wall built in three steps.

Because of space shortage, here only anchorage forces and distributions obtained in different analyses will be presented (Fig. 7)

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THE INFLUENCE OF SINKING UNDERGROUND STRUCTURES OR FOUNDATIONS
BY THE CAISSON METHOD ON THE SURROUNDING GROUND MASS AND
NEIGHBOURING BUILDINGS

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Caisson method is a technique used in underground building engineering. In many cases it is necessary to make a caisson in the foundation nearby or below foundation depth, which complicates work technology to a great extent. Some means must be taken in order to ensure stability of nearby foundations.

In the process of sinking and inside ground mining at caisson building there appear zones of the object surrounding soil mass self-consolidation. The width of the soil slide being formed determines the zone of work influence at caisson sinking upon nearby foundation stability.

The phenomenon of ground settlement zones has been investigated and analysed by the example of the construction of a monolithic, reinforced concrete caisson designed for sewage treatment. Internal diameter - 40 m, height - 13.1 m and thickness of walls - 1.8 m.

The caisson was sunk in a traditional way - gravitationally from an initial cut during the reconstruction of an operating industrial plant. The structure under construction was surrounded from three sides by operating production departments, and by a house designed to be pulled down from the fourth side. The buildings were situated at the distance of 8 to 20 m from the caisson walls. The building situated at the distance of 8.0 m was threatened by ground settlement. Therefore, in order to counteract this phenomenon a Larsen - IV type sheet pile wall was made in the ground around the caisson at the distance of 6.0 m from its walls.

Dusty damped, sandy clay es, and clayey damped sands were

found on the construction site. A systematic observation of the ground settlement around the caisson was carried on by means of a levelling instrument.

According to the adopted method of work the observation included:

- observation of cracks being formed in the ground,
- observation and measurements of ground deformation zones around the caissons,
- geodetic measurements of the caisson wall tilts during caisson sinking,
- geodetic measurements of the settlement of the surrounding buildings.

To this end, ground pins of the structure NIOSP were placed in the soil to carry out the measurement of strata settlement of soil mass.

Wall signs were placed on all the surrounding buildings and measurements were made with a levelling instrument both in the process of sinking of the sheet pile wall and of caisson sinking.

The measurements were made in the following way:

- while sinking up to 5 m, every 500 mm of caisson sinking,
- while sinking up to 5-10 m, every 200 mm of caisson sinking,
- while caisson tilting, every 100 mm.

Analysing the results of geometrical levelling, it was found that the buildings situated near the caisson did not subside markedly.

The application of the sheet pile wall did not give expected results and it subsided itself. The greatest deformations and landslip were observed in the zone between the caisson walls and the sheet pile wall. Fig.1 represents ground settlement on different depths.

As a result of the investigation three zones of ground deformation were distinguished at different distances from the caisson walls:

- 1 - landslide zone - complete destruction of the natural state of ground near the structure's wall,
- 2 - microcrack zone,

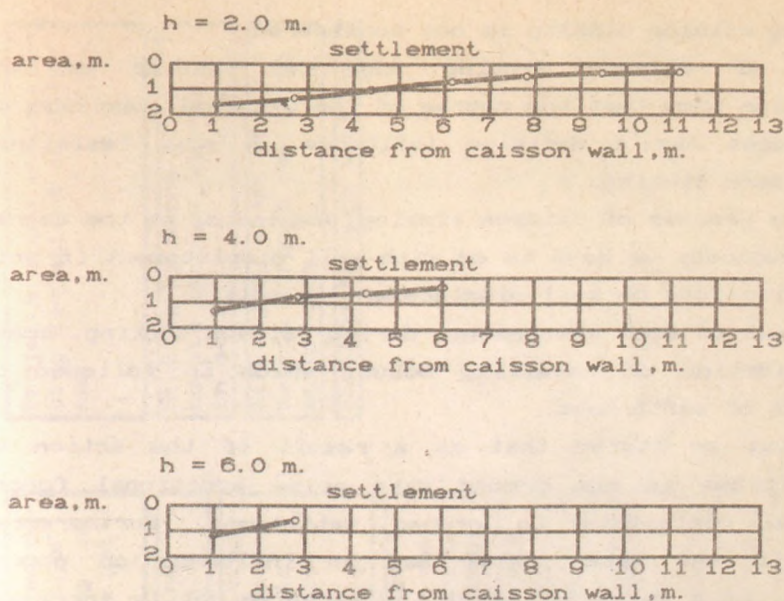


Fig.1. Ground settlement at different depths.

3 - circumferential zone whose settlement amounts to a few millimetres.

Analysing the results of the investigation of the phenomenon of ground displacements and subsiding of the adjacent buildings during caisson sinking we come to a conclusion that irrespective of a high range of underground work executed in this technique the problem in question is hardly solved.

The main causes which have an effect on the settlement of adjacent ground during caisson sinking include:

- caisson tilts,
- protrusions on the external lateral surface and its roughness,
- friction forces which act along the external surface of the caisson wall,
- outflow of thixotropic suspended matter,
- lowering of soil water level.

Analysing the standards for caisson designing we conclude that they take into account only the soil settlement zones around the caisson within which the existing structures should not exist. However, the effect of the causes of soil deforma-

tion during caisson sinking is not considered.

Data of caisson sinking and the results of the investigation show that the degree of the external roughness of wall surfaces has a decisive influence on soil resistance during caisson sinking.

In the process of caisson sinking, depending on the degree of wall roughness we have to do with wall displacement in soil (surface friction) or soil displacement in soil.

As a result of soil settlement, during caisson sinking, zones of consolidation or ravelling occur, which is followed by deformation of earth mass.

It must be stated that as a result of the action of friction forces in the ground mass arise additional forces which also contribute to ground settlement. Furthermore, lowering of the water level had an influence on ground settlement, as a result of which the earth weight is increased within the limits of the sinking zone.

On the basis of an analysis of different factors which affect the deformation of the earth mass which surrounds the caisson in the process of sinking, it is possible to classify engineering projects aimed at the improvement of the method of caisson sinking during construction in the conditions of direct neighbourhood of foundations according to the following criteria (Fig.2.):

- projects aimed at decreasing the soil friction forces along the lateral surface of the caissons,
- projects aimed at decreasing deviations from the caisson perpendicular,
- projects aimed at decreasing deformation of soil mass depending on the method of earth getting.

Conclusions

1. It is recommended that in designing underground structures in conditions of direct neighbourhood of the existing foundations caisson sinking techniques should take into account ground conditions, distances to existing structures, and permissible magnitudes of the settlement of buildings which are inside and outside the ground deformation

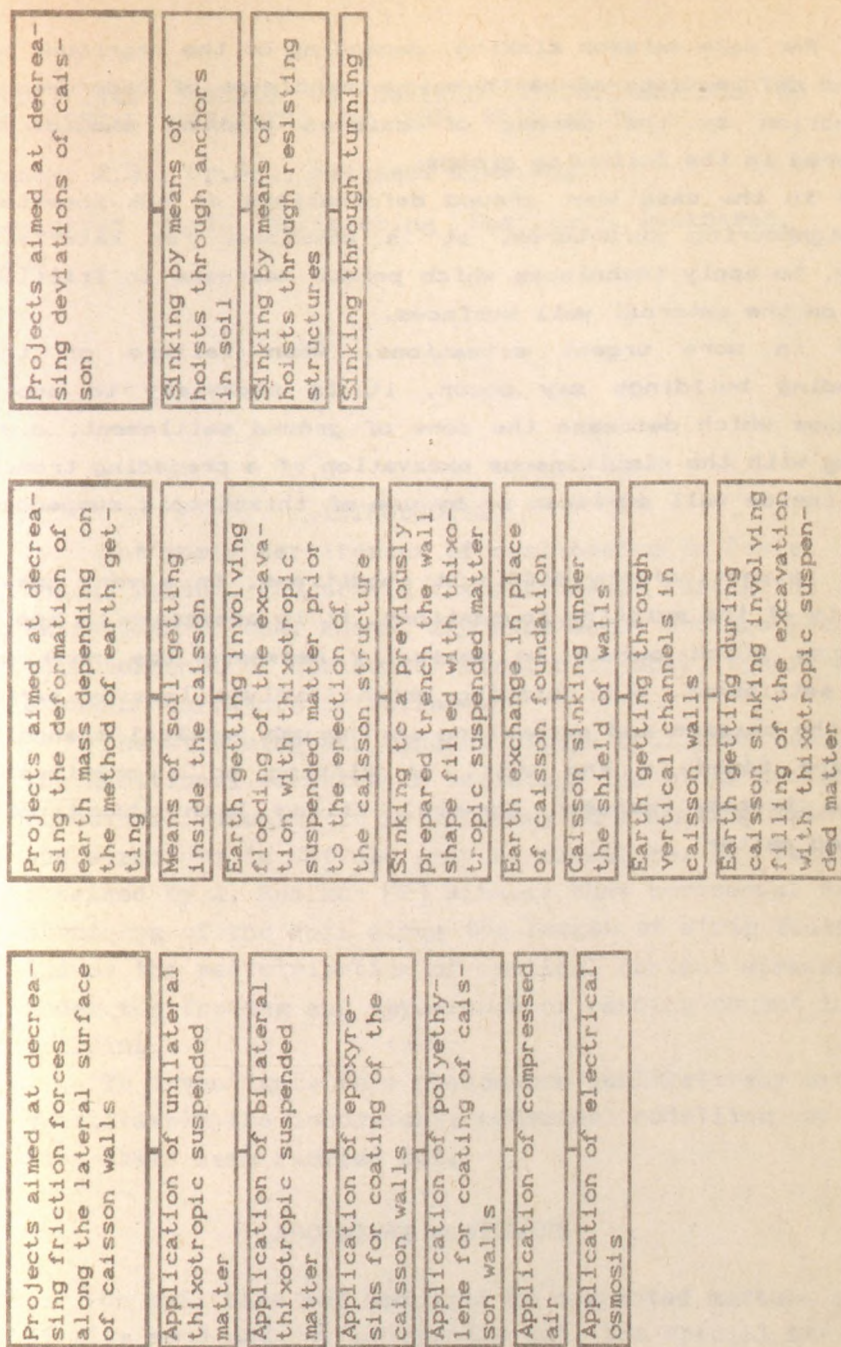


Fig. 2. Engineering projects aimed at the improvement of the way of caisson sinking in direct neighbourhood of existing foundations.

zone.

2. For safe caisson sinking, depending on the magnitude of expected deformations of earth mass, techniques of underground construction by the method of caisson sinking should be considered in the following groups:

a) in the case when ground deformations do not threaten the neighbouring structures, it is possible, for materials economy, to apply techniques which permit decrease in friction forces on the external wall surfaces,

b) in more urgent situations, when failure of the surrounding buildings may occur, it is necessary to apply techniques which decrease the zone of ground settlement, e.g. lowering with the simultaneous excavation of a preceding trench at the inside wall surfaces or by use of thixotropic suspended matter,

c) in particularly difficult conditions, in a very great proximity of the existing foundations, it is necessary to apply techniques which permit to maximally decrease the zone of ground settlement, e.g. sinking which involves leaving earth inside the caisson and excavating it thorough vertical channels which are found in the wall, or sinking to a previously prepared trench (caisson forwork) filled with thixotropic suspended matter.

ADDITIONAL BENDING OF SURFACE STRUCTURES DUE TO HORIZONTAL STRAINING OF THE SOIL BASE.

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INTRODUCTION

The waste territories of coal basins in Soviet Union, Poland, FRG obtain the subsidence during the extraction of coal seams. The subsidence process leads to appearance of horizontal strains in soil, upward and downward curvature of surface.

The straining and bending of the soil base causes the additional stresses in buildings and other surface structures. Usually it is believed that bending moment in the structure is defined only by curvature. But results obtained by A. Rosikon [2] witness that horizontal tensile straining of the soil along the length of strip footing induce the redistribution of vertical contact stresses under the footing and appearance of bending moment in the building.

To investigate this phenomenon qualitatively and quantitatively the laboratory, numerical modelling and in-situ tests were carried out.

LABORATORY MODELLING

The soil base was modelled by compacted mixture of 97 % of fine sand and 3% of spindle oil. The special stand promoted the opportunity to strain the model of the soil base along the axis of surface structure.

Three series of laboratory experiments were carried out with various types of surface structures (Fig.1). The 1-st type was presented by flexible strip. The 2-nd type was presented by a beam with finite stiffness instrumented by strain gauges. The 3-rd type was presented by gypsum-concrete model walls.

The abovementioned model structures were uniformly loaded in accordance to criteria of similarity.

When the soil base under the flexible structure of the 1-st type was extended the latter obtained the upward curvature, that is it's ends obtained greater settlements than middle part.

As it is known [4] the horizontal tensile straining decreases the stiffness and bearing capacity of soil. But due to reinforcing effect of the strip footing the strains of the soil under it in it's middle part decreased in comparison to end parts, so the decrease of stiffness of soil base was greater under the middle of structure. This consideration entirely explains the phenomenon of induced bending of structure in absence of curvature of surface.

The second series of experiments the initially uniform distribution of contact stresses under the footing changed becoming more dome-like with increasing the stiffness of beam (Fig. 2b).

The curvature of beam was the lower the higher was it's stiffness.

In accordance to back calculations in all models of 1-st and 2-nd series the final stiffness modulus of soil after horizontal straining decreased from middle of footings to it's ends.

The diagrams on Fig.3 summarizing the results of 3-rd series of experiments, relate the width of cracks appeared in model of wall to the horizontal strain of soil. The width of cracks is given in the scale of natural 1,3 and 5-floors brickwork houses.

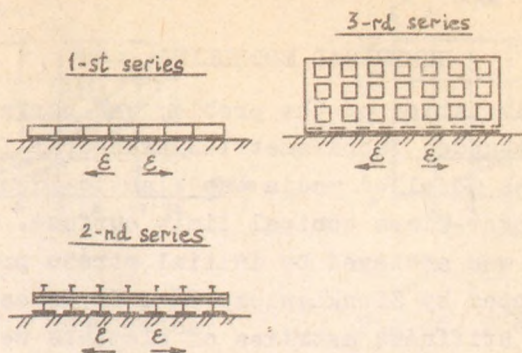


Fig. 1. The series of the laboratory tests.

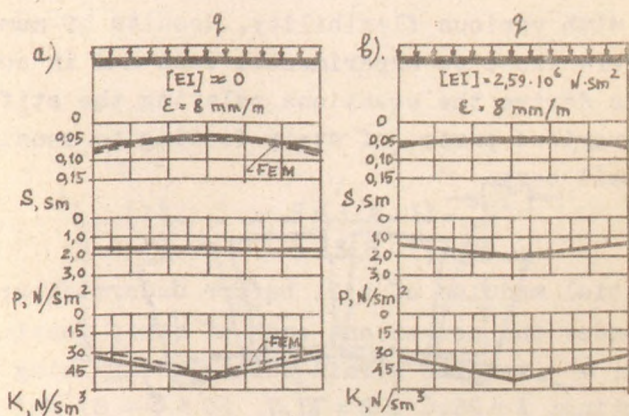


Fig. 2. The settlements (S), the contact pressure (P) and the stiffness coefficients along the laboratory beam.

The higher necessary to originate it's cracking, but since the crack has appeared it's width is proportional to the height of building.

NUMERICAL MODELLING.

Numerical simulation of the problem was performed by three-axial eight-nodes isoparametric hexahedral finite elements [1]. The modelled media was elastic-ideally plastic with Drucker-Mises conical limit surface. The elasto-plastic solution was achieved by initial stress procedure initially developed by Zienkiewicz [3]. It is essential to note that the stiffness matrixes of elements were compiled by eight Gaussian points of integration while the initial stresses and corresponding to them initial forces were calculated only by one central Gaussian point. Such an approach promoted the inverseability of stiffness matrix and stability of computation process. The grid of finite elements was presented by a cubical block loaded from above by a strip with various flexibility. Results of numerical modelling were close to experimental ones and in common they permitted to derive the equations relating the stiffness of soil under various parts of strip footing to tensile strain of soil base:

$$E_u^c = (1 - A \cdot \varepsilon) \cdot E \quad (1)$$

$$E_u^e = (1 - B \cdot \varepsilon) \cdot E \quad (2)$$

where E - initial modulus of soil before undermining; E_u^c , E_u^e - modulus under the center and ends of strip footing after undermining; ε - tensile strain during undermining along axis of footing; $A = 26,6$, $B = 77,7$ ($0 < \varepsilon \leq 8$) and $A = 22,3$, $B = 65,3$ ($8 < \varepsilon \leq 12$) - coefficients.

IN-SITU EXPERIMENTS

To check the laboratory and numerical results the full-scale experiments were carried out in Karaganda coal basin. The beams nine metres long were laid on the soil surface. The beams had stiffness similar to stiffness of real houses

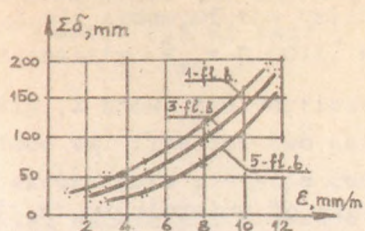


Fig. 3. The dependence of the summary values of the opening cracks of building $\Sigma\delta$ on the horizontal tensile deformations of the base ε

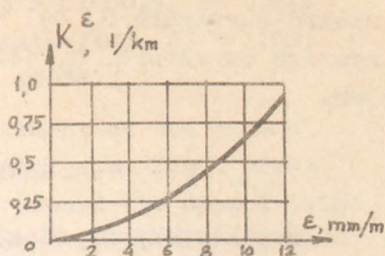


Fig. 4. The induced curvature of the flexible beam (K) versus horizontal tensile strain of the soil base (ε).

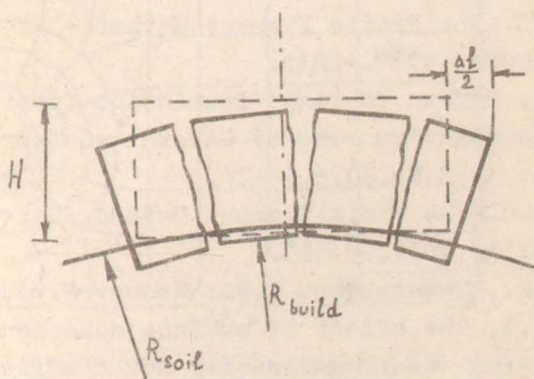


Fig. 5. To the equation (3).

and were loaded from above. During the undermining the soil surface obtained the horizontal strain $\varepsilon = 0,003$ (3 mm/m) and upward curvature $0,097 \text{ km}^{-1}$, while the beams obtained the average curvature $0,148 \text{ km}^{-1}$, that is 1,5 times greater.

ANALYSIS.

The most important final measure of undermining is the summary width of cracks in building caused by elongation of building due to tensile straining and bending of base. The results of performed investigations permit to include into the equation:

$$\Delta l = l \sqrt{m_{\varepsilon}^2 \varepsilon^2 + m_K^2 H_b^2 (K + K^{\varepsilon})} \quad (3)$$

where K - curvature of soil surface;
 ε - horizontal tensile strain;
 H_b - height of building;
 K^{ε} - additional curvature of building induced by straining (see Fig.4 and Fig. 5.)

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index, $C_c = 0.147$; hyperbolic parameters, $a = 0.006$, $b = 2.73$, $R_F = 0.9$; secondary compression index, $\psi = 0.0012$; Singh-Mitchell creep parameters, $A = 0.00014$, $m = 0.64$; $\bar{\alpha} = 2.47$; slope of the critical state line, $M = 1.05$; permeability, $k_{xx} = 0.0011$, $k_{yy} = 0.00054$.

Fig. 2 shows the vertical settlement at the ground surface vs. time for two sets of analysis. Fig.3 shows the excess pore pressure contours around the opening. The initial excess pore pressure predicted by both analyses are almost the same, although the creep-inclusive analysis tends to predict higher pore pressures. However, the rate of dissipation of excess pore pressure is significantly slower in creep-inclusive analysis than in no-creep analysis. Fig.3/c/ shows the excess pore pressure contours at the 301 day mark predicted by the creep-inclusive analysis. The excess pore pressures predicted by the no-creep analysis are virtually completely dissipated.

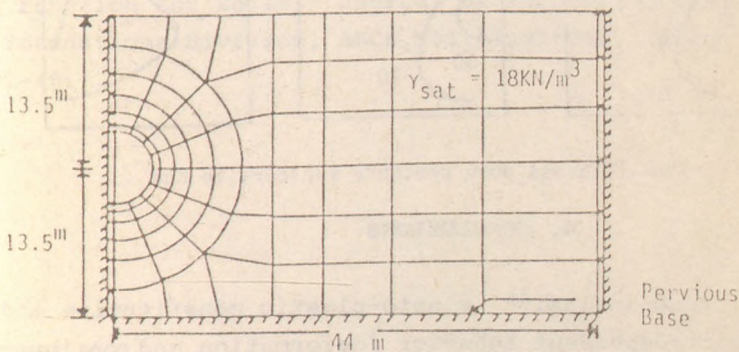


Fig. 1 Finite element mesh for numerical analysis

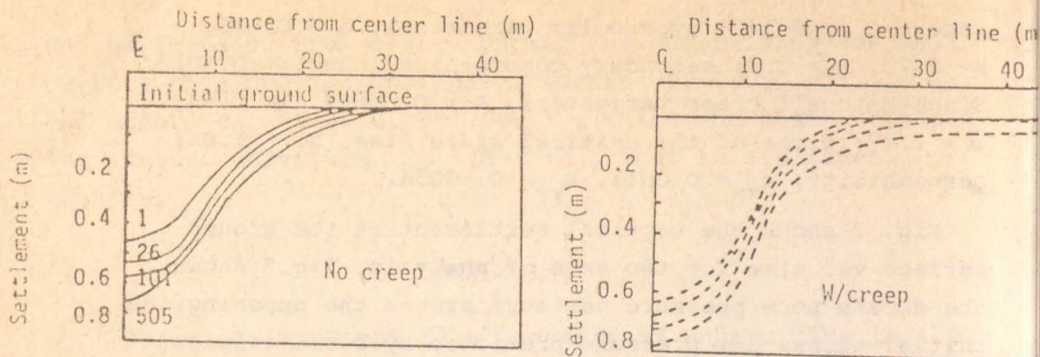


Fig. 2 Vertical settlement at the ground surface

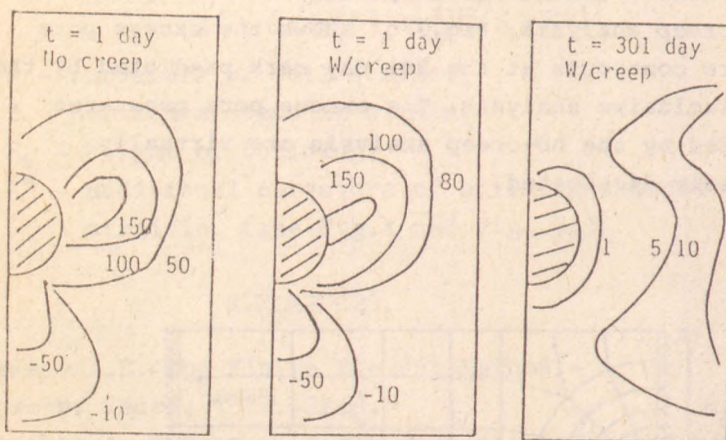


Fig. 3 Excess pore pressure contours in Kpa

4. CONCLUSIONS

A creep-inclusive elasto-plastic constitutive model for time-dependent behavior /deformation and pore pressure response/ of fully saturated, soft clays has been developed and coded into a "mixed formulation" finite element program SPIN2D. The program was used to analyze an illustrative example which involves tunneling in the

squeezing ground. According to the numerical analysis results, the creep induced deformation constitutes a major portion of the total deformation for highly plastic clays and should be considered during design and construction of tunnels in the squeezing ground.

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STRESS-STRAIN ANALYSIS OF KARSTIC SUBSOIL-SLAB (STRIP) FOOTING SYSTEM

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Hydro-geological events have increasingly been observed in densely populated industrial areas due to antropogeneous impact.

Ground water table rizing in karst-prone geological provinces brings about softening of the soil and intensive karst formation. Karst risk for buildings and structures consists in appearance of underground cavities and surface slumps that dramatically change "footing-soil" system behaviour. Therefore, the design of buildings in karst hazardous areas should make allowance for possible subsidence below the footing.

Development of appropriate methods of analysis is necessary to assess, in congested urban areas, the strength and safety of existing structures if karstic events in the neighbourhood may be expected.

The paper presents a solution of the slab (strip) footing-karstic base interaction problem.

In our view, the efficient solution of interaction problems may be obtained in the separate analysis of the base and the footing with their subsequent dovetailing. This concept outlines the modular architecture of the "GROS-3" software package for solving this class of problems.

The soilbase is simulated by an elastic multi-layered half-space, confined under an arbitrary piecewise-smooth surface. In order to determine its stress-strain state, boundary elements technique was applied. Analysis of such a

half-space presents a separate problem viewed both as the development of the theory and software and as extension of its practical application.

The principle of the method consists in submerging the zone in an infinite space that is identical to multiplying Heaviside function by displacements and their derivatives of the second order inclusive.

After solving the defined mixed elastic problem for the half-space limited by piecewise-smooth surface $\eta(x_1, x_2)$ $[-\infty < x_1, x_2 < \infty; -\infty < x_3 < \eta(x_1, x_2)]$ in displacements we obtain the transformed equations of equilibrium in the form proposed by Lamé as follows:

$$\Delta \tilde{u}_i + \nu \frac{\partial \tilde{\theta}}{\partial x_i} = A_i(S) \delta(x_3 - \eta) \frac{d^2 S}{dx_1 dx_2} \quad (1)$$

$i=1, 2, 3$

with

$$\nu = \frac{1}{(1-2\mu)}; \quad \theta = \sum_{i=1}^3 \frac{\partial u_i}{\partial x_i}; \quad \Delta = \sum_{i=1}^3 \frac{\partial^2}{\partial x_i^2};$$

μ - Poisson ratio;

S - boundary surface parameter;

u_i - elastic body movement along x_1, x_2, x_3 axes;

A_i - unknown densities that are identical to forces;

$\eta(x_1, x_2)$ - equation describing the surface of karstic subsidence;

$$H(t) = \begin{cases} 1, & t \leq 0 \\ 0, & t > 0 \end{cases}; \quad \delta(t) = \frac{dH(t)}{dt}$$

The boundary conditions are as follows:

$$p_i = \sigma_{ij} \bar{n}_j \quad (2)$$

$i, j = 1, 2, 3$

Here P_i are projections of the external force applied to a segment with the normal on the surface of the body;

σ_{ij} are stresses; $\bar{n}_i$ - cosines of the angles between the normal and coordinate axes x_1, x_2, x_3 , i.e.

$$\bar{n}_i = \cos(\hat{n} x_i)$$

or

$$n_1 = \frac{-\bar{p}}{\pm\sqrt{1+\bar{p}^2+\bar{q}^2}}; n_2 = \frac{-\bar{q}}{\pm\sqrt{1+\bar{p}^2+\bar{q}^2}}; n_3 = \frac{1}{\pm\sqrt{1+\bar{p}^2+\bar{q}^2}};$$

$$\bar{p} = \partial x_3 / \partial x_1, \quad \bar{q} = \partial x_3 / \partial x_2.$$

By applying 3D Fourier transform to the system (1) and to the stress vector we obtain a system of integral equations with unknown densities $A_i(S)$, /1/.

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$$\tilde{u}_k = \frac{q}{2} \iint_S [A_1(S) M_k + A_2(S) N_k + A_3(S) F_k] \frac{dS}{\Phi^3}$$

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$$i, j, k = 1, 2, 3;$$

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where t_i ($i=1, \dots, 9$) are directing cosines of coordinate axes of the local coordinate system obtained in translational and rotary movement of the global system;

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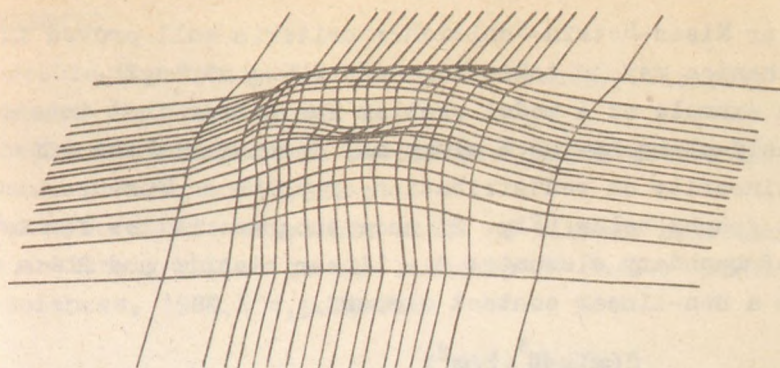


Fig.1 Grafical image of stress distributions in karstic soilbase

In this example the case is considered that corresponds to formation of cavities under the center and the corner which is the most unfavorable stress-strain state in the soilbase.

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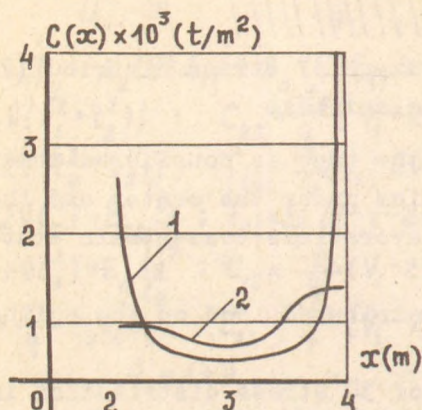


Fig. 2 Diagrams of soilbase rigidity coefficients
1- Linearly-elastic soilbase model;
2- Model with a non-linear contact element

A 3m span beam has a rectangular cross-section of 1,3 m², reinforced concrete elasticity modulus $E_b = 2650$ MPa, the beam half-width equal to 0,5 m; soilbase deformation modulus $E_0 = 20$ MPa; Poisson ration $\nu = 0,33$; depth of the slump depression 3 m; radius of the slump depression 2 m.

The equally distributed load is equal to 0,1 MPa. The contact elements complies with the limitation for normal contact stress $\sigma_z \leq 0,3$ MPa.

In conclusion we note that the "GROS-3" program is capable of selecting soilbase deformation characteristics that ensure specified settlements of the footing.

This solution can easily be applied to problems of rescuing buildings with the help of chemical or other techniques for stabilizing soil weakened by karstic cavities.

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NEW METHOD FOR CALCULATION OF NATM METHOD TUNNEL LININGS

Dr Müller, Miklós and Czap, Zoltán

Department of Geotechnique, Technical University of Budapest

ABSTRACT

The use of elastic soil grid method to determine stresses in shotcrete lining interacting with the soil environment is described. An example for the application is presented.

In problems of designing real underground structures, the engineer is expected to model the structure, its material and the load. This is how the mechanical model for solving the problem arises. Stresses and displacements obtained from the mechanical model suit to determine strength, deformation and safety of stability. Results from field measurements on the structure accomplished (normal stresses, loads, curvature variations, deformations, displacements and rock field loosening) may be confronted to those obtained on the model, permitting a model correction that will improve the truth of the model to be next applied.

Even for the design of the simplest underground structure - design of a circular tunnel without hinges, embedded in a homogeneous elastic, isotropic and weightless rock surrounding - the problem above can only be written in terms of a partial differential equation system with eight unknowns, to be solved taking the given boundary conditions into consideration.

Thus, already in establishing the mechanical model, the engineer is forced to neglect for the sake of simplicity that significantly impair the truth of results. Now, the mathematician follows, introducing neglects into the differential equation system of the problem for the sake of the

feasibility of a mathematical solution, often without being able to realize either by himself or with the engineer the further biasing of the reality modelled.

This is why in the design practice of underground structures, numerical mathematical models are applied for the "exact" solution of problems, and realistic approximate methods using simplified mechanical models for the design at sketch plan level.

Advantages of the application of numerical methods are:

- they enable the engineer to establish a more realistic structural model, permitting him to track development of the final structure in building phases;
- there is an increased possibility for a realistic modelling of materials;
- only the primary stress state has to be assumed as an ideal load, namely, provided material, structure, building order are correctly modelled, loads are obtained by computations;
- the engineer is able to track technical consequences of variations of the mechanical model, rather than to lose sight of them in the maze of analytic solution often untraceable to an engineer.

Computations relying on numerical methods to be effective require proper development of laboratory tests and field measurements.

Realistic approximations by simple mechanical models usually rely on results of "exacter" solutions, but the involved neglects permit simpler mathematical treatment.

Among numerical methods, FEM suits both great and small depth urban tunnels, provided there is proper modelling. A realistic output may be expected if also structure, material and load are truly modelled.

In modelling the structure, correct modelling of elastic or real hinges, possible voids and connections between wall lining and soil, as well as interaction between anchors and soil are of importance. To simulate connection between soil and wall, gap units have been developed, permitting simultaneous modelling of the empty back void, as well as of the connection meeting the Mohr-Coulomb condition of friction. Connection between anchor and soil has been modelled by elements that do not cut soil continuity as in reality but yield the possible anchorage force as

a function of soil stresses, anchor to soil friction and adhesion, reacting thereby also on soil stresses.

In modelling the soil, it has been attempted to correctly model structural behaviour of the experimental characteristic curve of the soil at failure, and the subsequent volumetric deformation possibility.

Loading has been simultaned by carefully taking all elements of the building sequence significantly affecting wall and soil stresses into consideration.

This modelling is an intricate, rather laborious process. Therefore an approximate method has been developed, to be presented below.

Among approximate methods for mountain tunnels build at great depths, having normally a single circular cross section or it is composed of circular arcs, approximate calculation for ultimate shear is applied, assuming horizontal and vertical stresses to be equal. Accordingly, dimensions, strength of the shotcrete lining, number and load capacity of anchors have to be chosen so that the soil core plasticized near the horizontal diameter does not cause shear failure in the built structure and the interacting rock zone.

This method is generally unsuitable for urban underground railway tunnels, namely:

- because of the small depth, the value of earth pressure at rest is lower by orders than that for maintain tunnels;
- the approximation of vertical and horizontal stresses to be equal is inadmissible;
- the load being lower, generally no plastic core can develop;
- structures delicate for building technology and from structural aspects, e. g. stations, are of composite cross sections, and even for outermost lateral tunnels, it is rather hardy to assume a circular cross section.

An approximate method will be presented below, involving rather realistic assumptions, and simple to be applied for the approximate calculation of underground engineering structures.

Approximate assumptions, mechanical model and load:

- Loads from earth pressure and carried by an interacting structure of soil environment and reinforced shotcrete.
- Thickness of the interacting rock zone is 30 % of the free opening, for a circular cross section, $0.3 \cdot D = 0.6 \cdot R$, a statement relying on exact computation methods and field measurement results.
- In the elastic range, at an axial point of the beam, cross sections remain plane after deformation.
- At the soil to r. c. interface, no slip between soil and concrete arises until there shear stress is lower than the ultimate shear strength of the soil. This condition generally does not develop.
- the thereby developed interacting structure can be modelled by a bar system of ideal cross section of a bending and compression stiffness equal to that of the composite structure (Fig. 1).

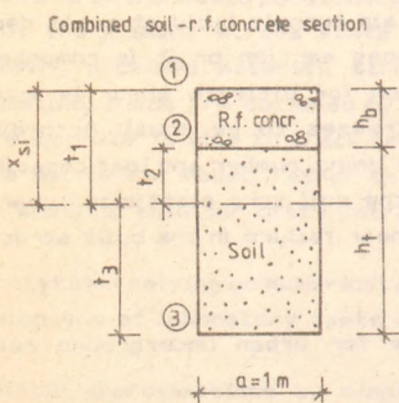


Fig. 1

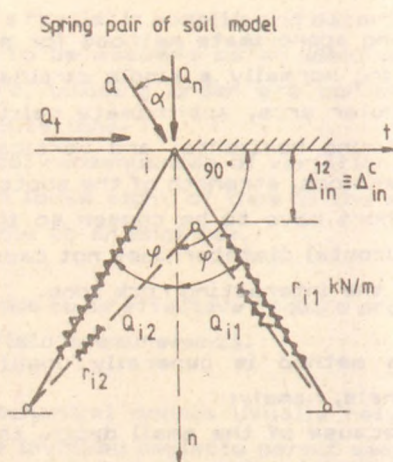


Fig. 2

$$x_{si} = \frac{\frac{h_b^2}{2} + n \cdot \left(\frac{h_t^2}{2} + h_t \cdot h_b \right)}{h_b + n \cdot h_t}$$

$$S_{Ni} = a \cdot h_b \cdot E_b + a \cdot h_t \cdot E_t$$

$$I_i = \frac{a \cdot h_b^3}{12} + a \cdot h_b \cdot \left(x_{si} - \frac{h_b}{2} \right)^2 + n \cdot \left[\frac{a \cdot h_t^3}{12} + a \cdot h_t \cdot \left(\frac{h_t}{2} + h_b - x_{si} \right)^2 \right]$$

Taking the modulus of elasticity of the ideal beam E_i as that of concrete E_b , stiffness data of the ideal cross section will be computed as (Fig. 2).

index, $C_c = 0.147$; hyperbolic parameters, $a = 0.006$, $b = 2.73$, $R_F = 0.9$; secondary compression index, $\psi = 0.0012$; Singh-Mitchell creep parameters, $A = 0.00014$, $m = 0.64$; $\bar{\alpha} = 2.47$; slope of the critical state line, $M = 1.05$; permeability, $k_{xx} = 0.0011$, $k_{yy} = 0.00054$.

Fig. 2 shows the vertical settlement at the ground surface vs. time for two sets of analysis. Fig.3 shows the excess pore pressure contours around the opening. The initial excess pore pressure predicted by both analyses are almost the same, although the creep-inclusive analysis tends to predict higher pore pressures. However, the rate of dissipation of excess pore pressure is significantly slower in creep-inclusive analysis than in no-creep analysis. Fig.3/c/ shows the excess pore pressure contours at the 301 day mark predicted by the creep-inclusive analysis. The excess pore pressures predicted by the no-creep analysis are virtually completely dissipated.

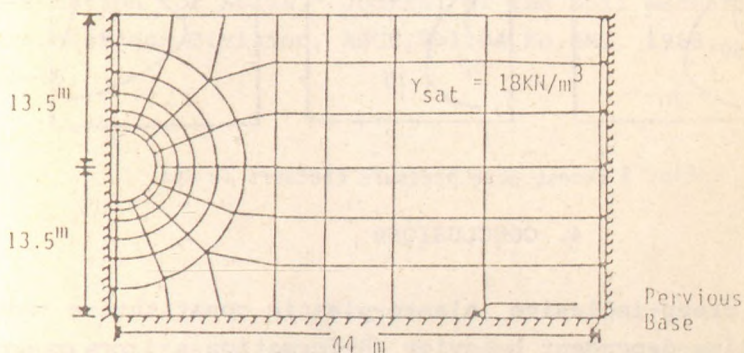


Fig. 1 Finite element mesh for numerical analysis

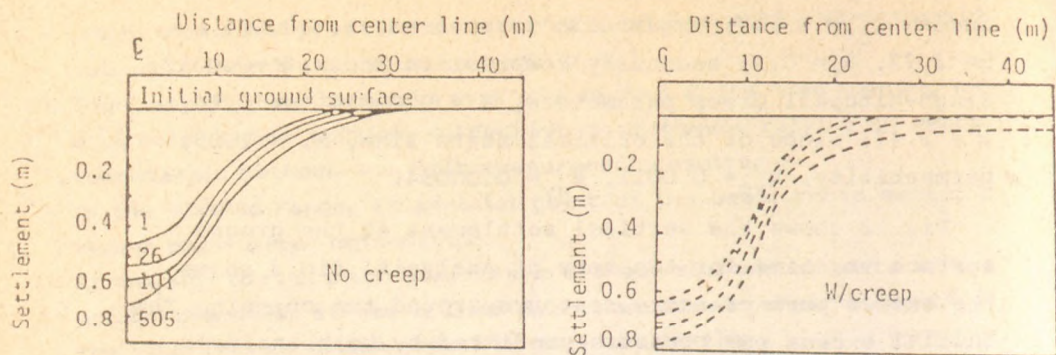


Fig. 2 Vertical settlement at the ground surface

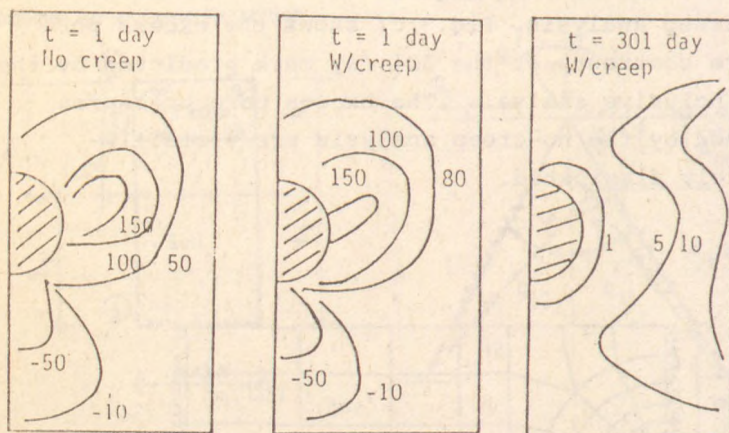


Fig. 3 Excess pore pressure contours in Kpa

4. CONCLUSIONS

A creep-inclusive elasto-plastic constitutive model for time-dependent behavior /deformation and pore pressure response/ of fully saturated, soft clays has been developed and coded into a "mixed formulation" finite element program SPIN2D. The program was used to analyze an illustrative example which involves tunneling in the

squeezing ground. According to the numerical analysis results, the creep induced deformation constitutes a major portion of the total deformation for highly plastic clays and should be considered during design and construction of tunnels in the squeezing ground.

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STRESS-STRAIN ANALYSIS OF KARSTIC SUBSOIL-SLAB (STRIP) FOOTING SYSTEM

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Structures, Moscow, USSR

Hydro-geological events have increasingly been observed in densely populated industrial areas due to antropogeneous impact.

Ground water table rizing in karst-prone geological provinces brings about softening of the soil and intensive karst formation. Karst risk for buildings and structures consists in appearance of underground cavities and surface slumps that dramatically change "footing-soil" system behaviour. Therefore, the design of buildings in karst hazardous areas should make allowance for possible subsidence below the footing.

Development of appropriate methods of analysis is necessary to assess, in congested urban areas, the strength and safety of existing structures if karstic events in the neighbourhood may be expected.

The paper presents a solution of the slab (strip) footing-karstic base interaction problem.

In our view, the efficient solution of interaction problems may be obtained in the separate analysis of the base and the footing with their subsequent dovetailing. This concept outlines the modular architecture of the "GROS-3" software package for solving this class of problems.

The soilbase is simulated by an elastic multi-layered half-space, confined under an arbitrary piecewise-smooth surface. In order to determine its stress-strain state, boundary elements technique was applied. Analysis of such a

half-space presents a separate problem viewed both as the development of the theory and software and as extension of its practical application.

The principle of the method consists in submerging the zone in an infinite space that is identical to multiplying Heaviside function by displacements and their derivatives of the second order inclusive.

After solving the defined mixed elastic problem for the half-space limited by piecewise-smooth surface $\eta(x_1, x_2)$ $[-\infty < x_1, x_2 < \infty; -\infty < x_3 < \eta(x_1, x_2)]$ in displacements we obtain the transformed equations of equilibrium in the form proposed by Lamé as follows:

$$\Delta \tilde{u}_i + \nu \frac{\partial \tilde{\theta}}{\partial x_i} = A_i(S) \delta(x_3 - \eta) \frac{d^2 S}{dx_1 dx_2} \quad (1)$$

$i=1, 2, 3$

with

$$\nu = \frac{1}{(1-2\mu)}; \quad \theta = \sum_{i=1}^3 \frac{\partial u_i}{\partial x_i}; \quad \Delta = \sum_{i=1}^3 \frac{\partial^2}{\partial x_i^2};$$

μ - Poisson ratio;

S - boundary surface parameter;

u_i - elastic body movement along x_1, x_2, x_3 axes;

A_i - unknown densities that are identical to forces;

$\eta(x_1, x_2)$ - equation describing the surface of karstic subsidence;

$$H(t) = \begin{cases} 1, & t \leq 0 \\ 0, & t > 0 \end{cases}; \quad \delta(t) = \frac{dH(t)}{dt}$$

The boundary conditions are as follows:

$$P_i = \sigma_{ij} \bar{n}_j \quad (2)$$

$i, j = 1, 2, 3$

Here P_i are projections of the external force applied to a segment with the normal on the surface of the body;

σ_{ij} are stresses; $\bar{n}_i$ - cosines of the angles between the normal and coordinate axes x_1, x_2, x_3 , i.e.

$$\bar{n}_i = \cos(\hat{n} x_i)$$

or

$$n_1 = \frac{-\bar{p}}{\pm\sqrt{1+\bar{p}^2+\bar{q}^2}}; n_2 = \frac{-\bar{q}}{\pm\sqrt{1+\bar{p}^2+\bar{q}^2}}; n_3 = \frac{1}{\pm\sqrt{1+\bar{p}^2+\bar{q}^2}};$$

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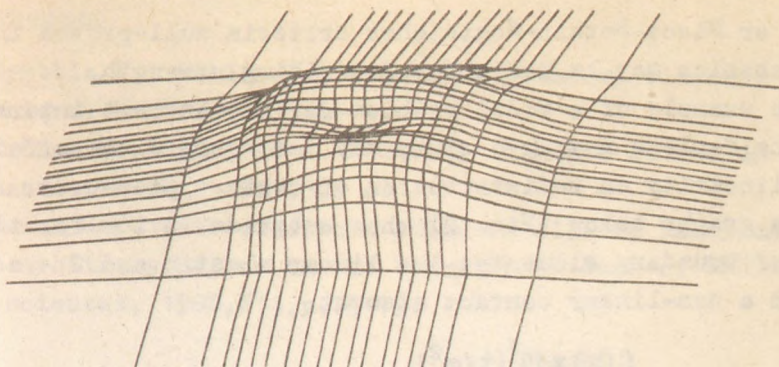


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The example of a model problem for a beam that interacts with a half-plane having a slump may demonstrate the effect of non-linearity on redistribution of contact pressures.

The graphs below (Fig. 2) show subgrade ratios for two models of boundary elements; 1 - linear elastic and 2 - a model with a non-linear contact element.

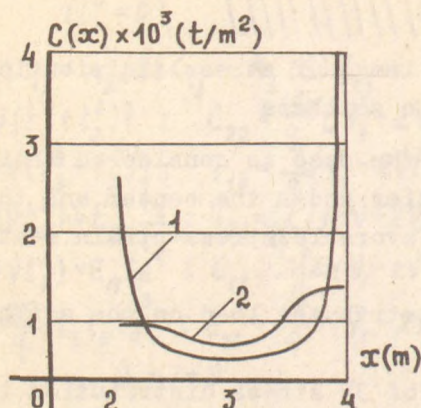


Fig. 2 Diagrams of soilbase rigity coefficients
1- Linearly-elastic soilbase model;
2- Model with a non-linear contact element

A 3m span beam has a rectangular cross-section of $1,3 \text{ m}^2$, reinforced concrete elasticity modulus $E_b = 2650 \text{ MPa}$, the beam half-width equal to $0,5 \text{ m}$; soilbase deformation modulus $E_o = 20 \text{ MPa}$; Poisson ration $\nu = 0,33$; depth of the slump depression 3 m ; radius of the slump depression 2 m .

The equally distributed load is equal to $0,1 \text{ MPa}$. The contact elements complies with the limitation for normal contact stress $\sigma_z \leq 0,3 \text{ MPa}$.

In conclusion we note that the "GROS-3" program is capable of selecting soilbase deformation characteristics that ensure specified settlements of the footing.

This solution can easily be applied to problems of rescuing buildings with the help of chemical or other techniques for stabilizing soil weakened by karstic cavities.

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NEW METHOD FOR CALCULATION OF NATM METHOD TUNNEL LININGS

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ABSTRACT

The use of elastic soil grid method to determine stresses in shotcrete lining interacting with the soil environment is described. An example for the application is presented.

In problems of designing real underground structures, the engineer is expected to model the structure, its material and the load. This is how the mechanical model for solving the problem arises. Stresses and displacements obtained from the mechanical model suit to determine strength, deformation and safety of stability. Results from field measurements on the structure accomplished (normal stresses, loads, curvature variations, deformations, displacements and rock field loosening) may be confronted to those obtained on the model, permitting a model correction that will improve the truth of the model to be next applied.

Even for the design of the simplest underground structure - design of a circular tunnel without hinges, embedded in a homogeneous elastic, isotropic and weightless rock surrounding - the problem above can only be written in terms of a partial differential equation system with eight unknowns, to be solved taking the given boundary conditions into consideration.

Thus, already in establishing the mechanical model, the engineer is forced to neglect for the sake of simplicity that significantly impair the truth of results. Now, the mathematician follows, introducing neglects into the differential equation system of the problem for the sake of the

feasibility of a mathematical solution, often without being able to realize either by himself or with the engineer the further biasing of the reality modelled.

This is why in the design practice of underground structures, numerical mathematical models are applied for the "exact" solution of problems, and realistic approximate methods using simplified mechanical models for the design at sketch plan level.

Advantages of the application of numerical methods are:

- they enable the engineer to establish a more realistic structural model, permitting him to track development of the final structure in building phases;
- there is an increased possibility for a realistic modelling of materials;
- only the primary stress state has to be assumed as an ideal load, namely, provided material, structure, building order are correctly modelled, loads are obtained by computations;
- the engineer is able to track technical consequences of variations of the mechanical model, rather than to lose sight of them in the maze of analytic solution often untraceable to an engineer.

Computations relying on numerical methods to be effective require proper development of laboratory tests and field measurements.

Realistic approximations by simple mechanical models usually rely on results of "exact" solutions, but the involved neglects permit simpler mathematical treatment.

Among numerical methods, FEM suits both great and small depth urban tunnels, provided there is proper modelling. A realistic output may be expected if also structure, material and load are truly modelled.

In modelling the structure, correct modelling of elastic or real hinges, possible voids and connections between wall lining and soil, as well as interaction between anchors and soil are of importance. To simulate connection between soil and wall, gap units have been developed, permitting simultaneous modelling of the empty back void, as well as of the connection meeting the Mohr-Coulomb condition of friction. Connection between anchor and soil has been modelled by elements that do not cut soil continuity as in reality but yield the possible anchorage force as

a function of soil stresses, anchor to soil friction and adhesion, reacting thereby also on soil stresses.

In modelling the soil, it has been attempted to correctly model structural behaviour of the experimental characteristic curve of the soil at failure, and the subsequent volumetric deformation possibility.

Loading has been simultaned by carefully taking all elements of the building sequence significantly affecting wall and soil stresses into consideration.

This modelling is an intricate, rather laborious process. Therefore an approximate method has been developed, to be presented below.

Among approximate methods for mountain tunnels build at great depths, having normally a single circular cross section or it is composed of circular arcs, approximate calculation for ultimate shear is applied, assuming horizontal and vertical stresses to be equal. Accordingly, dimensions, strength of the shotcrete lining, number and load capacity of anchors have to be chosen so that the soil core plasticized near the horizontal diameter does not cause shear failure in the built structure and the interacting rock zone.

This method is generally unsuitable for urban underground railway tunnels, namely:

- because of the small depth, the value of earth pressure at rest is lower by orders than that for maintain tunnels;
- the approximation of vertical and horizontal stresses to be equal is inadmissible;
- the load being lower, generally no plastic core can develop;
- structures delicate for building technology and from structural aspects, e. g. stations, are of composite cross sections, and even for outermost lateral tunnels, it is rather hardy to assume a circular cross section.

An approximate method will be presented below, involving rather realistic assumptions, and simple to be applied for the approximate calculation of underground engineering structures.

Approximate assumptions, mechanical model and load:

-
- Figure 10.10 consists of two diagrams. The left diagram, titled "Combined soil-r.f. concrete section", shows a cross-section of a foundation. It is divided into three horizontal layers: (1) R.f. concrete (top), (2) R.f. concrete (middle), and (3) Soil (bottom). The total width is $a = 1\text{ m}$. The vertical dimensions are t_1 , t_2 , and t_3 for the layers, and h_b and h_f for the total heights. The right diagram, titled "Spring pair of soil model", shows a spring pair model. It features a horizontal line representing the ground surface with a vertical line representing the centerline. A point i is marked on the centerline. A vertical force Q_n and a horizontal force Q_t are applied at point i . The angle between the vertical force and the centerline is α . The angle between the horizontal force and the centerline is 90° . The vertical displacement is $\Delta_{in}^{12} \equiv \Delta_{in}^c$. The spring forces are r_{i1} and r_{i2} kN/m, and the angles are ϕ and ψ .

Fig. 2

$$x_{si} = \frac{\frac{h_b^2}{2} + n \cdot \left(\frac{h_t^2}{2} + h_t \cdot h_b \right)}{h_b + n \cdot h_t}$$

$$S_{Ni} = a \cdot h_b \cdot E_b + a \cdot h_t \cdot E_t$$

$$I_i = \frac{a \cdot h_b^3}{12} + a \cdot h_b \cdot \left(x_{si} - \frac{h_b}{2} \right)^2 + n \cdot \left[\frac{a \cdot h_t^3}{12} + a \cdot h_t \cdot \left(\frac{h_t}{2} + h_b - x_{si} \right)^2 \right]$$

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Bending stiffness: $E_i \cdot I_i$

Cross section area: $F_i = \frac{S_{Ni}}{E_i}$

Cross section moduli for stress computation:

$$K_{1i} = \frac{I_i}{t_1} ; K_{2i} = \frac{I_i}{t_2} ; K_{3i} = \frac{I_i}{t_3}$$

Stresses of the ideal beam in lines 1, 2 and 3:

$$\sigma_{1i} = \frac{N}{F_i} \pm \frac{M}{K_{1i}} ; \sigma_{2i} = \frac{N}{F_i} \pm \frac{M}{K_{2i}} ; \sigma_{3i} = \frac{N}{F_i} \pm \frac{M}{K_{3i}} ;$$

Concrete and soil stresses:

$$\sigma_{1b} = \sigma_{1i} ; \sigma_{2b} = \sigma_{2i} ; \sigma_{2t} = \sigma_{2i} \cdot n ; \sigma_{3t} = \sigma_{3i} \cdot n$$

- The bar polygon has to be fitted to the beam cross section so as to have its break points lying on the centre of gravity of the ideal cross section.
 - The bar polygon has to be supported by the surface normal at break points by the pair of springs including the angle of friction.
 - Spring pair stiffness r has to be chosen so that stiffness Δ_{in}^{12} to compression in direction normal to the surface equals the compression stiffness Δ_{in}^c determined from the soil modulus of subgrade reaction c .
From equation $\Delta_{in}^{12} = \Delta_{in}^c$ the spring constant
- $$r_{i1} = r_{i2} = \frac{l_i \cdot b_i \cdot c}{2 \cdot \cos^2 \varphi}$$
- where l_i - arc length at node i ;
 b_i - tested tunnel length;
 c - bedding factor.
- Polygon side lengths have to be taken as shorter in beam sections of larger curvature, and longer in sections of smaller curvature.
 - Rock pressure acting on the resulting interacting structure has to be determined according to the earth pressure theory, maybe from the value of pressure at rest, taking the ratio of cover thickness to free opening into consideration.

SOLVING THE PROBLEM ON A MECHANICAL MODEL

Stresses and displacements of the bar polygon (E_i, F_i, I_i, K_i) of elastic bedding, ideal material and stiffness under the given load may be determined by any suitable bar system program.

Statical model

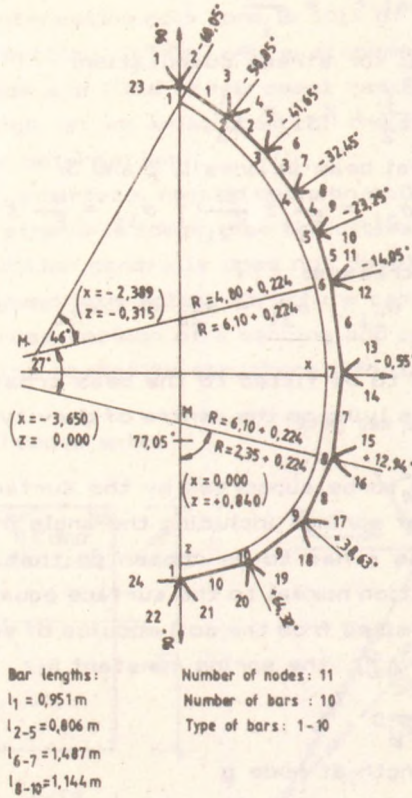


Fig. 3

Stresses and displacements of the beam of an ideal material are results obtained for a system with no spring in tension, but where every compressed spring is introduced to the stiffness matrix with all its stiffness. This condition may be achieved in several runs, so that after every run, tensile springs are omitted, but those becoming compressed again are returned to the stiffness matrix with their full stiffness.

The moment diagram is a curve plotted for a moment obtained for polygon nodes, the normal force diagram has been plotted for bar normal forces applied at polygon side centres.

In knowledge of moment and normal force diagrams, ideal stresses σ_x may be obtained from ideal cross section moduli K_x and areas F_x , then, multiplying by the n value, soil stresses result.

Computation results for a gallery of an upright elliptic cross section built according to NATM will be presented. The mechanical model and the load are seen in (Fig. 3).

	x, m	z, m	Angle °	P _x , kN/m	P _z , kN/m
1	0,000	- 4,750	- 90,00	360	300 kN/m
2	+ 0,783	- 4,211	- 50,85	364	
3	+1,365	- 3,654	- 41,65	369	
4	+1,851	- 3,011	- 32,45	375	
5	+2,227	- 2,298	- 23,25	379	
6	+2,485	-1,535	-14,05	386	
7	+2,674	-0,061	- 0,55	399	
8	+2,510	+1,417	+12,95	412	
9	+2,011	+2,447	+38,63	421	
10	+1,115	+ 3,160	+64,32	428	
11	0,000	+ 3,414	+90,00	430	

Table 1

Nodal coordinates and inflections of surface normals angle.
 P_x and P_z are respective nodal ordinate of horizontal and vertical of the load diagram

Input data have been compiled in (Table 1), results in (Table 2) and (Table 3).

Computations show that - in accordance with result of exacter computation methods - in the concrete wall, stresses from normal forces prevail. Stresses from moments are responsible at most for 2 % of that. Thus, in the wall only compression develops, modulated by bending by at most 2 %.

Among stress excess in the rock sheath, the maximum is 176.4 kPa mere 13.7 % of the mean anisotropic linear compressive strength.

Bar	Normal force kp	σ_{Ni} kp/cm ²	Moment kp.cm	σ_{Mi}^1 kp/cm ²	σ_{Mi}^2 kp/cm ²	σ_{Mi}^3 kp/cm ²
1	-154 659	-66,068	+1837400	+0,383	+0,041	-2,521
2	-161 719,5	-69,084	+4214 150	+0,878	+0,094	-5,783
3	-160 349	-68,499	+5122 000	+1,067	+0,114	-7,029
4	-159424	-68,103	+5685950	+1,184	+0,127	-7,803
5	-159072,5	-67 953	+5872550	+1,223	+0,131	-8,059
6	-158872,5	-67,868	+4912 300	+1,023	+0,110	-6,741
7	-164 388	-70,224	+1486750	+0,310	+0,033	-2,040
8	-177 162,5	-75,681	-2601 350	-0,542	-0,058	+3,570
9	-181427,5	-77,503	-4263000	-0,888	-0,095	+5,850
10	-173326,5	-74,042	-4007900	-0,835	-0,089	+5,500

Table 2

Ideal stresses in lines 1, 2 and 3 from normal force σ_{Ni} and mome
(σ_{Mi}^1 , σ_{Mi}^2 and σ_{Mi}^3)

Bar	σ_1^b kp/cm ²	σ_2^b kp/cm ²	σ_2^t kp/cm ²	σ_3 kp/cm ²	σ_3^t kp/cm ²	
1	-65,685	-66,027	-1,501	-68,589	-1,559	
2	-68,206	-68,990	-1,568	-74,867	-1,702	
3	-67,432	-68,385	-1,554	-75,528	-1,717	
4	-66,919	-67,967	-1,545	-75,906	-1,725	
5	-66,730	-67,822	-1,541	-76,012	-1,728	
6	-66,845	-67,758	-1,540	-74,609	-1,696	
7	-69,914	-70,191	-1,595	-72,264	-1,642	
8	-76,223	-75,739	-1,721	-72,111	-1,639	
9	-78,391	-77,598	-1,764	-71,653	-1,628	
10	-74,877	-74,131	-1,685	-68,542	-1,558	

Table 3

Concrete and soil stresses in lines 1, 2 and 3

STRESS DETERMINATION AND STRUCTURAL ANALYSIS OF SUPERFER REINFORCED TUNNELS

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Abstract: A mechanical model has been developed for determining stresses in, and analysis of a reinforced pipe section on elastic bedding, acting as a composite beam. Its application is illustrated by a numerical example.

Urban canalization systems suffer serious damaging from natural ageing, increasingly aggressive sewages, thereby leaking sewage penetrating the subsoil impairs the groundwater of entire cities, while important volumes of groundwater penetrating the water supply network overburden sewage purification plants of anyhow restricted capacities.

In all Europe, intensive renewal activities of these canalization systems have been launched. Several new procedures have been developed, and usually, patented. Among them, those exempt from soil surface opening up are preferred. Still better are methods not requiring to construct new canalization but renewal of the old one so as to make it permanently resistant to increasingly aggressive sewage and to meet operational requirements for another 50 to 100 years.

Such a method has been developed by the Superaqua division of AL-HAZ Ltd., and applied in several European countries.

Its essential consists in filling pipe deficiencies with concrete, applying Superaqua procedure for saturating and strengthening the cracks and the surrounding soil, then, if needed, a high-strength, compact, abrasion resistant special Superfer sand concrete internal layer about ≈ 2 cm

thick is made by grouting. Now, a design method - suiting a desk computer - for strength checking structures reinforced by this method will be presented.

Superfer-Superaqua reinforced tubes are structures of two materials of different stiffnesses and strengths, interacting in interface shear limited by the strength of the weaker material, exposed to bending and compression, embedded in surrounding, well consolidated, compact soil.

From those above (reinforced structure, compact, consolidated surrounding soil) it is manifest that crackfreeness rather than stability is the critical requirement in strength analysis.

To determine stresses and deformations, the reinforced pipe section is modelled by a polygon in its rectilinear axis consisting of bars of complex cross section, stiff to bending at inflection points, supported at the inflection points by a pair of springs modelling the soil, including an angle φ with the surface normal. Model load is determined according to given specifications or by some rock pressure theory. Pipe wall of two different materials. Omitting deductions (Fig. 1).

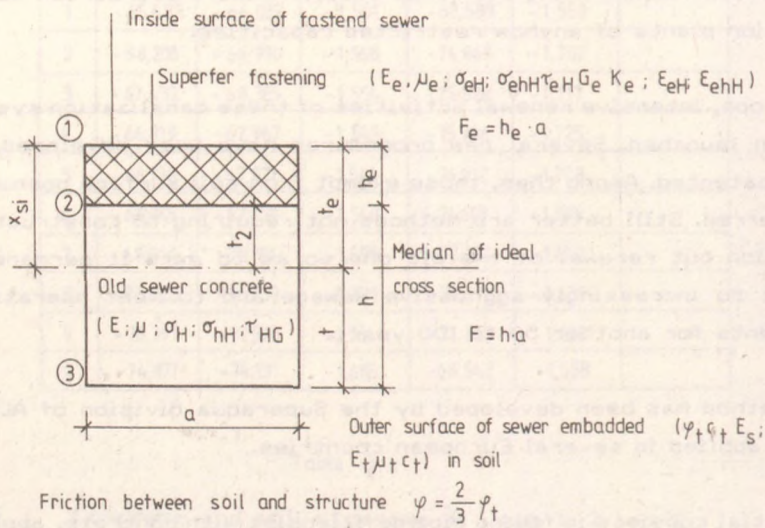


Fig. 1 Cross section of fastened sewer lining

If the pre-existent concrete has perfectly lost its strength so that it can be considered as equal to that of the surrounding soil, i.e. $h=0$, then $F_i = F_e = a \cdot h_e$, and its stiffness to compression $S_{M1} = a \cdot h_e \cdot E_e$.

Moment of inertia of the ideal cross section:

$$I_i = \frac{a \cdot h_e^3}{12} + a \cdot h_e \cdot \left(x_{si} - \frac{h_e}{2} \right)^2 + n \cdot \left[\frac{a \cdot h^3}{12} + a \cdot h \cdot \left(\frac{h}{2} + h_e - x_{si} \right)^2 \right]$$

where:

$$x_{si} = \frac{\frac{h_e^2}{2} + n \cdot \left(\frac{h^2}{2} + h_e \cdot h \right)}{h_e + n \cdot h}$$

(median of ideal cross section).

Bending stiffness:

$$S_{M1} = E_i \cdot I_i.$$

The ideal bending stiffness deduced above can only be reckoned with if computations result in no higher shear stress at the interface of the two materials than the ultimate shear stress τ_{th} of the weaker material, that is, of the concrete of the existing pipe. Interfacial shear stress in the ideal cross section is obtained by the Jourowski formula, that is,

$$\tau_i = \frac{T_i \cdot S_{ix}}{I_x \cdot a}$$

where T_i is shear in the examined cross section, and S_{ix} the static moment on the median of the ideal cross section of the part prone to displacement. At the same point, shear stress in the concrete of the pre-existing pipe $\tau = n \cdot \tau_x$. Thus, the previously deduced S_{ix} may only be applied if $\tau \leq \tau_{th}$.

At inflection points of the beam axis, linear springs stiffnesses for the soil model are:

$$r_{i2} = \frac{L \cdot B \cdot C_t}{2 \cdot \cos^2 \varphi}$$

where L - distance of two inflection points

and B - the examined pipe length.

Thus, $L \cdot B$ is the surface with a bedding effect simulated by the pair of springs (Fig. 2).

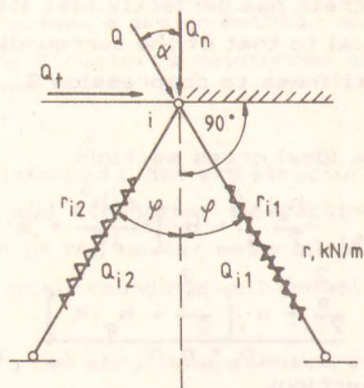


Fig. 2 Spring couple modelling the soil

Rather than the pair of springs, applying a single spring of surface normal direction, this one has a stiffness $r = L \cdot B \cdot C_e$.

In the formula, Winkler's bedding coefficient C_e is no material constant. Its value depends on the shape, curvature conditions and dimensions of the object. Published data comprise important uncertainties in concrete cases (Miner, Seastone, Koreniev and Tshernigovskaya), namely the indicated coefficient values are soil dependent, irrespective of the actual structure.

To eliminate this uncertainty and to obtain a reliable relationship between material constants determinable by laboratory tests - modulus of compressibility (E_u) in an oedometer, or modulus of elasticity (E_t) and Poisson's ratio in a triaxial cell - involving dimensions and curvature of the structure for definite load and bedding conditions, deformations of the pipe cross section embedded in elasto-plastic half-plane have been determined by a FEM program based on beam geometry and soil stiffnesses (E_t and μ_t). Thereafter, keeping numerical equidimensionality of the pipe wall, the problem has also been solved by the elastic soil lattice method, determining by iteration the bedding coefficient values C_e , giving the best agreement between pipe cross section curvature variations due to load determined both on bar system models and on half-plane models by the finite element method. In these analyses, differences between displacement square sums have been minimized.

From the obtained C_0 and E_s values the $\alpha^* = C_0 / E_s$ values have been derived and plotted in diagrams for circular and oval sections (Fig. 3).

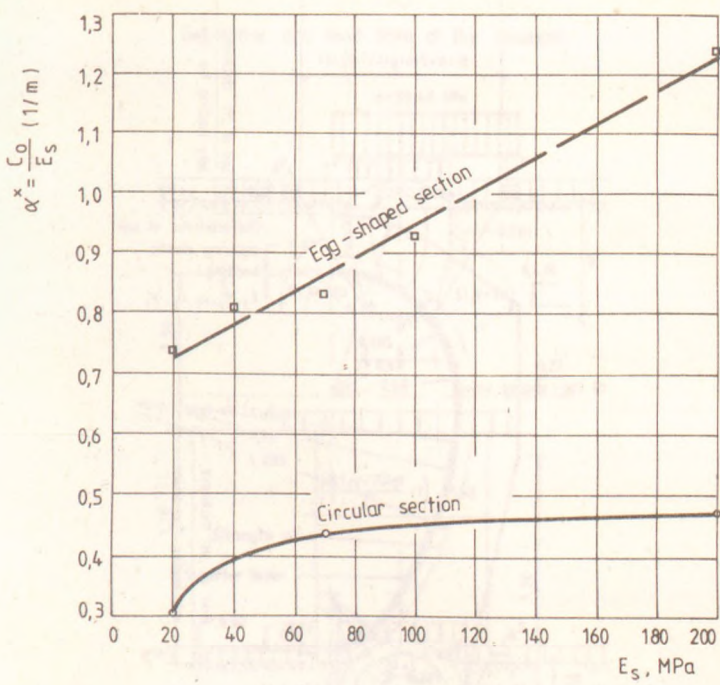


Fig. 3

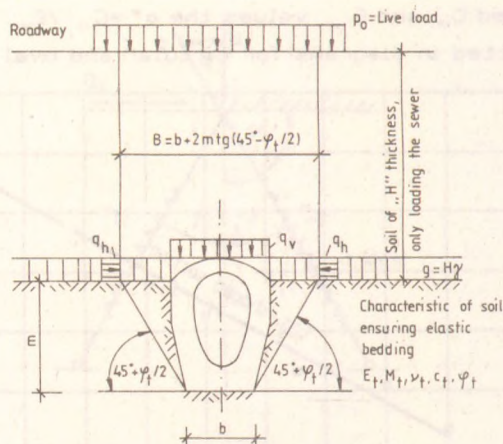
Using diagrams in (Fig. 3), bedding coefficients of oval or circular pipe cross sections are obtained as:

$$C_t = \frac{C_0}{R} = \frac{\alpha^* \cdot E_s}{R}$$

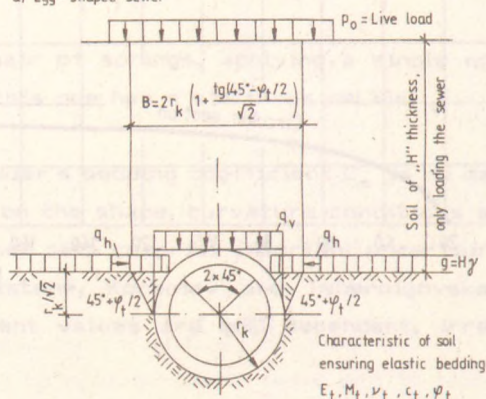
where R - radius of the outer surface arc of the pipe.

Soil surrounding the sewer partly imposes a load on it, partly supports it as an elastic medium - preventing horizontal deformation of the pipe, contributing to load bearing. In conformity with closer analyses and published data, load simulation involves the following - see (Fig. 4) - assumptions.

The outlined modelling of the pipe structure and its soil surrounding results in a mechanical model of bars with cross sections composed of



a) Egg-shaped sewer



b) Circular-shaped sewer

Loads on sewer in case of „a” and „b”

$$q_v = \frac{B\gamma}{2 \operatorname{ctg} \varphi_t} \left(1 - e^{-\operatorname{ctg} \varphi_t \frac{2H}{B}} \right) + p_0 e^{-\operatorname{ctg} \varphi_t \frac{2H}{B}} = g_1 + p_1$$

$$q_h = (1 - \sin \varphi_t) \frac{B\gamma}{2 \operatorname{ctg} \varphi_t} \left(1 - e^{-\operatorname{ctg} \varphi_t \frac{2H}{B}} \right) = k_0 g_1$$

where $k = 1$

Determination of loads on the sewer according to Terzaghi

Fig. 4

two different materials, of general position, of ideal stiffness, supported on pairs of springs against its surrounding, loaded by a force system uniformly distributed between its bars, that can be solved by a program for any general bar system. The problem presented as an illustration (Fig. 5) has been solved by means of a program made for a

computer Commodore 64, gradually eliminating all the tensile springs modelling the soil - as physical impossibilities.

Geometric and load data of the example

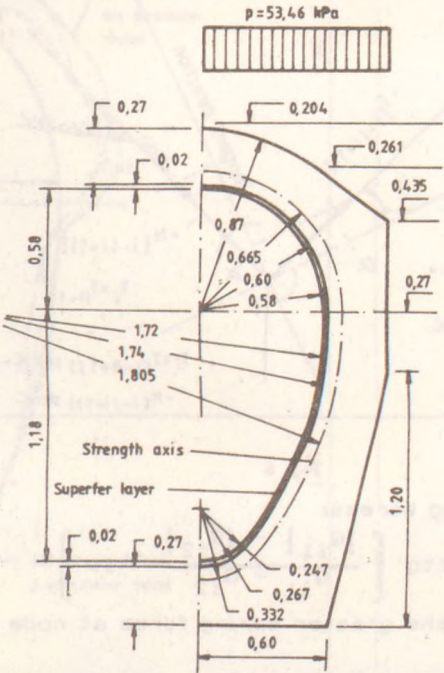


Fig. 5

Outputs included node displacements, bar normal forces and moments, and spring forces.

Exact values of normal and shear forces can be obtained from bar-end normal and shear force values at nodes, as tangential or normal projections of these values (Fig. 6).

Bedding stress value and direction are obtained from spring forces - see (Fig. 2). Let $\alpha < \varphi$ the angle included between bedding coefficient and surface normal, where φ is the angle of friction between soil and structure, then bedding reaction Q_1 due to the two compressive springs becomes:

$$Q_i = - \sqrt{Q_{i1}^2 + Q_{i2}^2 - 2 \cdot Q_{i1} \cdot Q_{i2} \cdot \cos(180^\circ - 2 \cdot \varphi)}$$

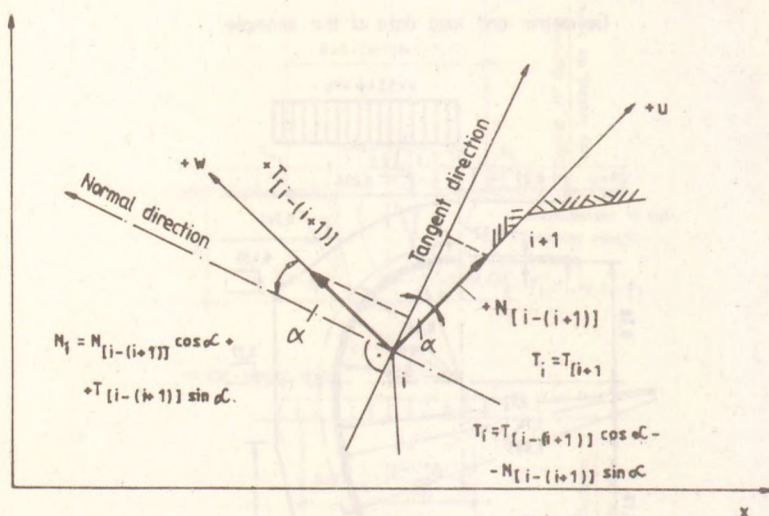


Fig. 6

Direction α of bedding stress:

$$\alpha = \arctg \left(\frac{|Q_{i1}| - |Q_{i2}|}{Q_{i1} - Q_{i2}} \cdot \operatorname{tg} \varphi \right)$$

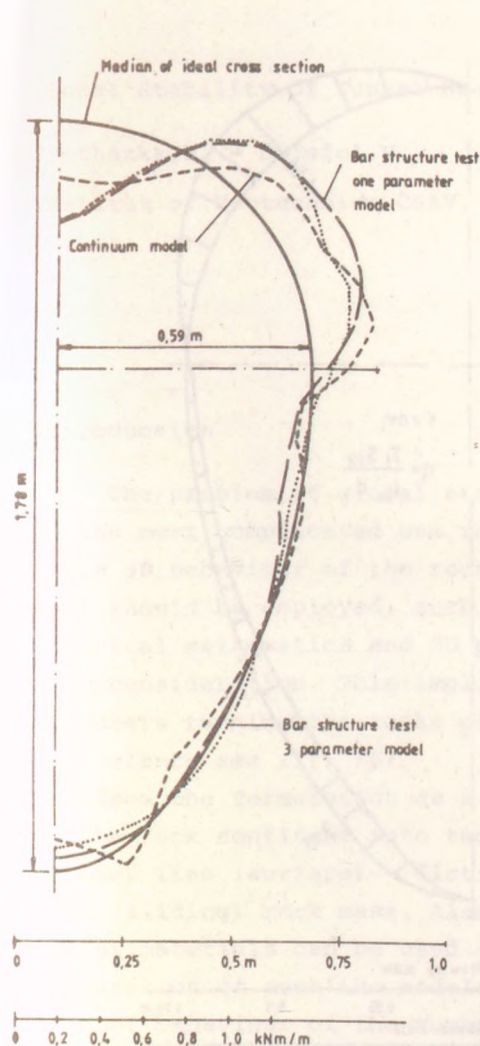
Where Q_{i1} is always the greater spring force at node i .

For the specific value of the bedding stress, the Q_i value has to be divided by product of mean value of arc lengths $[i-(i+1)]$ and $[i-(i-1)]$ by the length examined.

Establishing the mechanical model of the tunnel cross section and determining its bedding coefficient as described above, the resulting curvature changes, moments always somewhat exceed those resulting from a continuum model solution, an approximation on the safety side (Fig. 7).

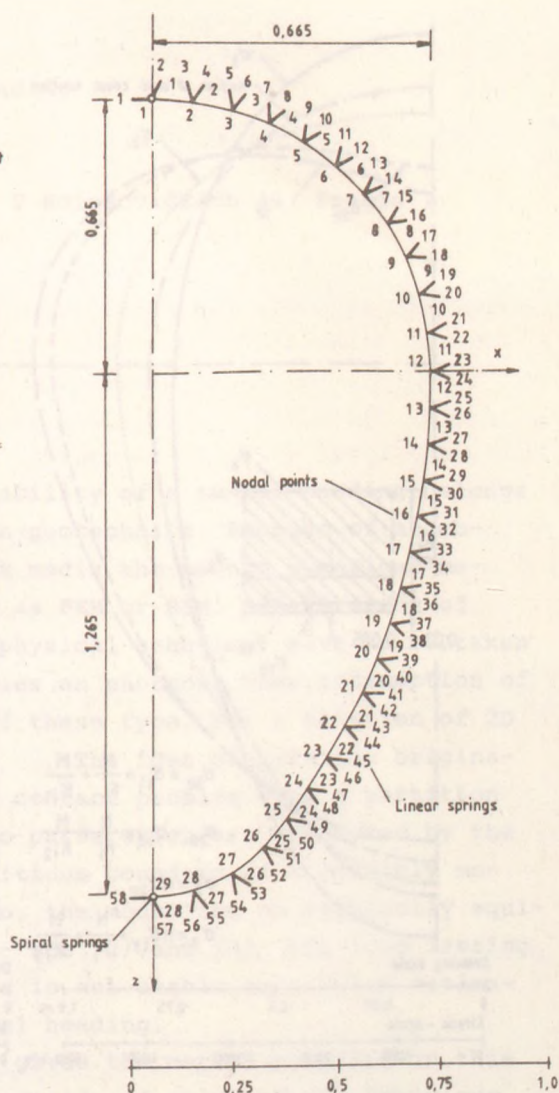
In knowledge of the stresses, it is examined whether the arising stresses in the pre-existing concrete after reinforcement, and/or at the interface do not exceed ultimate stresses of these materials.

Since impermeability and durability of the pipe would be impaired by cracks in the Superfer layer, it has been checked whether its maximum curvature change does not exceed ultimate deformation likely to cause



Moments of egg-shaped section calculated by bar-structure and continuum model

Fig. 7



Bar structure model of calculation example

Fig. 8

cracking of the Superfer layer. A test method has been developed for determining the ultimate curvature. mechanical model of, and stresses in, the example problem (Fig. 5), as well as relevant relationships are seen in (Figs 8, 9 and 10).

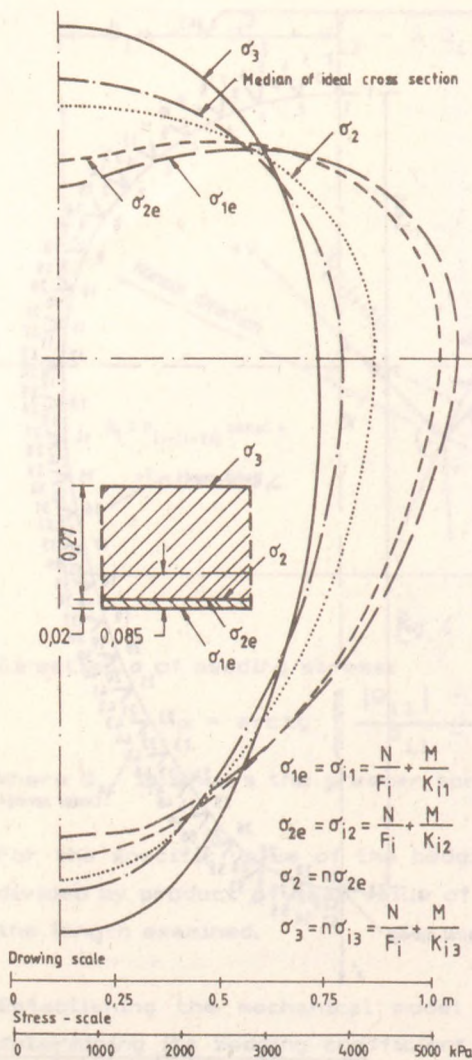


Fig. 9

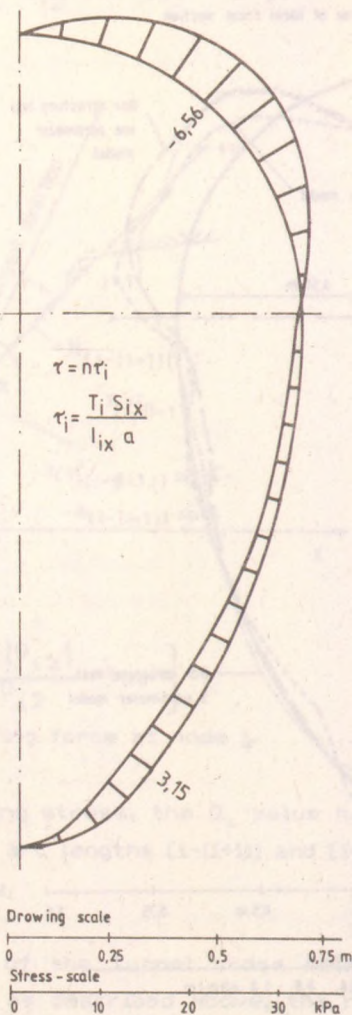


Fig. 10

Although the presented design method applies a simple mechanical model requiring no special programming and can be run with any bar system program, it yields a realistic result due to the realistic modelling of soil and structure stiffnesses, interactions and connections.

Global Stability of Tunnel Heading

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Introduction

The problem of global stability of a tunnel heading belongs to the most complicated one in geotechnics. Because of an obvious 3D behaviour of the rock media the modern numerical method should be employed, such as FEM or BEM, and 3D means of numerical mathematics and 3D physical behaviour have to be taken into consideration. This implies an enormous time consumption of computers running the tasks of these type. For a solution of 2D equivalents see /1/, /6/.

The idea of solution originates from the formulation as a contact problem with a partition of the rock continuum into two parts which are connected by the contact line (surface) - fictitious boundary of a possibly moving (sliding) rock mass. Also, the modelling on physically equivalent materials can be used - see /2/ and /3/. The long lasting preparation of such the models is not usable to a quick estimation of behaviour of the tunnel heading.

The extraordinary effect gives the method proposed in this paper. The sliding domain is considered only and its behaviour is derived from the simplified assumptions following the methods of Fellenius or Bishop's simplified method solving the global stability of slopes. In sense of optimal control the most probably shape of the domain is stated and according to the balance condition on the slip surface the safety factor of the tunnel heading is obtained.

The deformation of the rest of the rock media evidently influences the sliding domain. If we do not want to analyze the situation by means of numerical processes we can use the expe-

riences from the modelling on the physically equivalent materials. In the sequel the (possibly) simplest approximation of the fictitious boundary will be used. This approximation requests exactly one parameter which is delivered from the physical model.

1. Geometrical relations

In what follows the idea of the method of Fellenius is taken into account. The extension to the other methods is obvious. For the formulation in 2D see /5/.

First, we specify the shape of the admissible sliding domain G . The situation is illustrated in Fig.1. From above it is bounded by an elliptic paraboloid P and an elliptic cylinder Q , from below by the cylinder Q . If we consider the coordinate system and notation according to Fig.1 ($2s$ is a length of attack, α_x is an angle between the x -direction and the axis of Q , R is the radius of the stope), the equation of the cylinder Q reads:

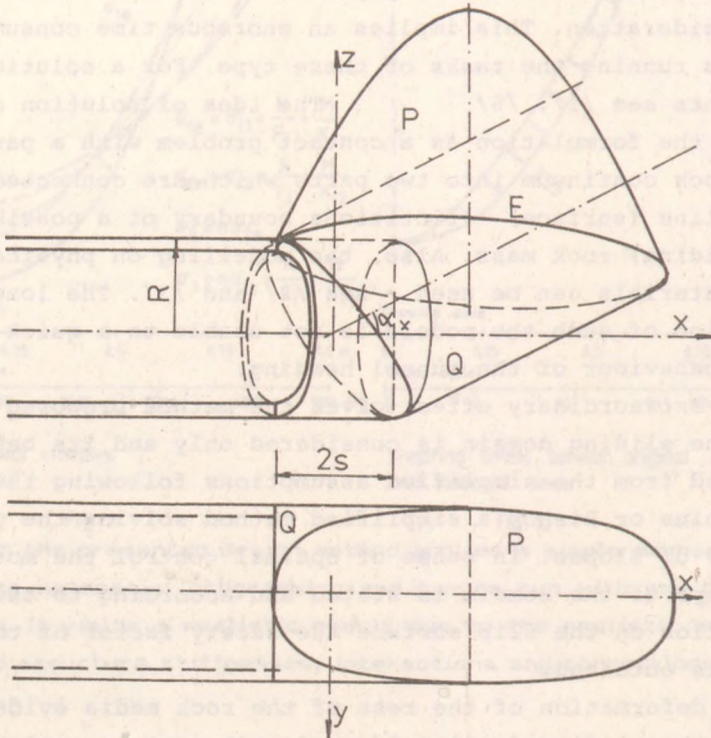


Fig.1 Geometry of the problem

$$\frac{(z - x \operatorname{tg} \alpha_x)^2}{a^2} + \frac{y^2}{R^2} = 1$$

where

$$a = s + R \cotg \alpha_x$$

From this the paraboloid P is described by the following equation

$$\frac{(x - s - a)^2}{A^2} + \frac{y^2}{B^2} = 2(z - R)$$

where

$$A = \sqrt{\frac{a}{k_x}}, \quad B = \frac{R}{\sqrt{ak_x}}$$

and k_x is a derivative of P prescribed at the point $(-s, 0, R)$ at the end of the tunnel wall. The value k_x will be taken from the experiences on the physical models. Alternatively, the derivative k_y of P can be prescribed at $(a - s, R, R)$.

The intersection of P and Q is an ellipse E:

$$\frac{(x - R \cotg \alpha_x)^2}{a^2} + \frac{y^2}{R^2} = 1$$

2. Variational formulation

Suppose the three dimensional extension of the Apriori Integration Method respecting the Fellenius model (see /5/). On the fixed slip surface - lower part of the cylinder Q - after a virtual displacement (assumed parallel to the plane Oxz) the measure of stability F can be expressed in the following manner (compare with /5/):

$$F = \frac{N \operatorname{tg} \varphi + C}{T}$$

where N is the vertical force projection to the normal of slip surface, C is the cohesion, T is the shear force from the unit weight projected to the direction of the virtual moving on the slip surface, φ is the angle of internal friction. With respect to the geometric considerations in the previous section and to

the Fellenius assumptions the quantities in F can be expressed in the following terms:

$$N \operatorname{tg} = \int_{-\infty}^{\infty} \int_{f(x,y)}^{t(x,y)} \gamma \operatorname{tg} \varphi \cos \alpha_x P_+(t(x,y) - f(x,y)) dy dx$$

$$T = \int_{-\infty}^{\infty} \int_{f(x,y)}^{t(x,y)} \gamma \sin \alpha_x P_+(t(x,y) - f(x,y)) dy dx$$

$$c \int_{-\infty}^{\infty} P_+(t(x,y) - f(x,y)) dS$$

where $P_+(p) = 1$ for $p > 0$, $P_+(p) = 0$ for $p \leq 0$, S is a true area of the slip surface, according to [4] equals

$$dS = \frac{\sqrt{1 - \sin^2 \alpha_x \sin^2 \alpha_y}}{\cos \alpha_x \cos \alpha_y} dx dy$$

where

$$\operatorname{tg} \alpha_y = \frac{R}{\operatorname{tg} \alpha_x} a.$$

The functions f and t describe the slip surface and the fictitious dome of breaking, respectively. The function f is a lower part of surface of the cylinder Q while t consists of the paraboloid P and an upper part of the cylinder Q . The situation is clear from Fig.1.

3. Numerical treatment

In this section the procedure for solving the integrals is proposed. The Gaussian quadrature can be applied in terms of Cartesian product formulas. The transformation

$$x = \frac{a}{c} \Theta, \quad y = \frac{\lambda}{c} \sqrt{c^2 - \Theta^2}$$

gives

$$\int_{x_0}^{x_1} \int_{-u(x)}^{u(x)} g(x,y) dy dx = \int_0^1 \int_{-b}^b g(\lambda, \Theta) \frac{a}{c^2} \sqrt{c^2 - \Theta^2} d\Theta d\lambda$$

where $a^2 = b^2 + c^2$, $\Theta_0 = \frac{x_0 c}{a}$, $\Theta_1 = \frac{x_1 c}{a}$,

$u(x) = b \sqrt{1 - \frac{x^2}{a^2}}$. The last integral admits the product formula of the following type:

$$\int_{-1}^1 \int_{-1}^1 g(p, q) dp dq = \sum_{i=1}^{n_1} A_i^1 \sum_{j=1}^{n_2} A_j^2 g(p_i, q_j)$$

4. Physical modelling

In Fig.2 the curves of the same value of vertical deformations are drawn in front of the tunnel heading. The values are obtained from the physical model of the Prague underground lines.

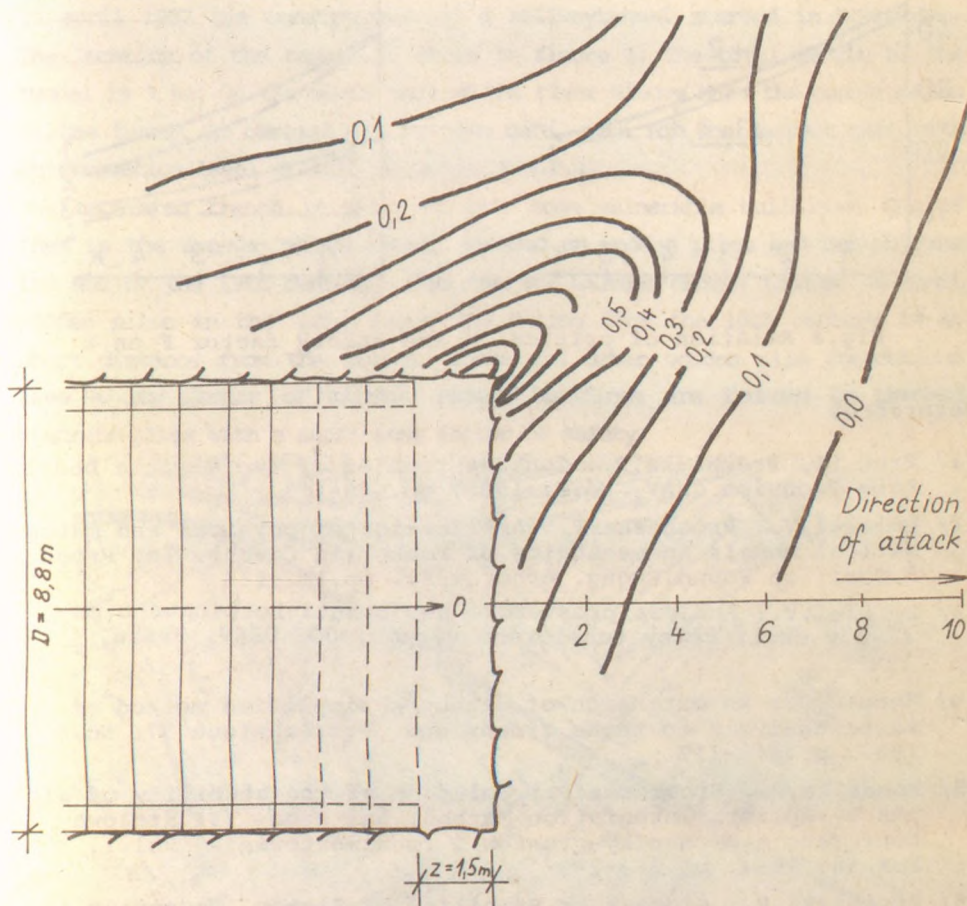


Fig.2 Vertical displacements

5. Example

For the unit weight $\gamma = 20$ kPa the following four cases of material properties and geometry were tested ($R = 2.6$ m, c is a cohesion on a unit length):

1. $s = 0.5$ m, $\varphi = 5^\circ$, $c = 10$ kPa
2. $s = 0.75$ m, $\varphi = 5^\circ$, $c = 20$ kPa
3. $s = 0.5$ m, $\varphi = 11^\circ$, $c = 10$ kPa
4. $s = 0.75$ m, $\varphi = 11^\circ$, $c = 10$ kPa

In Fig.3 the critical angle α and safety factor of the tunnel heading in relation with k are described.

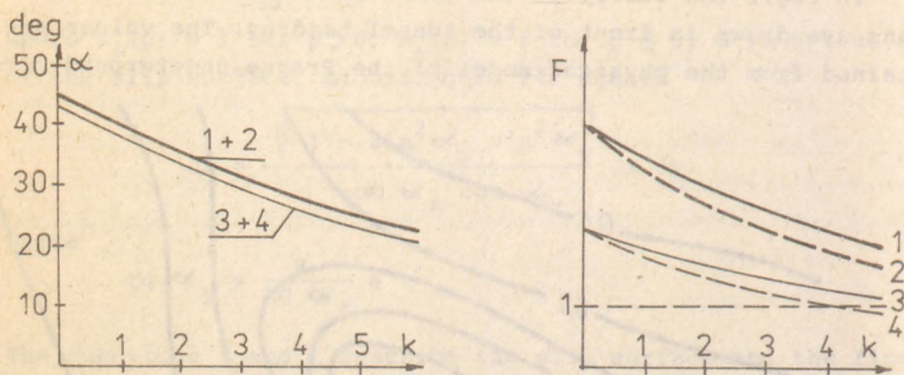


Fig.3 Relation of critical α_x and safety factor F on k

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THE CONSTRUCTION OF THE BUILDING TRENCH FOR A RAILWAY TUNNEL IN ROTTERDAM

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Introduction

In april 1987 the construction of a railwaytunnel started in Rotterdam. The location of the tunnel is shown in figure 1. The total length of the tunnel is 3 km. On the north bank of the river Nieuwe Maas the construction of the tunnel is carried out in open cuts, even for the deepest part with an excavation level of 18.0 m- datum (N.A.P.).

The excavated trench is situated near some vulnerable buildings. One of them is the famous "Witte Huis", founded on wooden piles and dating from the end of the 19th century. Also the St. Laurens church founded on short wooden piles in the upper layer and dating from the 15th century is at short distance from the trench. These and other wooden pile foundations have a low factor of safety. Recent buildings are founded on precast concrete piles with a sufficient factor of safety.

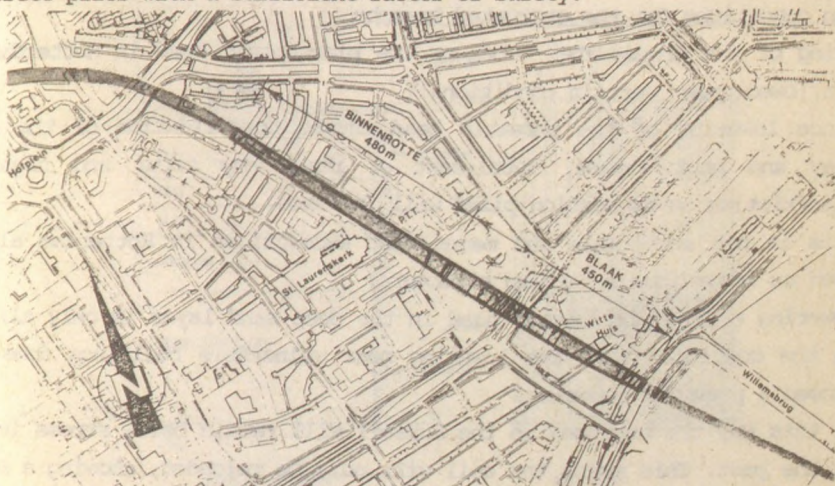


FIG 1 GROUNDPLAN OF THE TUNNEL

Soil conditions

The design of the excavation was determined by the geotechnical conditions: the soil, the groundwater and the building area. The soil conditions are shown in figure 2, representing the soil profile along a part of the trench near the north bank of the river. The phreatic watertable lies between 1.0 and 2.0 below groundlevel. The piezometric head of the groundwater in the pleistocene sandlayer is 1.0 m- N.A.P. near the river and varies to about 2.0 m- N.A.P. level at the northern part of the location. In figure 2 also the bottom line of the tunnel is shown.

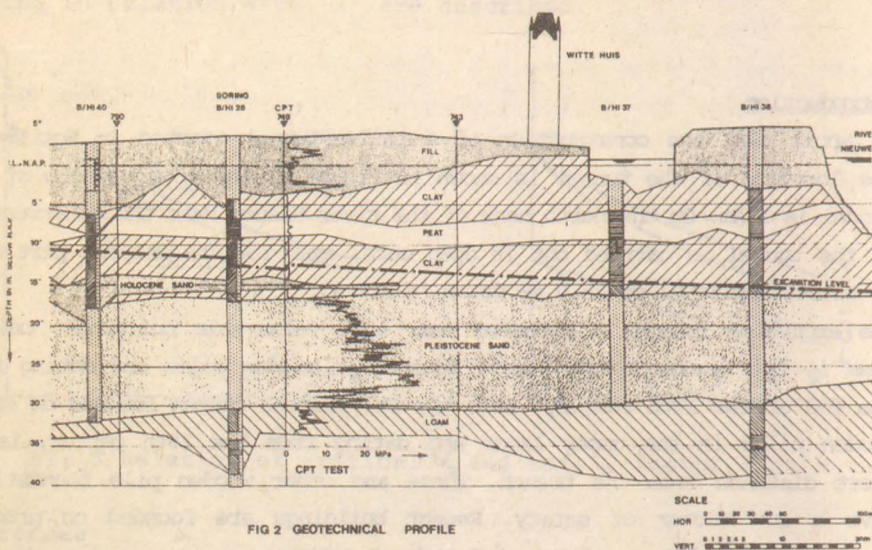


FIG 2 GEOTECHNICAL PROFILE

Design conditions for the excavated trenches

Lowering of the groundwater tabel or the piezometric head in Rotterdam is only allowed under certain conditions:

- since lowering of the phreatic head causes consolidation of the soft clay and peat layers, settlement of roads may occur and negative skinfriction on foundation piles will increase.

This is not acceptable and means that excavations in Rotterdam always need an impervious retaining structure;

- lowering of the piezometric head in the deep sand layer is only allowed on the condition that the lowering near vulnerable buildings does not exceed a previous lowering.

In this way the stresses in the subsoil will remain below stress levels in the past. This means the soil will only be reloaded, showing a stiff behaviour and usually resulting in acceptable settlements.

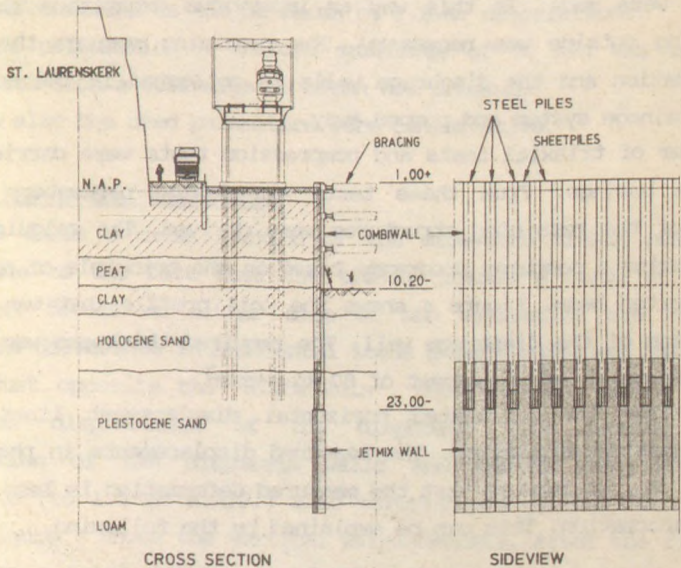
Some deep parts of the excavation required a lowering that exceeded the previous lowering. In these cases techniques such as deep infiltration of groundwater or construction methods such as underwater concrete or chemical injection of soil layers were used.

The walls of the excavations consist of sheet-pile walls, combi-walls or diaphragm walls. Which type was to be used depended on the retaining height, the expectance of vibrations due to hammering and the limit for deformation in the adjacent soil.

Excavation at location "Binnenrotte"

In the design of this part of the excavation the effect of deformation behind the retaining structure had to be taken into account. The allowable deformations depend on the existing buildings along the trench, such as buildings on concrete piles without prestressing, the existing railway viaduct on wooden piles and the St. Laurens church. The allowable horizontal deformation varies from 30 mm near the church to 90 mm at other places.

As retaining structure a combi-wall solution was chosen. The combi-wall exists of steel tubes with a diameter of 1.016 mm spaced at 2.30 m with sheet-piles inbetween. The length of the piles is about 23 m, several meters into the pleistocene sand formation. At 2 or 3 levels, depending on the allowable deformations, struts are used.



CROSS SECTION
FIG 3 BUILDING TRENCH BINNENROTTE

From the groundwater conditions it was concluded that near the St. Laurens church lowering of the piezometric head of the pleistocene sand was allowed to 4.0 m- N.A.P. The required lowering in the excavation nearby is 8.75 m- N.A.P. Even with deepwell infiltration this difference in head over the distance of 20 m is not realistic. Therefore it was necessary to install an impervious skirt between the bottom of the holocene formation and the top of the deep clay formation as shown in figure 3.

From geohydrological calculations it was concluded that the length of this skirt should be 250 m. With the aid of deepwell infiltration at the outside of this skirt it was possible to realise the required difference of piezometric head of 4.75 m. The conductivity of the pleistocene sandlayer amounts 900 m² per day. These calculations were carried out with a finite element model. The skirt was installed by the jetgrout system, in this case consisting of jetgrout columns alternated by jetgrout panels.

Excavation at location "Blaak"

From a comparison of costs of different building methods it turned out that for the deepest part of the excavation, it was advantageous to use the impervious clay layer of the Kedichem formation at 30.0 m- N.A.P. as a seal against groundwater. The retaining structure had to be extended into this layer. Diaphragm walls to 33.0 m- N.A.P. were used for this purpose. At the bulk head of the excavation and to subdivide the excavation, bentonite cement walls were made. In this way an impervious excavation was formed and no pumping outside was necessary. The remaining seepage through the Kedichem formation and the diaphragm walls is collected in the excavation by an open drainage system and pumped away.

A large number of triaxial tests and compression tests were carried out on samples from borings. From these tests the design parameters for the calculation of the retaining structures were derived. The calculation was carried out using a computer programme based on the principle of an elastoplastic supported beam. Figure 4 shows the soil profile that was used for the calculation of the diaphragm wall. The required thickness was 1.200 mm with a percentage of reinforcement of 80 kg per m³.

Figure 5a shows the calculated horizontal displacement lines at the different stages of excavation. The measured displacements in phase 4 are shown in fig. 5b. It appears that the measured deformation is less than the calculated deformation. This can be explained by the following.

	DENSITY γ kN/m ³	FRICTION ANGLE φ	COHESION c kN/m ²	WALL FRICTION δ	K kN/m ³
+3.0 SAND	18.0	30.0	0.0	20.0	5000
N.A.P.					
-4.0 CLAY	20.0	30.0	0.0	20.0	5000
-7.6 PEAT	16.0	25.0	8.0	12.5	3750
-10.0 CLAY	11.0	17.5	8.0	0	2000
-15.5 SAND	16.0	25.0	10.0	12.5	3750
-16.5 CLAY	18.0	27.5	10.0	14.0	5200
-31.0 CLAY	20.0	32.5	0.0	20.0	30.000
					45.000
	19	27.5	10.0	14.0	10.000

FIG. 4 SOIL PROFILE

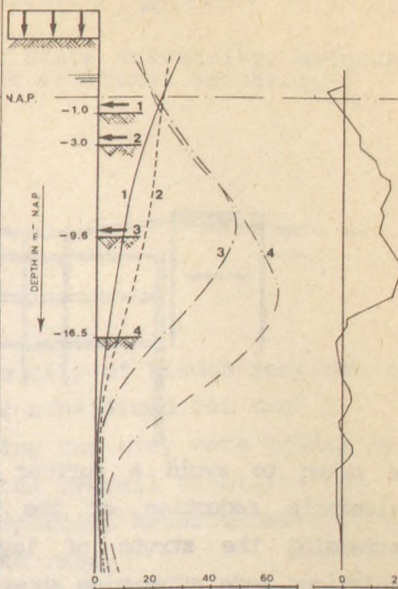


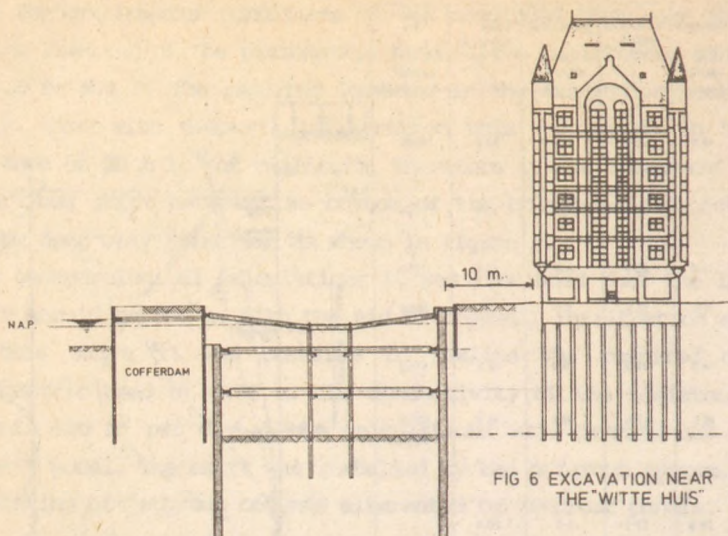
FIG. 5^a CALCULATED DISPLACEMENTS

FIG. 5^b MEASURED DEFORMATION

- The used calculation method neglects arching in soils. Arching results in higher strut loads and reduces the bending moments and deformations as can be shown by F.E.M. calculations.
- In the calculation a uniform surcharge of 20 kPa was introduced, in practice such a surcharge is often not present.
- Maybe also the used parameters were conservative.

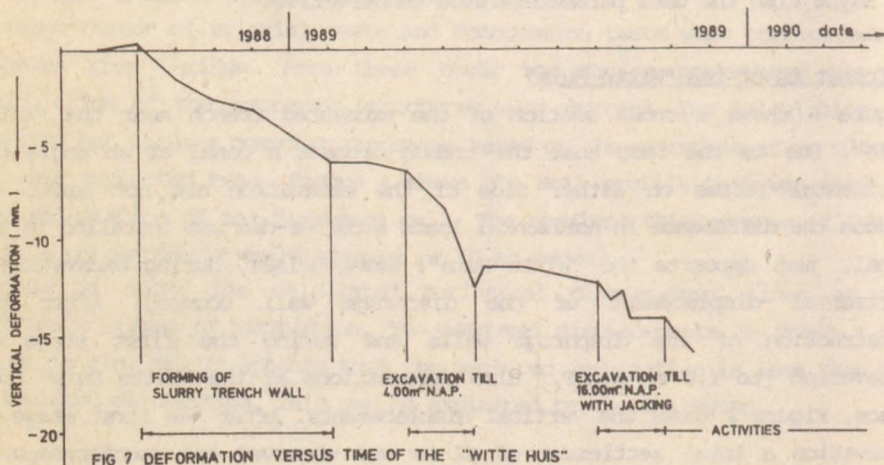
Deformation of the "Witte Huis"

Figure 6 shows a cross section of the excavated trench near the "Witte Huis". Due to the fact that the trench crosses a canal at an angle the horizontal forces on either side of the excavation are not equal. To reduce the difference in horizontal loads a coffer-dam was installed in the canal, just opposite the "Witte Huis". Nevertheless, during excavation a horizontal displacement of the diaphragm wall occurred. After the construction of the diaphragm walls and during the first stage of excavation (to 1.0 m- N.A.P.) also deformations at the "Witte Huis" took place. Figure 7 shows the vertical displacements. After the first stage of excavation a total settlement of 12 mm was observed and countermeasures



were taken to avoid a further increase. It was possible to gain a considerable reduction of the settlements of the "Witte Huis" by prestressing the struts of layers 2 and 3 with hydraulic jacks. Calculations made afterwards showed that the deformation of the diaphragm wall with and without the countermeasures amounted respectively 40 and 70 mm.

Subsequently the settlement of the "Witte Huis" based on this calculated deformation of the diaphragm wall was estimated at 36 mm. This settlement was unacceptable. Further calculations showed that with jacking the calculated settlement reduced to 20 mm. The actual observed settlement after completion of the trench was 17 mm.



STABILIZING CANAL DIKES IN A MINE SUBSIDENCE URBAN AREA

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SUMMARY

Over the past 25 years the city of Eisden-Belgium, due to deep covered mining activity, has subsided for over 7 meter. The dikes of the canal crossing the city were gradually heightened. For some cross sections the overall stability was questionable, also because of the important heterogeneity of the necessarily stepwise constructed dike mass.

This being a threat for the city of Eisden, a thorough stability analysis was made, taking into account different seepage models. Finally a soil nailing and gravel column improvement method was applied to some cross sections of the dike in order to reach the required safety conditions.

SITE CHARACTERISATION

In some area of the East of Belgium soil subsidence has been going on for many years, due to recently abandoned mining activity at depths of about 800 m below ground level. The deep mining technique was one of total extraction, with collapse of the roofs. This resulted in important settlements at the surface which can be estimated using the "zone method".

The boundaries of influence extend far beyond the excavations. The zone method estimates surface subsidence using a series of zones or circles around a surface point. Soil subsidence is proportional to the sum of the percentage of each zone being extracted at depth. Empirical zone or subsidence factors are required for the estimation (ref. 1 and 4). The well matching experimental settlements have been calculated according to the previous method, (fig. 1).

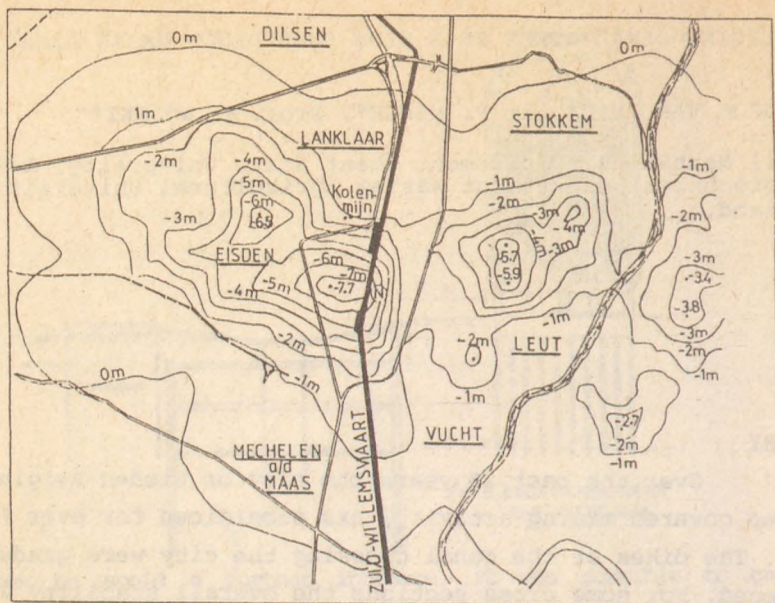


Fig. 1 : Iso settlement curves

It appears that a canal is crossing the region with the most important overall subsidence of ± 7.7 m occurring close to it. The embankments of the canal in this respect had to be heightened several times ; the actual water level in the canal being ± 8 m above the today's natural ground level. The subsidence not only causes problems to houses, constructions, sewers,... but also to the dike stability and the whole of the hydrogeological system. The natural groundwater level has to be lowered continuously in order to prevent the area from being flooded. A mean amount of $30,000 \text{ m}^3$ water a day has to be pumped out of the ground ; partly used as drinkwater and partly removed from the area through the canal.

The heterogeneous natural ground layers, dating from a period of arid climate conditions during glacial and interglacial, consist predominantly of gravel, loam and sand. The stage construction of the canal dikes over a period of several decades, causes important heterogeneities in their general build-up. Out of preliminary rough stability analyses, a critical slope stability section of about 500 m was isolated. In order to perform a more rational and systematical soil investigation, different cross-sections (profiles) - named from A to P - were chosen ; each of them investigated very carefully.

At least two borings with sampling and vane tests and two CPT were performed in each profile. Disturbed and undisturbed samples were reserved for the laboratory tests. Also porepressure measurement devices were installed at different levels in the borings, so as to be able monitor continuously the porewaterpressures in the dikes. Some field permeability tests were performed using the "falling head" method. In the laboratory, apart from the general physical tests, triaxial, direct and simple shear and also vane tests were performed on the undisturbed samples. In order to gather some information on the shear resistance of the disturbed samples, direct shear tests were performed on large size samples. This was mainly done because of the non negligible amount of gravel in the samples. Additionally, oedometer tests were performed to gather some information about stress history of the soil and deformation behaviour.

Out of the cross sections, longitudinal sections were obtained allowing, in combination, for a more or less three dimensional view of the embankment and its layering. The shear characteristics for drained and undrained conditions, used for the calculations, are the result of a profound analysis of all results of field and laboratory tests. The mean volume weight for each layer was found to be nearby at constant value of $\gamma_n = 21 \text{ kN/m}^3$ in saturated and $\gamma_d = 17 \text{ kN/m}^3$ in dry conditions. An example of a critical dike profile is given in fig. 2.

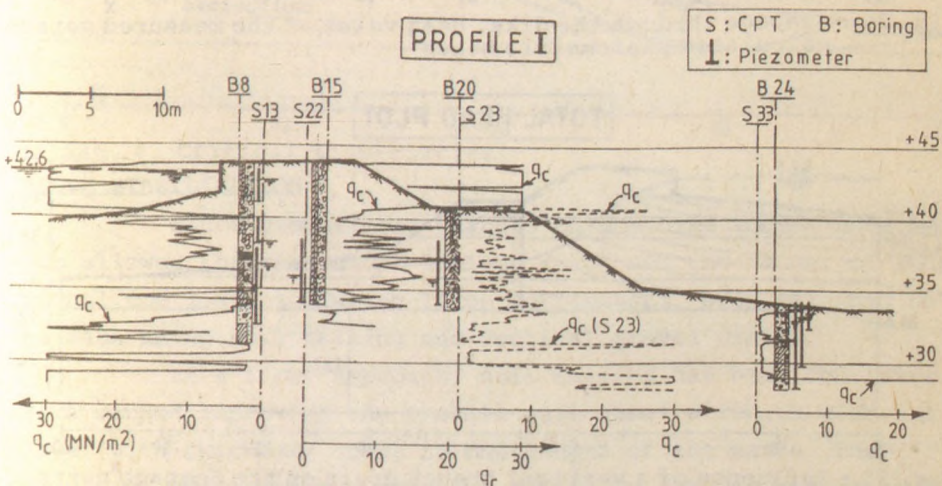


Fig. 2 : Typical dike profile with indication of in situ tests and mean piezometric head levels

SEEPAGE PROBLEMS

The monitoring of the porewater pressures over a period of about 8 months lead to valuable information for calculating the best fitting flow pattern and the related porewater pressures throughout the dike..

In the flow net given in fig. 3a, obtained by means of a finite element seepage program, mean values of the in situ measured porewater pressures out are indicated. Local seepage also could be detected at the toe of the embankment, giving rise to the danger of erosion at the toe of the dike. In order to avoid this it is necessary to provide reliable drains at the toe of the embankment and even better at the intermediate stage. Fig. 3b shows the influence of a vertical drain (gravel drain) on the flow pattern.

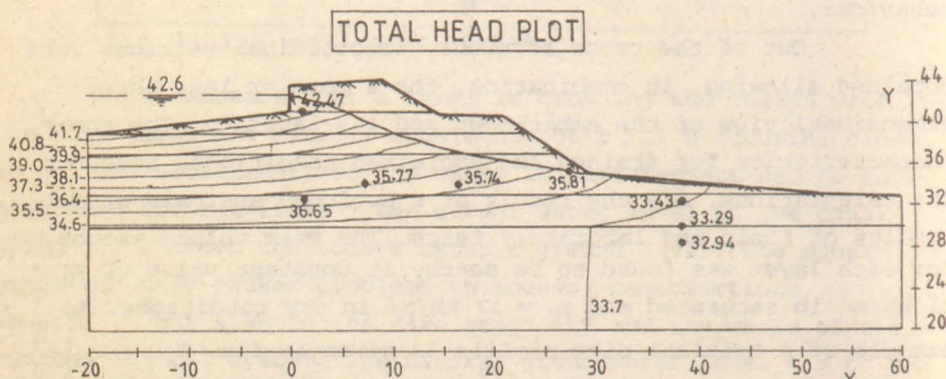


Fig. 3a : Seepage through the dike. Mean value of the measured versus calculated piezometric heads

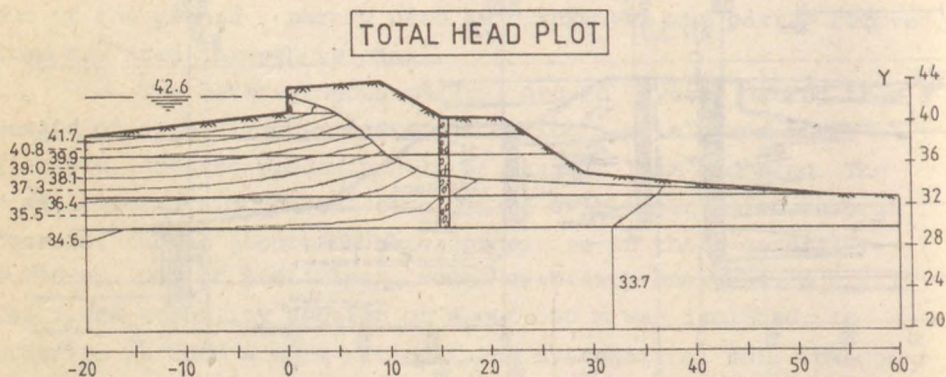


Fig. 3b : Influence of a vertical trench drain on the Seepage pattern

STABILITY ANALYSIS

Geometry, soil layering, dike material characteristics and the flow net, being given, it was possible to perform a finite element stability analysis. In fig. 4 the obtained most critical slip surfaces using the Bishop method are plotted. Some circles with safety factor $F_s \approx 1$ were only superficial slip surfaces which can be excluded taking into account possible local densification and suction due to vegetation at the surface. A more dangerous slip surface however is the one with safety factor $F_s = 1.16$, because, out of a completed stability analysis, this surface was able to trigger an important progressive failure mechanism. Moreover minor changes of the seepage pattern apparently gave rise to important decreases of the safety factor. This emphasised once more the necessity for a reliable drainage system.

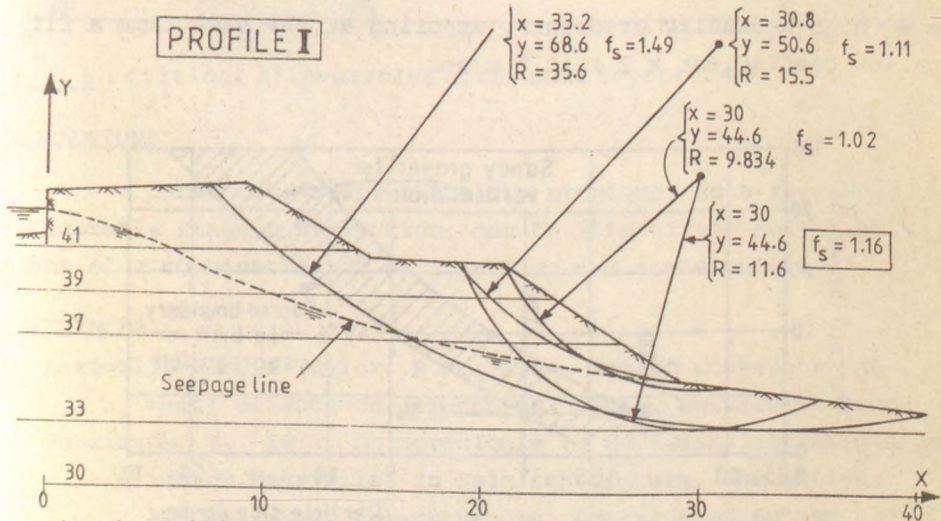


Fig. 4 : Critical slip-circles

SLOPE STABILISATION

Since geometrical changes on the dike embankments were not allowed for reasons of limited space and the danger of additional toe loading causing large settlements, proposals had to be made using soil nailing and vertical gravel drains.

In a first approach, soil nailing has been considered as a method improving the overall soil shear strength within a layer with thickness equal to the length of the nails. The resulting uprise of the cohesion c finally lead to a reasonably introduced value of $c = 4$ kPa.

The material used for the vertical drains has to secure both drainage and filter requirements : it means that vertical drains having sufficient water conductivity should prevent at the same time erosion in the surrounding soil. On account of the heterogeneity of the dike layering and the diversity of the dike material, the vertical drains have to fulfill the filter criteria for the finest soil which is sandy clay. In addition, the vertical drains will be working in rather unfavorable conditions. Due to the layered structure of the dikes, local but very high seepage gradients are likely to develop. Sandy gravel material, fulfilling the criteria of the so called "critical" filter introduced by Sherard et al (ref.3) is proposed for the vertical drains. The granulometric boundaries of filter material are given in Fig. 5, for guaranteeing effectiveness in soils with d_{80} varying from 0.03 to 0.1 mm at high hydraulic gradients, assuring at the same time a filtration coefficient $k \geq 1.10^{-4}$ m/sec.

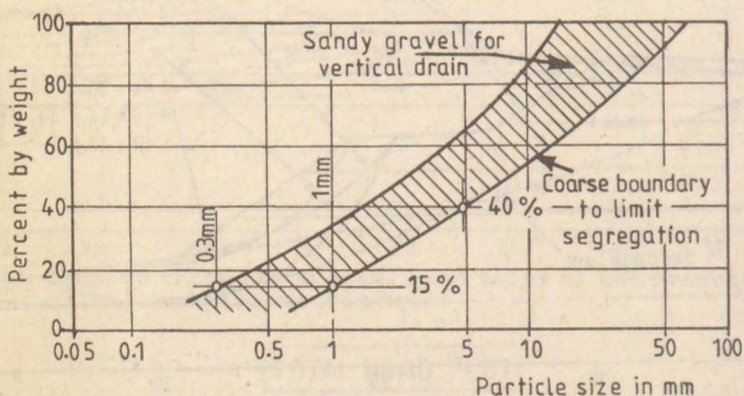


Fig. 5 : Granulometric bands of material proposed for vertical drains

When the spacing of the stone columns is carefully chosen, they secondly allow for an overall soil improvement. The interaction of the stone columns with the embankment dike material has been most favourable in the stability calculations. Installed, at interdistances of about $3 \phi_{col}$ in two zigzag rows next to each other, the equivalent stone column improvement was simulated as a continuous gravel wall of 1 m of thickness.

Fig. 6 shows the result of such stability improvement for profile I.

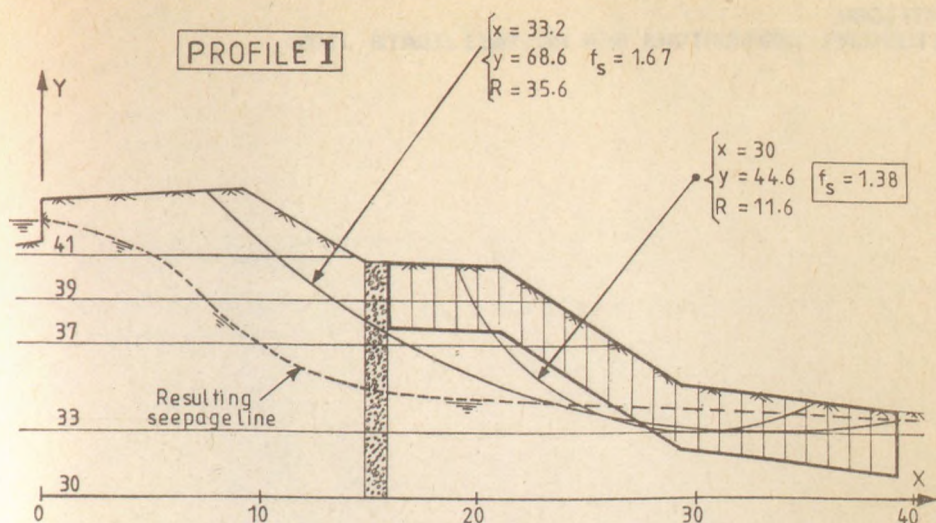


Fig. 6 : Critical slip-circles after soil improvement

CONCLUSIONS

Canal dike slope stability problems, as a result of unfavorable stage construction, can be effectively solved by means of a combination of soil nailing and stone columns.

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SECTION 4
SOIL STABILIZATION AND ENVIRONMENT PROTECTION

Stress-Deformation characteristics of a soil after freezing and thawing.

By

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Tokyo, JAPAN

(Abstract)

The phenomena of frost heave and loss of strength on thawing produce serious road and other earthwork engineering damages at many places of Tateshina plateau in Nagano prefecture.

The reduction in shearing strength causes a decrease, for example, of the bearing capacity of flexible and rigid pavement, a decrease of the slope stability for embankments or cuts.

The mechanical properties of soils, such as shear strength and deformation characteristics, are affected by freezing and subsequent thawing of the soil.

This paper discusses the results of experimental investigation of a cohesive soil (Tateshina loam) that has been subjected to freezing and thawing.

The soils used for this investigation were undisturbed samples. Consolidated undrained triaxial tests were conducted on the thawed soil at one and several freeze and thaw cycles.

The results are summarized in several graphs showing stress-deformation characteristics.

Stress-Deformation characteristics of a soil after freezing and thawing.

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The results are summarized in several graphs showing stress-deformation characteristics.

I. Introduction

Economic growth in recent years has brought about the development of the cold and mountainous areas, construction of roads passing through high and mountainous districts, and further, a remarkable increase in road sections which are completely cleared of snow during winter in the snowy areas. As a result, the range of the area subject to frost heave is increasing nationwide. In addition to these natural frost damages caused by invasion of freezing line into the ground due to drop in the temperature, newly arising is a problem of artificial frost damage relating to construction of underground tanks for storing liquefied gas and the freezing method.

Among frost damages, many studies have been made in the past on the frost heave phenomena, moisture move at the time of freezing or physical and mechanical properties of frozen soils. However, relatively few studies have been made on the study of mechanical properties of soils after thawing which is closely linked to a decline in bearing capacity.

The decline in strength of the soil by freezing and thawing was considered in the past to be mainly attributable to moisture supply from below and subsequent ice formation. However, the authors have made it clear that a decline in the

strength of the soil can occur due to freezing and thawing even in the condition of Closed-System where there is no moisture supply from below.

Consolidated undrained triaxial tests (CU Test) were conducted on undisturbed cohesive soil samples in both unfrozen and frozen/thawed states, and the strength and deformation characteristics in both cases were compared and studied in order to determine the mechanical properties of the soil after freezing/thawing. An outline of the results is reported in this paper.

II. The Soil for the Experiment

The Tateishina Plateau in Nagano Prefecture is covered with volcanic ash cohesive soil with a high moisture content. In this area, various freezing damages have occurred including cracks on the surfaces of pavement due to uneven frost heave.



Photo 1. Damage of flexible pavement due to frost heave



Photo 2. Uplift of manhole due to adfreezing force.



Photo. 3 Failure of the cut slope due to frost damage.

As to samples used for the experiment, undisturbed samples were collected with a thin wall sampler from a portion considered utterly absent from undergoing freezing/thawing by digging the ground below freezing depth at a site 1,300 m above sea level in the Tateshina Plateau (Hiromi, Chino City).

The lowest temperature in the area is around -20°C , and the freezing depth as measured with a methylene blue freezing depth indicator is approximately 70 cm.

Fig. 1 shows the result of particle size analysis of this volcanic ash cohesive soils, where the particle size range of soils subject to freezing damages is indicated at the same time. The soil property values of samples as determined by the soil testing method based on JIS and JSF are as indicated in Table 1. This indicates that this samples is cohesive soil with a high silt content and a high water content implying that the soil is vulnerable to freezing damages.

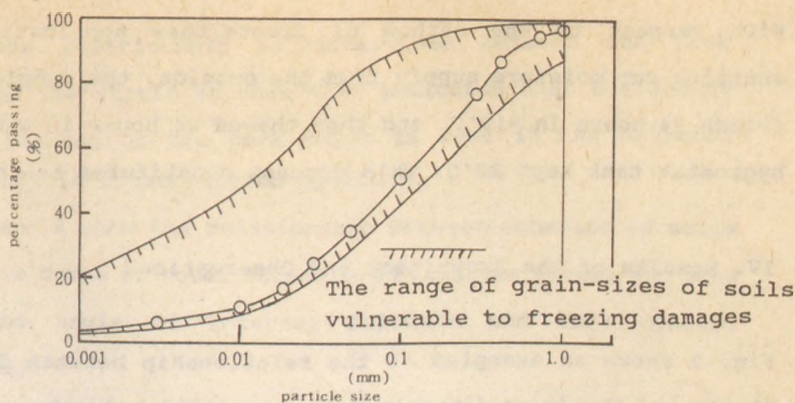


Fig. 1 Grain-size accumulation curve of the sample.

Table .1 Geotechnical properties.

G_s	W_L	W_p	W_n	γ_t	e	k
2.66	80.7 %	56.7 %	90 ~ 100 %	1.3 ~ 1.4 g/cm ³	2.5 ~ 3.0	cm/sec 4.4×10^{-5}

III. Outline of Experiment

Because the samples at the time of collection were unsaturated water was injected for two weeks to heighten the degree of saturation. After repeating 1, 3, 5, and 7 freeze and thaw cycles on these samples, test samples were prepared.

These test samples were set in the triaxial compression tester, and sheared undrained after isotropically consolidated. After consolidated, back-pressure was applied to come up with the average value of B-coef exceeding 0.95. Further, the identical shearing test was conducted on unfrozen samples for the sake of comparison.

With respect to the method of freeze/thaw application, by shutting out moisture supply from the outside, the samples were frozen 24 hours in -18°C , and then thawed 24 hours in a thermo-hygrostat tank kept 20°C . This process constitutes one cycle.

IV. Results of the Experiment and Observations

Fig. 2 shows an examples of the relationship between deviator stress and strain as determined in a consolidated undrained soil test conducted on the test samples which underwent freezing/thawing and those that did not.

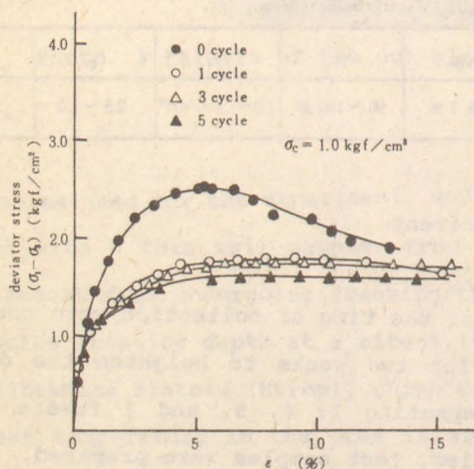


Fig.2 Relationship between deviator stress and strain.

It brings to light that while the unfrozen specimen indicates brittleness deformation with the peak value, that which underwent freeze/thaw shows plasticity deformation with its peak value equivocal.

Consequently, the stress and strain curve of consolidated cohesive soil frozen/thawed is similar to that of normal consolidated clay and repeated cohesive soil.

Further, the relationship as determined between the peak strength and the strain at that time indicates that the amount of strain leading to the peak value is less in the unfrozen specimen than in frozen/thawed specimen.

Figures 3 and 4 show the relationship between cohesion of soils indicated in terms of total stress and effective stress and thaw cycles, and angle of internal friction and thaw cycles, respectively.

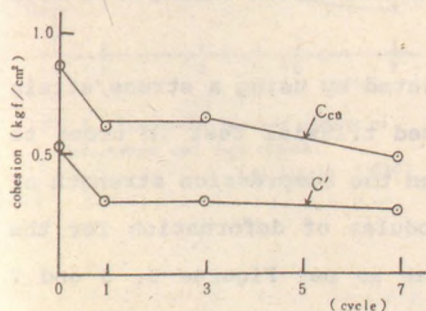


Fig.3 Relationship between cohesion and thaw cycles.

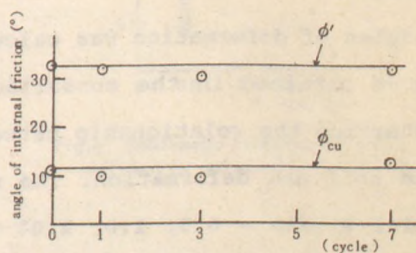


Fig.4 Relationship between angle of internal friction and thaw cycles.

Both figures show that: the decrease in cohesion by freeze/thaw is conspicuously indicated in the first cycle with its value scarcely changing along with the increase in the number of cycles thereafter: the angle of internal friction is nearly uniform without being influenced by freeze/thaw.

This decline in the strength of soil after freeze/thaw is caused by the change in structure of soil brought about by ice crystal formation in the closed formula and the growth of ice crystal. The following can be considered for the changes in the structure:

- 1) Occurrence of minute cracks in the soil
- 2) Change in the arrangement of soil grains
- 3) Change in the thickness of adsorbed water around soil particles

Ice crystals grow as they absorb pore water and adsorbed water to form double layers around soil grains. Subsequently, the adsorbed water film around soil grains becomes thin to cause the decline in the electric potential among soil grains, thereby weakening cohesion among grains. Further, the ice crystals cause minute cracks in the soil and bring about a change in the arrangement of soil grains. Moreover, it can be considered that because these structural changes occurring during freezing persist even after thawing, the decline in the strength of soil occurs.

Modules of deformation was calculated by using a stress/strain curve obtained in the consolidated triaxial test in order to determine the relationship between the compression strength of the soil and deformation. The modules of deformation for the axial strain = 0.5, 1.0, 2.0% was as per Figures 5, 6 and 7 respectively.

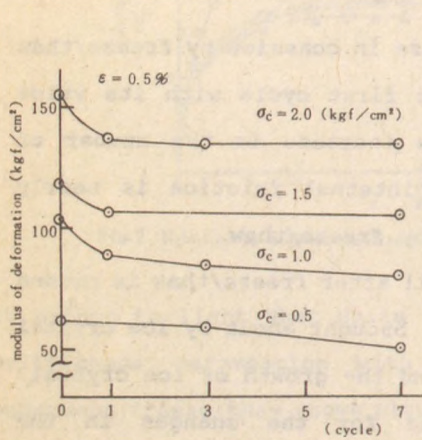


Fig.5 Relationship between modulus of deformation and thaw cycles.
(at strain 0.5%)

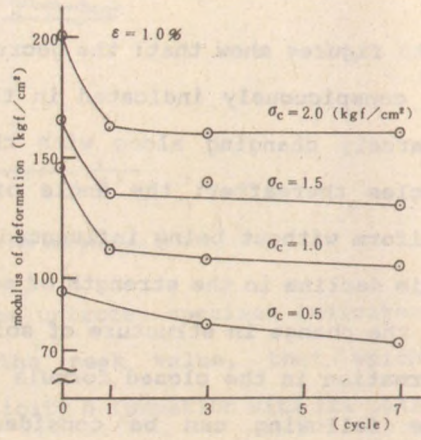


Fig.6 Relationship between modulus of deformation and thaw cycles.
(at strain 1.0%)

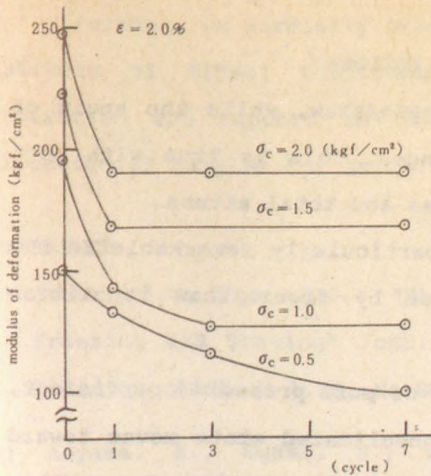


Fig. 7 Relationship between modulus of deformation and thaw cycles. (at strain 2.0%)

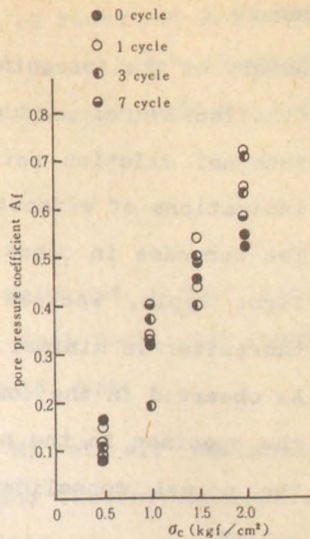


Fig. 8 Relationship between pore pressure coefficient A_f and consolidated pressure σ_c

The modulus of deformation of the frozen/thawed soil lessens in the value as compared with unfrozen soil, where the value decreases as the number of freeze/thaw cycles increases.

It can be observed that the degree of decrease in the modulus of deformation is notable observed in the first cycle. Plotting by respective consolidated pressures by finding the maximum deviator stress in respective specimens, and pore pressure coefficient A at the time of $(\sigma_1 - \sigma_3)_{\max}$ is shown in Fig. 8.

The A value decrease along with the rise in consolidated pressure, where the value is larger in frozen/thawed soil than in unfrozen soil though minimal fluctuation exists. This can be phenomenally perceived as a transition of the highly consolidated soil toward normal consolidated state by undergoing the freeze/thaw experience.

V. Summary

A summary of the foregoing is as follows:

- 1) Cohesion decreases due to freeze/thaw, while the angle of internal friction barely changes. This is true with both indications of effective stress and total stress.
- 2) The decrease in cohesion is particularly remarkable in the first cycle, whereas decrease by freeze/thaw repetition thereafter is minimal.
- 3) As observed in the comparison of pore pressure coefficient, the specimen in the highly consolidated state moves toward the normal consolidated state by undergoing freeze/thaw action.
- 4) The modulus of deformation is smaller in the frozen/thawed soil than in the unfrozen soil in the initial stage of strain.
- 5) The stress/strain curve illustrates that the unfrozen soil shows brittleness deformation with the peak value whereas the frozen/thawed soil indicates plasticity deformation with ambiguous peak value.

In conclusion, it is necessary in design and construction to fully consider freeze damages, i.e. the future deterioration in the strength, or to take measures to counter a decline in strength. When dealing with the soil vulnerable to freeze/thaw effect that has no experience of freeze/thaw in the past.

ACKNOWLEDGEMENT

We would like to express our hearty appreciation to Professor Shoji Ogawa of Nagaoka University of Technology, who has rendered precious advice and assistance in continuing this study.

Further, we cordially thank those of Technical Research Institute of Mitsui Construction Co. for their constant cooperation and support in the freeze damage research and collection of samples for tests.

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ERKUNDUNG, IN-SITU SANIERUNG UND ERFOLGSKONTROLLE EINER
BODENVERUNREINIGUNG, VERURSACHT DURCH AROMATISCHE KOHLEN-
WASSERSTOFFE

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1. AUSGANGSSITUATION

Auf einem innerstädtischen Grundstück wurden ab Januar 1988 zehn viergeschossige Wohnhäuser mit zwei Tiefgaragen in zwei Bauabschnitten errichtet. Nach Fertigstellung der Rohbauarbeiten des ersten Bauabschnittes wurden bei der Räumung des zweiten Baufeldes erdverlegte Tanks freigelegt, deren anschließende Leitungen Roststellen aufwiesen. Die Tanks selbst waren unversehrt, jedoch teilweise noch mit Lösemitteln gefüllt. Auffälliger Fremdgeruch oder eine Verfärbung des Bodens um die Tanks wurde nicht wahrgenommen.

Im Rahmen der Wasseraufsicht wurde der Nachweis gefordert, daß keine Verunreinigung des Bodens und damit eine Gefährdung des Grundwassers eingetreten war.

Die weiteren Erkundungsarbeiten mußten kurzfristig durchgeführt werden, da die Fundamentierungsarbeiten für den zweiten Bauabschnitt unmittelbar bevorstanden.

2. VORERKUNDUNGEN

Aus alten Plänen ergab sich eine Vornutzung des Geländes durch eine Druckerei. Die alten Bestandspläne zeigten mehrere Tanks zur Lagerung bzw. Rückgewinnung von Heizöl und aromatischen Lösemitteln.

Eine Analyse der vorgefundenen Tankfüllung ergab als Hauptsubstanzen Toluol und Xylol.

Erst eine zweite, 9 m tiefe Bohrung ergab unterhalb einer etwa 40 cm starken Schluffschicht in 3 m Tiefe unter Tankunterkante einen starken Fremdgeruch und bei der anschließenden Analyse einen hohen Toluolgehalt im Boden.

Auf der Grundlage von:

- zwanzig, etwa 3 m tiefen Bodenluftpegeln,
 - zehn, bis 20 m tiefen Kernbohrungen und
 - vier, 33 m tiefen Grundwassermeßstellen
- erfolgte die genaue Eingrenzung des Schadensbereichs.

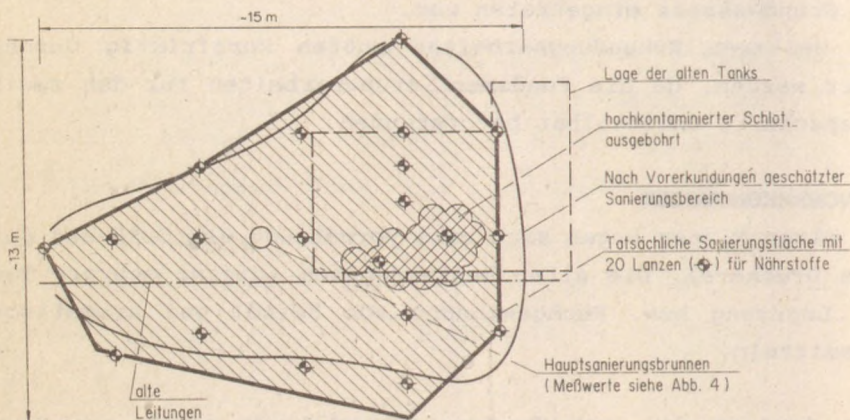
3. SCHADENSAUSMASS

Die vorerkundete Schadensfläche deckt sich im wesentlichen mit dem im Zuge der Sanierungsarbeiten endgültig festgestellten Schadensbereich und mißt etwa 100 m^2 einschl. eines hochkontaminierten "Schlotes", in welchem die Schadstoffe offensichtlich in den Boden eingedrungen sind und sich ausgebreitet haben. Unzulässig hohe Schadstoffkonzentrationen lagen in einer Tiefe von etwa 5 bis 18 m unter ursprünglichem Gelände vor und sind im wesentlichen oberhalb des Grundwassers durch Sperrschichten zurückgehalten worden.

Das Schadensvolumen beträgt 875 m^3 . Die mittlere Schadstoffkonzentration wurde zu etwa 2.500 mg/kg TS ermittelt. Die maximale Schadstoffkonzentration betrug 35.000 mg/kg TS .

Der Hauptbestandteil ist zu etwa 90 % Toluol.

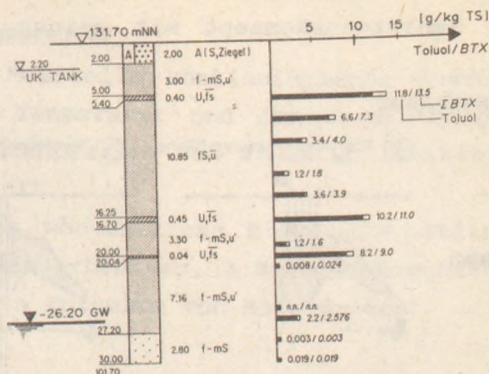
Abb. 1 - Lage von Tanks, Schadensfläche und Nährstofflanzen



4. BODENVERHÄLTNISSE/GRUNDWASSERSTAND

Im Schadensbereich steht überwiegend schwach schluffiger Fein- und Mittelsand an, in den in etwa 5 m bzw. 16,5 m Tiefe unter ursprünglichem Gelände zwei, etwa 0,4 m starke, schluffige Hauptsperrschichten eingelagert sind. Diese Schichten wurden in allen Erkundungsbohrungen angetroffen und verlaufen überwiegend horizontal.

Abb. 2 - Boden- und Schadstoffgehalt-Profil



Der Flurstand von ursprünglichem Gelände zum Grundwasser betrug etwa 26 m.

5. RAHMENBEDINGUNGEN

Bei der endgültigen Auswahl und Planung des Sanierungsverfahrens mußten folgende besonderen Gesichtspunkte berücksichtigt werden:

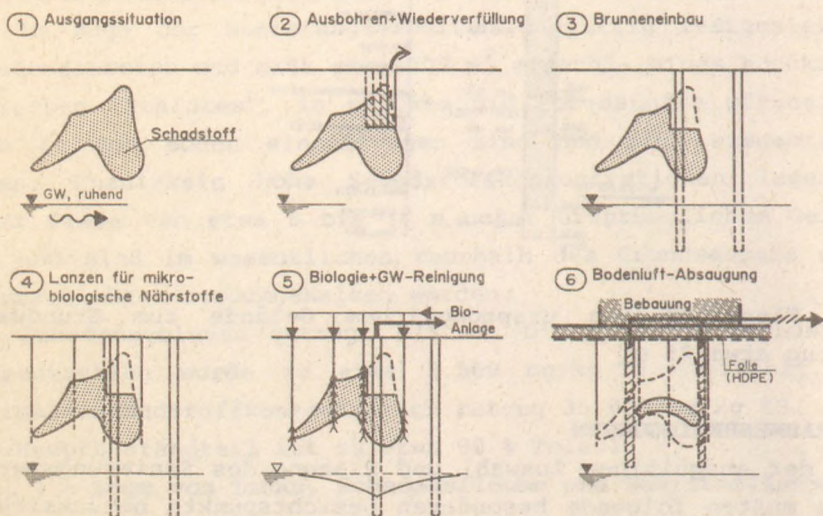
- Besonders schutzbedürftige Anwohner (Kindergarten, Wohnbauten und wichtiger Dienstleistungsbetrieb) in unmittelbar angrenzender Nachbarschaft
- Hohe Sicherheitsanforderungen für das Baustellenpersonal bei nicht auszuschließender Explosionsgefahr infolge hochgradiger Toluolverunreinigung im Schadenszentrum
- Fehlende Deponiekapazität im betroffenen Landkreis
- Extrem kurze, verfügbare Bauzeit für die Sanierungsmaßnahmen wegen der Absicht, die Schadensfläche noch während des biologischen Schadstoffabbaus im Boden zu überbauen

- Nutzung der Wohnhäuser nach Beseitigung der Hauptverunreinigung, jedoch noch vor endgültigem Abschluß der Sanierung. Damit war eine wesentliche Nachbesserung bzw. Änderung des Sanierungssystems im Zuge des Sanierungsverlaufs ausgeschlossen, so daß alle notwendigen Sanierungselemente vor Beginn der Überbauung der Schadensfläche installiert sein mußten.

6. SANIERUNGSKONZEPT UND DURCHFÜHRUNG

Abb. 3 zeigt das Sanierungskonzept mit seinen wesentlichen Bauphasen.

Abb. 3 - Sanierungsablauf



- Der hochkontaminierte Schlot wurde durch 14 Bohrungen, $\varnothing$ 900 mm, auf eine Tiefe von 16 m ausgebohrt und als Sonderabfall verbrannt.
- Anschließend wurden 20 Lanzen, $\varnothing$ 50 mm, bis maximal 20 m Tiefe unter Gelände gesetzt, um durch sie eine mit flüssigem Sauerstoff aufbereitete Nährstofflösung in den Boden zu infiltrieren.
- Anlegen eines Flächenfilters aus Quarzkies und Einbau von Absaugrohren in die Drainage oberhalb der Schadensfläche.

- Gasdichter Abschluß des Schadensbereichs zum Bauwerk durch eine 2 mm starke, verschweißte HDPE-Folie.
- Abpumpen von Grundwasser im Schadensherd, Aufbereitung in der biologischen Reinigungsanlage und erneute Infiltration mit Zugabe von Nährstoffen sowie Sauerstoff.
- Vorhalten von vier Gasbrunnen für eine eventuelle, spätere Bodenluftabsaugung.
- Einbau von Kontrollelementen im Boden und Grundwasser.

7. ERFOLGSKONTROLLEN

An je einer Meßstelle kontinuierliche Messung des Aromaten-Gehalts, der Temperatur und des Sauerstoffgehalts im Wasser sowie der Konzentration von Aromaten, Methan und Kohlendioxyd in der Bodenluft.

Darüber hinaus wöchentliche bis zweimonatliche Aromaten-Messungen in 6 Wasserbrunnen, 3 Bodenluftbrunnen und 4 Kontrollpunkten für die Entnahme von Bodenproben.

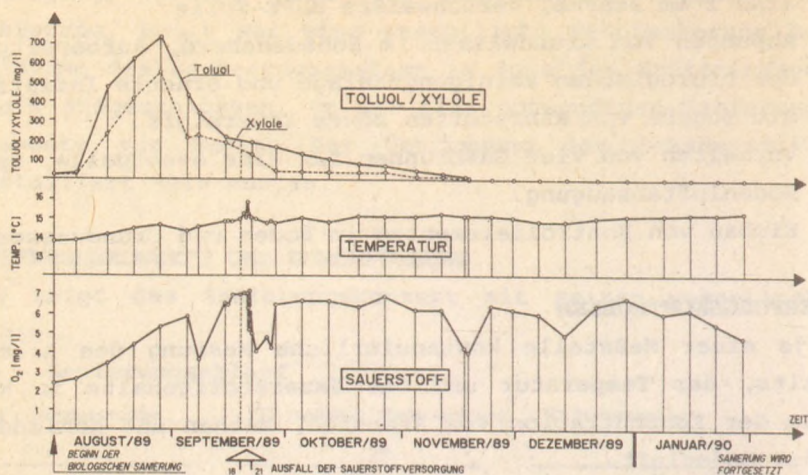
8. ERGEBNISSE

Die Arbeiten zum Ausbohren des hochkontaminierten Bodens begannen im März 1989. Die biologische Reinigungsanlage wurde im August 1989 in Betrieb genommen. Etwa 6 Monate nach Beginn der biologischen Sanierung sind etwa 90 % des ursprünglichen Schadstoffgehalts abgebaut.

Dabei erfolgt der Abbau des Toluols schneller als der der Xylole. Die Sanierung wird auch nach Bezug der Wohngebäude im Sommer 1990 fortgesetzt. Es wird damit gerechnet, daß die Sanierungsanlage im Jahr 1992 abgeschaltet werden kann.

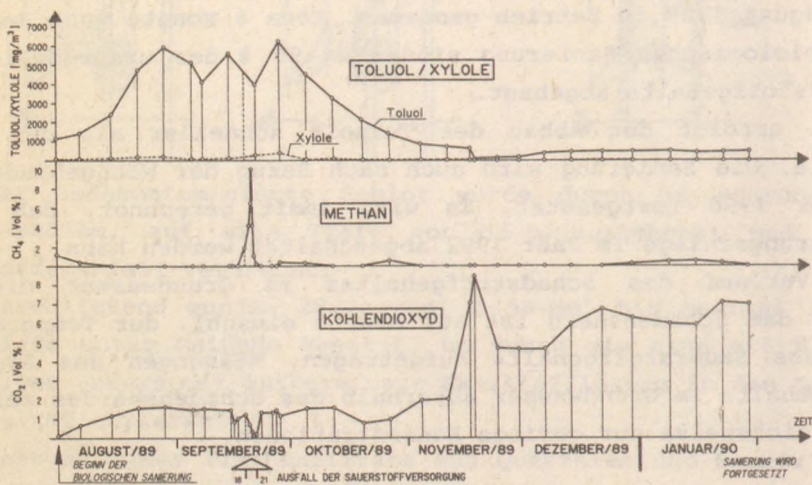
Der Verlauf des Schadstoffgehaltes im Grundwasser direkt unter dem Schadensherd ist auf Abb. 4 einschl. der Temperatur und des Sauerstoffgehalts aufgetragen. Messungen des Aromaten-Gehalts im Grundwasser außerhalb des Schadensherdes zeigen vergleichsweise nur geringe Konzentrationen.

Abb. 4 - Verlauf der Kontrollparameter im Wasser



Auf Abb. 5 ist der Aromaten-, Methan- und Kohlendioxydgehalt in der Bodenluft für den gleichen Zeitraum dargestellt. Messungen im Boden haben an den Kontrollpunkten einen Abbau des Schadstoffgehalts von etwa 2.500 mg/kg TS auf unter 200 mg/kg TS ergeben.

Abb. 5 - Verlauf der Kontrollparameter in der Bodenluft



Die Ergebnisse belegen einerseits den guten Erfolg der eingeleiteten Maßnahmen, jedoch auch die Empfindlichkeit des Sanierungssystems gegen Betriebsstörungen und die Notwendigkeit umfassender Überwachung.

Minimization of harmful chemical reagents discharge for grouting of soil bases.

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The intensive increase of industrial construction and civil engineering in areas covered by loess collapsible soils actualizes problem of technical melioration of these soil bases with purpose of elimination of collapsible subsidence followed by the structure settlements of allowless magnitude and ununiformity. Effective and widely used method of the melioration is a silication of loess soils.

Besides the great advantages this method also suffers from the essential shortcomings-high cost and undesirable effect on organic soil matter.

One of ways of overcoming of these shortcomings lays in maximum possible decrease of dimensions of grouted massif and the volume of applied chemical reagents.

The unit (per 1 m³ of soil) discharge of chemical reagent within the grouted massif limits is usually accepted to be constant. The dimensions of grouted massif and the unit discharge are designed by selection, i.e. for the given scheme it is possible to select the geometrical parameters and unit discharge of grouting matter, proceeding from the designing requirements.

The traditional approach to design of chemically improved bases, viz., "the wild cat method" leads to excessive discharge of sodium silicate, that results in the increase of the soil improvement cost, and the aggravation of ecological state in the area of construction. The existent technology of collapsible soils silication allows to obtain the grouted massives of vari-

ous form and horizontal and vertical dimensions varying the distribution, quantity and depth of injection holes. It is possible to create deliberately the ununiform base stabilization varying the discharge of natrium silicate in different points of massif. The most rational design of grouting massif that satisfies to the requirements of structure base design and simultaneously ensures the minimum discharge of grouting matter may be carried out by variation of geometrical parameters of grouted massif and unit discharge of grouting matter at its different points.

The solution of this problem from the point of view of multiversion designing would demand the consideration and comparison of large number of design versions with the purpose of selection of the best one. Such solution is hard-laboured even by means of the computer, and the chosen version may be proved to be far from the most rational.

The most prospective way in solution of such problems lays in usage of applied theory of optimal designing. The purpose of optimization is the determination of the best solution among the variety of the possible ones in accordance with some criteria of efficiency and quality. The criterion of optimum (the aim function) is the single index of construction quality that must be optimized during the designing. The optimum is reached when the aim function is minimized

$$z = z(\bar{x}, \bar{u}) \longrightarrow \min \quad (1)$$

where $\bar{x}$ -designing parameters, $\bar{u}$ -functions of spatial coordinates. Besides the aim function some limitations are usually present in problems of optimal designing.

The problem of optimal design of chemically grouted base under the strip foundation is discussed in this report. The criterion of optimum is the total discharge of natrium silicate. This discharge has to be minimum from the economical and ecological considerations

$$Q = \int q(x, y) d\Omega \longrightarrow \min$$

where q - unit discharge of grouting matter per unit of soil volume in the point of massif with coordinates (x, y) ; Q -total

discharge of sodium silicate.

In the case of vertically loaded loess bases the excessive settlements (especially differential ones) of structures under soil wetting are the most dangerous phenomena. Thus the parameters of grouted massif have to satisfy the following condition: maximum settlement of footing is less than allowable one $S_{\max} < S_{\text{allow}}$ (2)

where $S_{\max}$ is maximum predicted settlement; S_{allow} - allowable settlement of structure.

The model of linearly deformable solid is used as mechanical model of base material. The base grouted with sodium silicate suspension is simulated by nonuniform elastic halfspace with parts, having different characteristics of deformability - Young modulus E and Poisson ratio ν . Solution of plane problem of theory elasticity is performed by means of finite element method. Collapsible change of loess deformability under wetting is simulated by means of decrease of Young modulus E and increase of Poisson ratio in the wetted zone.

In conformity with experimental data linear form of relationship Young modulus E_c of grouted soil - unit discharge q of sodium silicate was accepted

$$E_c = E_0 + Aq \quad (3)$$

where E_0 is the deformation modulus of ungrouted soil at the natural moisture; A is coefficient of proportionality.

Thus we perform the search of optimal distribution of q , that minimizes the aim function (1) and simultaneously satisfies the condition (2), by varying the values of q at very point of soil base with corresponding varying the soil modulus values.

The search of the aim function minimum is performed by the "deformed polyhedron" method of Nelder-Mead (1964) and the "penalty functions" method. The deformed polyhedron method may be used for rather arbitrary continuous aim functions. It is approved in many applied investigations. This method in its classic version is developed for unconditional optimization, i.e., for the case when the independent variables are not limited. So the aim function must be modified with the aim of solu-

tion of conditional optimization problem by the Nelder-Mead method.

The penalty functions method is effective for this purpose. The basic idea of this method (Himmelblau, 1972) consists in transformation of minimization problem with limitations expressed as equalities

$h_j(\bar{x}) = 0, j = 1, \dots, m$
and as inequalities

$g_j(\bar{x}) \geq 0, j = m+1, \dots, p$
to the problem of search of function minimum without limitations

$$z = f(\bar{x}) + R \left\{ \sum_{j=1}^m |h_j(\bar{x})| + \sum_{j=m+1}^p b_j g_j(\bar{x}) \right\},$$

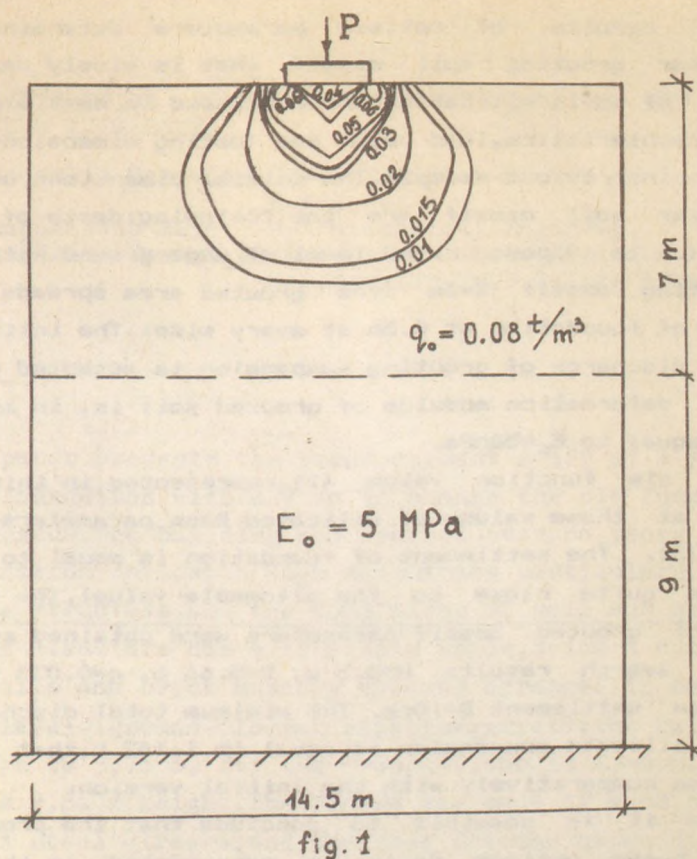
where R -penalty coefficient,

$$b_j = \begin{cases} 0, & \text{if } g_j \geq 0 \\ -1, & \text{if } g_j < 0 \end{cases}$$

The search of optimum solution begins from determination of initial values (initial approximations) for parameters to be optimized for grouted soil massif with account of designing experience.

The designing of grouted base under the strip foundation of 2m width (fig.1) is discussed in this report as an example. The base consists of soils with deformation modulus $E_0 = 5 \text{ MPa}$ at natural moisture and $E_0 = 2 \text{ MPa}$ at water saturated state. The mean pressure under the base of footing is equal to 0.2 MPa. The uniform grouting of base rectangular area of dimensions of 14.5 * 7.0 (m) is accepted as an initial approximation, that corresponds to the existing designing practice. The initial unit discharge value $q = 0.08 \text{ t/m}^3$ of grouting suspension is accepted, that provides in accordance to (3) the deformation modulus value of grouted soil $E_c = 47.4 \text{ MPa}$, $S_{\text{allow}} = 0.1 \text{ m}$. The total discharge of grouting suspension is equal to 8.21 t, the foundation settlement is $S = 6.98 \text{ cm}$.

The search of the aim function minimum may be performed in



accordance to suggested algorithm using these values of the grouted base characteristics as the initial approximation. The analysis of footing settlement is performed by means of finite elements method at every step of search, at every new set of varying parameters. The calculated settlement is compared with the allowable one. The optimum distribution of unit discharge of sodium silicate suspension in grouted massif is received at the end of search; the unit discharge contours are drawn for grouted massif as the graphical representation of solution. Total discharge of sodium silicate suspension is equal to $Q=0.8t$, that is about an order of magnitude less than for initial version of grouting, and the settlement of the footing on this ununiformly grouted base is equal to $S=10 \text{ cm}$.

The results of optimum parameters determination of rectangular grouting soil massif that is widely used in the practice of soils silication, are discussed in next example. The soil characteristics, load value and footing dimensions are the same as in previous example. The initial dimensions of grouted rectangular soil massif are the following: depth of grouting $H=8.28\text{m}$ (up to supposed rised level of underground water), width of grouting massif $B=3\text{m}$ (the grouted area spreads over the contours of foundation at 0.5m at every side). The initial value of unit discharge of grouting suspension is accepted $q=0.1\text{t/m}^3$ and the deformation modulus of grouted soil is, in accordance to (3), equal to $E_c=58\text{MPa}$.

The aim function value (1), represented in this case as $Q=B \cdot H \cdot q$ at these values of silicated base parameters, is equal to $Q=1.75\text{t}$. The settlement of foundation is equal to $S=9.2\text{ cm}$ (that is quite close to the allowable value). The following values of grouted massif parameters were obtained as optimum solution search result: $H=5.5\text{ m}$, $B=5.66\text{ m}$, $q=0.035\text{ t/m}^3$, the foundation settlement $S=10\text{cm}$. The minimum total discharge Q of natrium silicate suspension is equal to 1.142 t , that is at 1.5 times less comparatively with the initial version.

Thus it is possible to conclude that the procedure of grouted bases optimum designing represented in this report allows to reduce essentially the total discharge of natrium silicate suspension and cut the harmful effect on organic matter of soil in the area of construction.

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Title: THE STRUTTING FOUNDATION CONCEPTION OF THE NEW HISTORY MUSEUM IN GALATZ

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Text: The paper presents the reinforcement draft of a public building foundation with a view to change the old function in History Museum. The building was erected over 80 years ago, in a weak foundation soil, with high seismicity particulars.

A. The structure and the conditions of soil and seismicity.

The structure has a trapezoid shape, being a combined character (walls and brick masonry columns arranged in both directions), 4 levels (ground-floor + 2 storeys + garret); the height of the storeys is 3,70 m, for the ground-floor is 4,00 m. and the garret has 2,80 m height. The floors are made of wood, partly combined with steel girders, and the roof framing of wood with brick sheet roofing. The foundation, made in brick masonry, have 2,20 m depth, measured from the natural soil level.

The foundation soil consists of loess of ab 8m depth, located under a non-homogeneous filling of ab 2,00 m. Under the loess layer, a series of clay like and loamy dusts have been intercepted, with depths from 1,50-4,00m. On the loess level, saturation wetting level from 6,20m. depth to the surface has been found, the level being of ascending character. The lithological typical column is shown in Fig. 1 and the geotechnical particulars are given in the Table No. 1.

The area is characterized by its VIII degree on MSK scale.

The active unit loads on foundation continuous footing vary from 170-220 KPa. By investigating the structure state, after removing the plaster, unpierced cracks having 0,5-10 mm width have been found in structural and non-structural items; but they

LYTHOLOGICAL SUCCESSION OF THE LAYERS

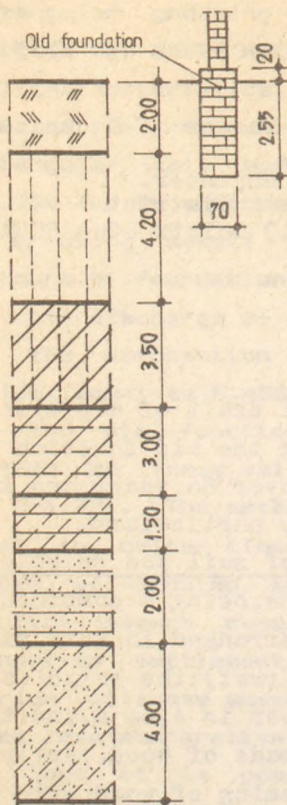


FIG.1

GEOTECHNICAL CHARACTERISTICS

Table Nr.1

Stratum	Granulometric composition			W%	Structure			Shearing strength	
	C	S	D		$\lambda_{\text{KNm}^{-3}}$	n%	$\epsilon\%$	C_{KPa}	ϕ°
Loess	14	11	75	13.5	1.57	48	0.39	0.30	21
Saturated loess	15	9	76	235	-	-	-	-	-
Clayey dust	42	3	55	28.0	1.67	47	0.38	0.35	17
Dusty clay	73	8	19	26.0	1.70	43	0.35	0.38	13
Sandy clay	73	17	10	22.0	1.81	44	0.36	0.39	11
Argilla-ceous sand	9	91	-	21.0	-	-	-	-	-

did not highly affect the strength and the stability of the structure.

2. Analysis of strengthening problems.

The strutting solution has been combined with the ground-floor modifications, imposed by the new function and this is shown in Fig.2, having in view the increase of the safety and the stability, on the account of the real soil conditions and seismicity. One of the main problems to be settled was the decrease of the bearing capacity of the soil foundation following the presence of the phreatic level (of ascending nature) inside the active zone. In this case, the limit resistance of the loess layer as against the increase of the wetness will be (Dianu, V., Coşovliu, O., 1984):

$$\sigma_{lim;m} = q_{cr} = \frac{\pi [\gamma(W_{cr})D + C(W_{cr}) \operatorname{ctg} \phi(W_{cr})]}{\operatorname{ctg} \phi(W_{cr}) - \pi/2 + \phi(W_{cr})} + \gamma(W_{cr})D \quad (1)$$

where $C(W_{cr})$, $\phi(W_{cr})$ are the resistance parameters of the loess at critical wetness, $\gamma(W_{cr})$ -volumetric weight of the soil, D - foundation depth. The analysis of the real properties of the lo- cation soil, carried out with this relationship has made evident the behaviour as shown in fig.3.

PLAN OF THE GROUND FLOOR

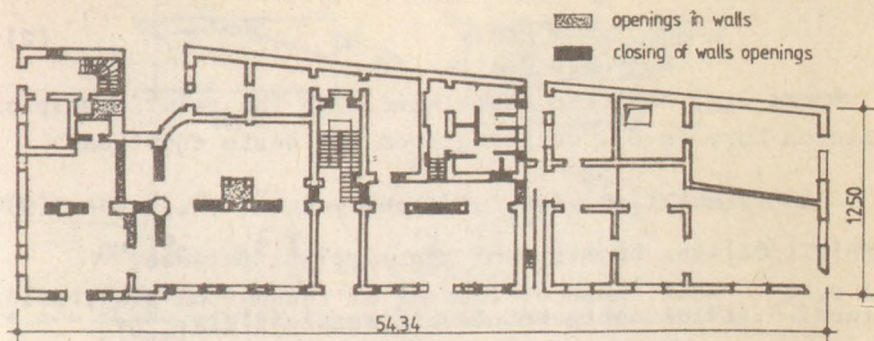


FIG.2

On the account of this situation, the adopted solution is based upon the calculation of the interaction soil-structure, as presented in ref. (Coşovliu, 1986). The building was considered

DECREASE OF THE LIMIT SOIL BEARING CAPACITY OVER THE WATER CONTENT

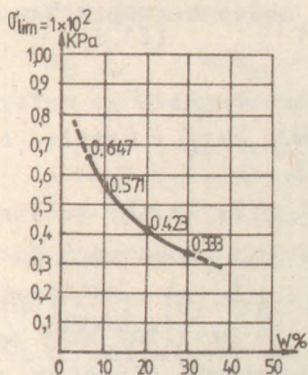


FIG.3

THE VARIATION IN LONGITUDINAL OF THE BUILDING STIFFNESS

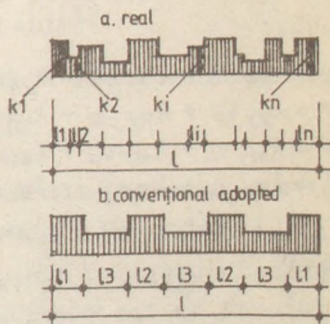


FIG.4

to be modelled by an equivalent beam with variable rigidity, as in the Fig.4, where the rigidities $K_1, K_2 \dots K_3$, of various portions have been computed according to known relationships, assuming-at

limit—a reduction of their values to two average values K_1, K_2 , which have been reduced by 40% for calculation purpose. Two deformation schemes with non-uniform character have been considered for the soil, as shown in Fig. 5. For the calculation, the scheme of the Fig. 5a was used, because it led practically to maximum overall strains (M, T, M_t). The values of the settlements S_1, S_2 and $S_{\max}$ have been calculated with the relationship (Coşovliu, 1986):

$$\begin{aligned} S_1 &= 0,6 S_{\max}. \\ S_2 &= 0,8 S_{\max}. \end{aligned} \quad (2)$$

where $S_{\max}$ has been calculated with the relationship of the Romanian Norm P7-87. Outgoing from the basic equation:

$$[EI]y^{IV} - K \frac{[EI]}{[GA]} y'' + cy - q = 0 \quad (3)$$

with $[EI], [GA]$ —the bending and shearing stiffnesses, and c, q —stiffness coefficient of foundation soil, building load per unit length respectively.

One can determine the rigidity characteristics of the soil and structure, which should be introduced into the eq(3). This equation could be solved by the method of the parameters at the origin. The beam-soil system is divided, with the accepted deformation scheme, into a number of sections—six, in our case then write the continuity equations of the four parameters (y, φ, M, T) in each sections. A system of 24 equations with 24 unknown terms is available:

$$\begin{aligned} y &= k_{yy}y_0 + k_{y\varphi}\varphi_0 + k_{yM}M_0 + k_{yT}T_0 + F_y \\ \varphi &= k_{\varphi y}y_0 + k_{\varphi\varphi}\varphi_0 + k_{\varphi M}M_0 + k_{\varphi T}T_0 + F_\varphi \\ M &= k_{My}y_0 + k_{M\varphi}\varphi_0 + k_{MM}M_0 + k_{MT}T_0 + F_M \\ T &= k_{Ty}y_0 + k_{T\varphi}\varphi_0 + k_{TM}M_0 + k_{TT}T_0 + F_T \end{aligned} \quad (4)$$

The solution of this system gives the considered parameter values for each point of the section. The values of the twisting moments induced by the non-uniform settling have determined by an additional calculation on transverse direction, outgoing from the basic equation:

$$\Theta'' + \frac{m_t}{[GI_t]} = 0 \quad (5)$$

where Θ is the rotation on transverse direction, m_t —the free twisting moment in structure and $[GI_t]$ —the torsion stiffness of the structure, the signification being those given in

fig.6. For calculation, the system should be divided into the same number of section (six) on longitudinal direction. Expressing v_1, v_2

SOIL DEFORMATION ASSUMPTIONS

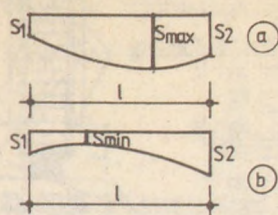


FIG. 5

SOIL CROSS SPECIFIC
FEATURES OF RIGIDITY

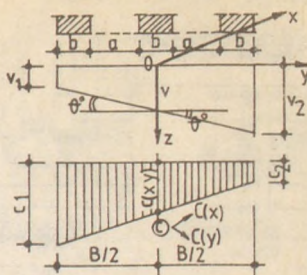


FIG. 6

versus v and C_1, C_2 versus, δ we obtain-by transformations -:

$$\Theta'' - A^2 \Theta + A^2 \frac{\delta}{c} (v_1 - \frac{v_j - v_1}{l_{ij}} x) = 0 \quad (5')$$

where $A^2 = \frac{bB^2c}{2[GIt]}$, with the significance given in Fig.6. Knowing the values of v , in several characteristical sections of the beam, and assuming they vary linearly on transversal direction, as shown in fig.6, the differential equation (5'') is transformed into:

$$\Theta'' - \beta^2 \Theta + \beta^2 \frac{\delta}{c} (v_1 + \frac{v_j - v_1}{l_{ij}} x) = 0 \quad (5'')$$

The values of the twisting moments can be obtained writing the equations for each section as under:

$$m_t \frac{l_p(1)}{l_{ij}} = m_t \frac{l_{(p+1)}(1)}{l_{ij}} \quad (6)$$

so that a system of 7 equations with 7 unknown factors results, the solution being:

$$\Theta_i = \frac{\Delta \Theta_i}{\Delta_i} \quad (7)$$

The values of the twisting moments for each section may be found with the recurrence relationship as under:

$$m_t = -[GIt] \left\{ \frac{A}{ShAl_{ij}} \left[\Theta_j - \frac{\delta}{c} v_j - \left(\Theta_i - \frac{\delta}{c} v_i \right) ChAl_{ij} \right] + \frac{\delta}{c} \cdot \frac{v_j - v_i}{l_{ij}} \right\} \quad (8)$$

According to the stress table, available for each section, the sizes have been determined and the reinforcement of the concrete elements of the structure have realised. The system consists in two-directional horizontal girdles and vertical columns placed at each level, at the ends of the floors and at corners and intersections of the longitudinal and transversal walls, as follows from fig.7.

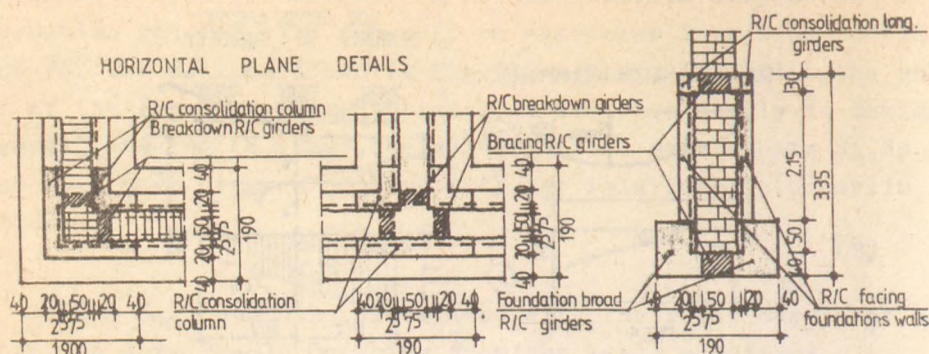


FIG. 7

The foundation width increase was adopted by applying the formula:

$$(b + 2b') = \frac{Q}{\sigma_z} \quad (9)$$

where the only unknown factor is b' . For σ_z a higher value than that shown in the geotechnical report was assumed, namely 110 KPa. The building is already in service by three years ago and the recorded of the total settlements is of 1 - 1.5 mm.

C. Acknowledgement

The authors express their gratitude to the management of the Building Design Institute of Galatz District for the allowance to use in this paper a series of the geotechnical data [4] which have contributed to the elaboration of the speciality part of the paper.

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ABLAGERUNGSBEHÄLTER VON KALKSCHLAMM IN DEN SODAWERKEN IN JANIKOWO (POLEN).

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Im Vortrag sind Gestaltungsarten äußerer Eindämmungen von Lagerplätzen für Abfälle aus der Sodaherstellung sowie auch Ergebnisse aus Untersuchungen zur Standfestigkeitsbestimmung dieser Eindämmungen dargestellt. Hingewiesen wird auf die Abhängigkeit der Standfestigkeit von der Gestaltungsart der Eindämmungen sowie von der Lagertechnologie der Abfälle auf Lagerplätzen. Darüber hinaus ist ein Vorschlag beschrieben, nach dem es möglich ist, eine sichere Abfallagerung bis zu 30 m Höhe zu realisieren.

1. EINFÜHRUNG

Die Sodawerke in Janikowo sind ein bedeutender Hersteller von Ammoniaksoda und Kochsalz. Die Jahresproduktion beträgt im Durchschnitt 500 000 t. Bei der Herstellung dieser Produkte fallen Abfälle in fester und flüssiger Form an, die auf speziellen Lagerplätzen gelagert werden. Die Durchschnittsmenge der einzelnen Abfallarten gestaltet sich folgendermaßen:

- **feste Abfälle:** Asche 8t/h; andere, wie Siebrücklauf aus Kalkstein, **Verbrennungsrückstände** aus den Kalkbrennöfen, Schlacke aus Kalzination 7 t/h - Gesamtmenge der festen Abfälle 15 t/h,
- flüssige Abfälle in Form von Destillierschlamm - Gesamtmenge 700 m³/h.

2. HERKÖMLICHE METHODE DER ABFALLAGERUNG

Die Beseitigung von Abfällen geht folgendermaßen vor sich: Die festen Abfälle, außer Asche, werden mit einer Schmalspurbahn zu den Klärbecken transportiert und werden genutzt zur Aufkastung der äußeren Eindämmungen. Die flüssigen Abfälle werden hydraulisch zu den Kläranlagen geleitet, wo

eine Abscheidung der festen Bestandteile erfolgt und die geklärte Flüssigkeit wird über Rampenanlagen und Abfanggräben sowie Rohrleitungen durch Schwerkraftwirkung zur Auswurf-Pumpstation geleitet. Die ursprüngliche Eindämmung der Klärbecken wird vor deren Betriebsnahme errichtet, indem Dämme in ca. 3,0 m Höhe (auf ebenen Geländen) aus grobem Sand, Kies, Steinsplitt mit einem Rückfilter in Form einer Lüftungsböschung aufgeschüttet werden. Der Boden der Klärbecken wird mit wasserdichter Folie ausgelegt und ein stark verzweigtes Drännetz angeordnet. Nach der Auffüllung eines Klärbeckens mit Schlamm in einer Höhe von ca. 0,5 m unterhalb der Dammkrone wird die Einflutung unterbrochen und die flüssigen Abfälle in ein weiteres Becken geleitet. Das so stillgelegte Klärbecken trocknet im Verlauf von einigen Monaten aus. Danach werden die Eindämmungen auf eine Höhe von 1,5 - 2,0 m aufgeschichtet mit Stoffen der festen Abfälle und das Becken wird erneut in Betrieb genommen.

3. ENTWORFENE METHODE ZUR LAGERUNG DER ABFÄLLE.

Die Menge der festen Abfälle zur Lagerung auf Lagerplätzen beträgt in Janikowo pro 1 Tonne hergestellten Sodas $1,4 \text{ m}^3$. Das bedeutet, daß jährlich etwa $700\,000 \text{ m}^3$ Abfall gelagert werden muß. Nach der herkömmlichen Lagermethode sowie bei abgesicherter Lagerhöhe wären jährlich etwa 6 ha Gelände von neu zu errichtenden Klärbecken belegt. Die Möglichkeit, neue Gelände dafür zu erschließen, sind begrenzt und belaufen sich auf 122 ha. Zusätzliche Reserven liegen darin, daß u.a. erwogen wurde, die schon bestehenden Lagerplätze zu erhöhen. Es sei hier angeführt, daß gegenwärtig die Klärbecken in einigen Fällen eine Höhe von etwa 17,0 m ü.d.M. erreichen.

Das Wesen einer hohen Lagerung sollte auf dem Einsatz einer neuen Technologie zur Abfallbesitigung und Gestaltung von Lagerplätzen beruhen. Nach den Grundsätzen dieser Technologie sind alle flüssigen und festen Abfallstoffe zu vermischen und über hydraulischen Transport in Rohrleitungen auf den Lagerplatz zu transportieren. Auf dem Platz (Klärbecken) werden die Stoffe bei der Absetzung einer gesteuerten Sedimentierung unterliegen. Im Ergebnis eines solchen Vorgehens setzt sich in der Dammzone das grösste Material ab und in Richtung Mitte des Klärbeckens werden die Absetzungen immer feiner. Dadurch werden die Festigkeitseigenschaften des Materials, das in Böschungsnähe der äußeren Eindämmungen abgesetzt ist,

verbessert und damit wird es möglich, dieses Material zur Aufschüttung der Eindämmungen einzusetzen bis eine Höhe der Klärbecken bis zu 30 m erzielt ist. Die Gestaltung eines Klärbeckens nach dem neuen Verfahren ist vom oberen Niveau aus vorgesehen, das durch die bisherigen Becken erreicht worden ist, wobei eine Neigung der Böschungen der äußeren Eindämmung von 1:3 vorgesehen ist. Die Art der Einfüllung der Klärbecken sowie deren Aufschüttung soll nach dem gleichen Prinzip wie bisher erfolgen, d.h. im Zyklus: Einfüllung, Trocknung, Erhöhung der Eindämmung, erneute Einfüllung usw., jedoch mit dem Unterschied, daß die Dammerhöhung ausgeführt werden soll mit Baggern und Einsatz des im Klärbecken abgesetzten Materials (Bild 1).

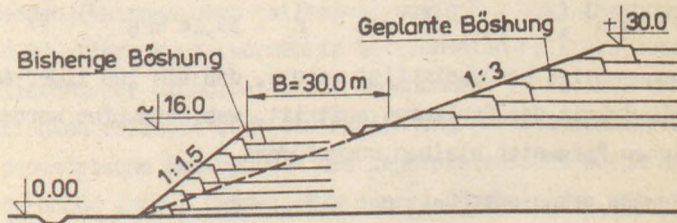


Bild 1. Böschung Querschnitt

4. STANDFESTIGKEITSANALYSE VON HOHEN KLÄRBECKEN

4.1. Bestimmung der physikalisch-mechanischen Parameter der Ablagerung

Unter Annahme, daß die Gleitfläche bei Standfestigkeitsverlust an den Eindämmungen im abgesetzten Stoff verlaufen wird, hatte die Bestimmung der Eigenschaften des Kalkschlammes grundsätzliche Bedeutung. Dieser Stoff zeichnet sich aus durch eine beträchtliche Veränderlichkeit der physikalisch-mechanischen Eigenschaften im Vertikalschnitt und im Aufriß des Klärbeckens las auch in der Zeit [1], [2], [3]. Diese Veränderlichkeit folgt aus dem Charakter des Sedimentierungsvorganges des Schlammes, der Art der hydraulisch angelieferten Abfälle sowie auch der Liegezeit und Trockenzeit. In der bestehenden Situation wurden die Ablagerungen in den

einzelnen Becken als eine homogene geotechnische Schicht betrachtet mit verallgemeinerten, statistisch bestimmten physikalisch-mechanischen Parametern σ , φ_u und c_u . Die Festlegung dieser Parameter erfolgte unter Nutzung einer großen Anzahl Untersuchungen an Klärbecken mit Hilfe einer dynamischen Sonde mit Kreuzaufsatz. Hinsichtlich der Möglichkeit von Variieren des Lagerplatzes wurde die statistische Analyse mit einem Vertrauensfaktor von 90% durchgeführt, das heißt, daß zu 90% die Wahrscheinlichkeit besteht, daß keine Parameter unterhalb der bestimmten auftreten. Im Ergebnis der durchgeführten statistischen Analyse wurden folgende Werte der charakteristischen Parameter von Kalkschlamm festgelegt:

$$\sigma = 12,75 \text{ kN/m}^3, \quad \varphi_u = 9,3^\circ, \quad c_u = 18,70 \text{ kPa}$$

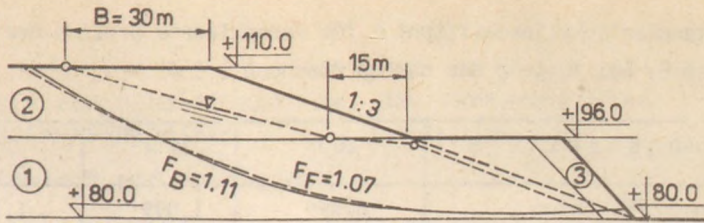
Für den Eindämmungsstoff der bestehenden Becken wurde angenommen:

$$\sigma = 13,73 \text{ kN/m}^3, \quad \varphi_u = 38^\circ \text{ und } c_u = 39,14 \text{ kPa}$$

Bei den Berechnungen ist berücksichtigt worden, daß ein 20% tiger Anstieg des inneren Reibwinkels des Schlammes auftritt, was entworfen worden war, und die restlichen Parameter bleiben unverändert.

4.2. Berechnung der Standfestigkeit der Klärbecken

Die Durchsicht der verschiedenen Berechnungsmethoden und - schemata sowie die Möglichkeit und Zweckmäßigkeit deren Einsatz in Betrachtungen von Soda-Klärbecken führten zu der Schlußfolgerung, daß die entsprechende Einschätzung der Standfestigkeit erzielt werden kann bei Stützung auf die Methode der zylindrischen Gleitfläche (Streifenmethode). Breite Berechnungen des Standfestigkeitskoeffizienten von Becken mit einer Erhöhung bis zu 30 m im Einklang mit der o.a. Technologie ergaben, daß eine Gewährleistung der Standfestigkeit der Eindämmungen eine beträchtliche Verschiebung der Linie der piezometrischen Drücke von der Böschung des entworfenen Lagerplatzes erfordert (Bild 2). Praktisch bedeutet dies die unbedingte Notwendigkeit, bei dem Betrieb des Klärbeckens die sogen. "Strand" zu erhalten (d.h. Abstand der Wasserrandlinie vom oberen Rand der inneren Böschungseindämmung) mit einer Breite nicht kleiner als 30 m. Gleichzeitig wurde die Notwendigkeit bestätigt, ein funktionierendes Dränsystem anzordnen, das ein Zurückweichen der Filtrationskurve auf eine Entfernung von mindestens 15 zur Böschungstiefe hin gewährleistet.



- ① Kalkschlamm $\gamma = 12.75 \text{ kN/m}^3$, $\varphi_U = 9,3^\circ$, $c_U = 18,7 \text{ kPa}$
- ② Geplante Sedimentation Schichte $\gamma = 12,75 \text{ kN/m}^3$, $\varphi_U = 11,2^\circ$, $c_U = 18,7 \text{ kPa}$
- ③ Schlacke+Kalkstein $\gamma = 15,93 \text{ kN/m}^3$, $\varphi_U = 38^\circ$, $c_U = 39,14 \text{ kPa}$

Bild 2. Rechnerische Querschnitt

Bei den o.g. Voraussetzungen konnte ein Standfestigkeitsfaktor von $F = 1,07$ erzielt werden (Methode nach Fellenius) sowie $F = 1,11$ (Methode nach Bishop) (Bild 2). Diese Werte wurden im betrachteten Fall als ausreichend für die Garantierung der Standfestigkeit angenommen. Im Verlauf der Analyse mit vielen Varianten erfolgte auch die Berechnung der Standfestigkeit für verschiedene geometrische Bedingungen des Lagerplatzes. Es wurde der Einfluß von Veränderungen in der Neigung der oberen Böschung des entworfenen Klärbeckens auf den Wert des Standfestigkeitsfaktors untersucht, Tafel 1 + 2. Es wurde festgestellt, daß ein relativ geringer Anstieg von F die Verringerung der Neigung begleitet. Berechnungen haben auch bewiesen, daß einen großen Einfluß auf die Standfestigkeit die Breite der sog. Absetzungen (Breme) auf der bestehenden Dammkrone haben. Es wurde die Notwendigkeit nachgewiesen, Absetzungen mit einer Breite von ca. 30 m zu setzen, denn sonst verschlechtert sich die Standfestigkeit des Beckens bedeutend, sogar bei einer Böschungsneigung von 1 : 4.

Tafel 1. Standfestigkeitskoeffizient F für verschiedene Neigungen der oberen Böschung, bei Absetzungen $B = 30 \text{ m}$.

B ö s c h u n g s n e i g u n g	1 : 3	1 : 3,5	1 : 4
Methode nach Fellenius	1,069	1,086	1,129
Methode nach Bishop	1,107	1,122	1,164

Tafel 2. Standfestigkeitskoeffizient F für verschiedene Breiten der Absetzungen B , bei Neigung der oberen Böschung 1 : 4.

Absetzungsbreite B	20 m	25 m	30 m
Methode nach Fellenius	0,999	1,099	1,129
Methode nach Bishop	1,037	1,105	1,164

5. SCHLUSSFOLGERUNGEN

Durchgeführte Varianten-Berechnungen für äußere Eindämmungen von Kalkschlamm-Klärbecken in den Sodawerken Janikowo haben ergeben, daß die Standfestigkeit u.a. abhängig ist von:

- den angenommenen physikalisch-mechanischen Parametern der Ablagerungen,
- Lage der piezometrischen Drücke in der Böschung der entworfenen Lagerplatzes,
- der Breite der Absetzungen auf der bestehenden Dammkrone,
- Neigung der Böschung der entworfenen Lagerplatzes.

Es wurde bestätigt, daß die Standfestigkeit von bis zu 30 m erhöhten Klärbecken bei Einsatz der neuen Lagerungs-Technologie (Aufschwemmung) erhalten bleibt bei Annahme der geometrischen Parameter: Absetzung mit 30 m Breite, Böschungsneigung 1 : 3 und Verschiebung der Linie piezometrischer Drücke entsprechend Bild 2.

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Der Einsatz von Schmalwänden bei der Sanierung einer Grundwasserkontamination

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1. Allgemeine Charakteristik des Altstandortes

Der Schadensfall der stillgelegten Altölraffinerie Pintsch in Hanau [1] stellte sich aufgrund der Erkundung des Hessischen Landesamtes für Bodenforschung [2] und der weiteren Untersuchungen des Ingenieurbüros Trischler und Partner [3] zusammenfassend wie folgt dar:

- Auf dem in ca. 3,50 m Tiefe anstehenden Grundwasser breitete sich eine bis zu 70 cm mächtige Altölphase mit einer flächenhaften Ausdehnung über ca. 20.000 m² aus.
- Das unter der Öllinse anstehende freie Grundwasser war erheblich mit gelösten oder emulgierten Schadstoffen belastet (chlorierte Kohlenwasserstoffe, aromatische Kohlenwasserstoffe).
- Die nachfolgend beschriebenen Bodenschichten sind insbesondere in der Wasserwechselzone, in der sich die ölige Phase bewegt, mit Altöl gesättigt.

2. Geologische und hydrologische Standortsituation

Die geologischen Verhältnisse des Standortes (Abb. 1) sind durch folgende Schichtenbereiche charakterisiert: Nach unterschiedlich mächtiger Auffüllung von 1 - 2 m folgt eine im Mittel 0,5 - 1,8 m mächtige Auelehmschicht, die jedoch an zahlreichen Stellen bei früheren Baumaßnahmen entfernt und durch Auffüllung ersetzt wurde.

Der Auelehm ist unterlagert von einer 5 - 8 m mächtigen quartären Schichtenfolge der Mainterrassen aus Kiesen und Sanden. An der Basis des Quartärs sind z.T. grobe Kiese und Gerölle mit geringem Sandanteil abgelagert.

Das Liegende besteht bis in große Tiefen ($t > 50$ m) aus einer tertiären Schichtenfolge, die mit Tonen und Schluffen eine wirksame Barriere gegen das Eindringen von Schadstoffen in tiefere Grundwasserleiter darstellt.

Der Standort ist charakterisiert durch seine unmittelbare Nähe zum Main. Bei Normalwasserstand des staugeregelten Mains liegt der Grundwasserspiegel geringfügig höher als der Mainwasserspiegel, so daß ein natürliches Gefälle von durchschnittlich 1 ‰ in Richtung Main vorhanden ist.

3. Sanierung

Die Maßnahmen zur Behandlung des Umweltschadens wurden in folgende Sanierungsstufen eingeteilt:

- Phase 1: Erkundung, vertiefende Untersuchung, Aufstellung eines Sanierungskonzepts
- Phase 2: Sofortmaßnahme zur Gefahrenabwehr
- Phase 3: Dauerhafte Gefahrenabwehr, Grundwasserreinigung, Abschöpfung des in Phase auf dem Grundwasser schwimmenden Öls
- Phase 4: Beseitigung von Schadstoffquellen, Abriß der alten Betriebsanlagen
- Phase 5: Bodenbehandlung und endgültige Sanierung des Standortes

Für die dauerhafte Gefahrenabwehr (Phase 3) wurde eine Kombination aus konstruktiven und hydraulischen Maßnahmen ausgewählt:

Aus 7 Abschöpfbrunnen wird kontinuierlich Grundwasser entnommen und dadurch ein Absenktrichter im Bereich der Öllinse erzeugt. Das kontaminierte Grundwasser wird in einer mehrstufigen Aufbereitungsanlage gereinigt. Die aufschwimmende Ölphase wird separat abgeschöpft (Abb. 1). Der Schadensbereich wird mit einer Dichtwand vollständig eingeschlossen. Diese Dichtwand ist dabei nicht vordergründig zur Einkapselung der Schadstoffe vorgesehen, sondern als temporäre Maßnahme zur Verminderung der anfallenden Pumpwassermengen in den Abschöpfbrunnen.

Aus diesem Grunde mußten die Anforderungen an Dichtigkeit und Beständigkeit der Wand nicht übermäßig hoch angesetzt werden und man konnte sich daher für die kostengünstige Variante einer Rüttelschmalwand entscheiden.

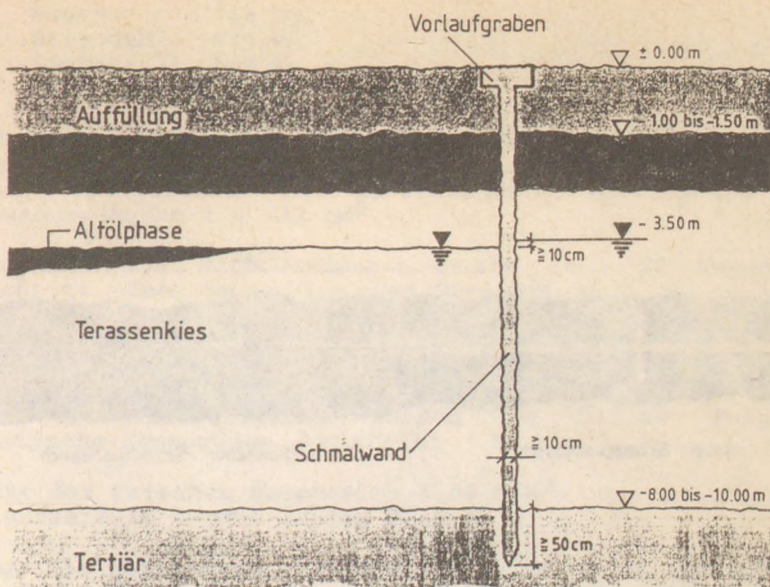


Abb. 1: Typischer Schnitt durch die Schmalwand bei Betrieb der hydraulischen Sanierung

Eine Schmalwand wird hergestellt, indem ein Stahlprofil (z.B. leichtes doppeltes H-Profil) in erforderliche Tiefe in den Boden gerüttelt wird. In diesem Falle wurde die Länge der Stahlbohle zu 10 m gewählt, um mindestens eine Einbindung von 50 cm in die tertiären Schichten zu erreichen (Abb. 1).

Beim Einrütteln und vor allem beim Ziehen des Stahlprofils wird der Hohlraum im Boden durch Einpressen der Dichtwandsuspension (Kap. 4) durch aufgeschweißte Injektionslanzen mit Öffnungen am Fuß gefüllt. Die Dicke der Wand wird durch Verstärkungen am Fuß des Profils bestimmt und beträgt in der Regel 8 bis 10 cm.

Der Anschluß eines neuen Schmalwandstichs an den vorherigen Stich wird durch eine Überlappung von ca. 10 cm (Abb. 2) erzielt, was in der Regel dadurch erreicht wird, daß der Flansch des Stahlprofils in den noch flüssigen Flanschbereich des vorherigen Stiches geführt wird.

Darüberhinaus ist die durchgehende Verbindung und Dichtigkeit der Wand durch ständige Vertikalitätsprüfungen des Rüttelprofils zu kontrollieren.

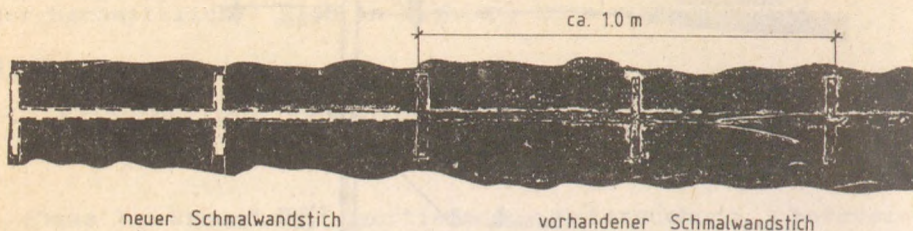


Abb. 2: Typische Querschnittsformen und Abmessungen der Schmalwand

Zur Vermeidung von starker Verschlämmung der Geländeoberfläche wird die Überschußsuspension in einem ca. $0,5 \times 0,5$ m großen Vorlaufgraben aufgefangen (Abb. 1). Die Zusammensetzung der Suspension ist von den Bodenverhältnissen abhängig und wird für das vorliegende Beispiel im Folgenden erläutert:

4. Anforderungen an die Schmalwand und Eignungsversuche

Vor Beginn der Arbeiten war das Mischungsverhältnis der Suspension im Rahmen einer Eignungsprüfung im Hinblick auf Festigkeit, Durchlässigkeit und Erosionsbeständigkeit festzulegen. Bei der Auswahl der Zementsorte war zu berücksichtigen, daß die Zusammensetzung des Grundwassers als "stark angreifend" gemäß DIN 4030 zu bezeichnen war. Die Eignungsuntersuchung enthielt die Forderung der Durchlässigkeit $k_f \leq 10^{-8}$ m/s nach 14 Tagen (nach DIN 18130).

Auf der Grundlage der Ergebnisse der Eignungsuntersuchungen wurde folgendes Mischungsverhältnis je m^3 frischer Suspension festgelegt:

Wasser:	749 kg
Steinmehl:	494 kg
Zement:	150 kg
Bentonit:	26 kg

Die frische Suspension hatte im Durchschnitt folgende physikalische Eigenschaften: Dichte $1,43 \text{ t/m}^3$, Fließgrenze 14 N/m^2 , Auslaufzeit im Marshtrichter $t_M = 34 \text{ sec}$ und Filterwasserabgabe $f = 142 \text{ cm}^3$.

Die ausgehärtete Dichtwandmasse hatte nach 28 Tagen im Durchschnitt einen Durchlässigkeitsbeiwert $k_f = 5,1 \times 10^{-10} \text{ m/s}$

5. Kontrollen während der Bauzeit

Während des Baubetriebes wurden im Mittel folgende physikalische Kenngrößen ermittelt:

- Dichte der frischen Suspension 1.45 t/m^3
- Auslaufzeit im Marshtrichter $40,5 \text{ s}$

So lange die geforderte Dichte eingehalten wurde, konnte die Bestimmung der Filtratwasserabgabe auf Stichproben beschränkt werden. Gleiches gilt für die Fließgrenze, solange die Auslaufzeit im Marshtrichter nicht unterschritten wurde. Die beiden oben genannten Parameter, die bei jedem Schmalwandstich gemessen wurden, zeigten konstante Werte mit vernachlässigbar kleinen Standardabweichungen und stellten so die gleichbleibende Qualität der frischen Suspension sicher.

An insgesamt 6 Proben nach 28 Tagen Aushärtung wurden folgende Kennwerte im Mittel gemessen:

- Durchlässigkeit $k_f = 3,4 \cdot 10^{-4} \text{ m/s}$
- einaxiale Druckfestigkeit $q_u = 2,85 \text{ MN/m}^2$

Während der Schmalwand-Arbeiten wurden zahlreiche Qualitätskontrollen durchgeführt:

- Je 1.000 m^2 fertiggestellte Schmalwand waren die Eignungsversuche an der frischen Schmalwandsuspension bzw. am erhärteten Probekörper zu wiederholen. Mit Ausnahme von Durchlässigkeit und Druckfestigkeit wurden alle Versuche in einem Feldlabor auf der Baustelle durchgeführt.
- Für jeden Schmalwandstich wurden alle wesentlichen Produktionsdaten, wie Suspensionsverbrauch, Einbindetiefe ab Geländeoberkante, Einbindetiefe in den Tertiärhorizont, Injektionsdruck, Zusammensetzung und Dichte der Suspension und Verarbeitbarkeit der Suspension (Auslaufzeit im Marshtrichter) protokolliert.

- Der Nachweis der Einbindung in den Tertiärhorizont war durch eine automatische, kontinuierliche Registrierung des Hydraulikdruckes des Rüttlers zu erbringen. Zusätzlich wurde bei jedem 3. Stich eine Bodenprobe vom Fuß der Rüttelbohle, die hier mit einer speziellen Probenfangvorrichtung ausgestattet war, abgeschalt und geologisch angesprochen.

Im Bereich von Kreuzungen der Schmalwandtrasse mit Leitungen der öffentlichen Versorgungsträger, war es vorgesehen, die Schmalwand beidseitig bis auf ca. 1,5 m Abstand an diese Leitungen heranzuführen und den verbleibenden Zwischenraum nach Herstellung der Schmalwand mit Hilfe von Manschettenrohrinjektionen zu schließen. Diese Maßnahme erwies sich als nicht erforderlich, da alle Leitungen oberhalb des Grundwasserspiegels liegen und mit einem Schwert am Fuße der Rüttelbohle unterfahren werden konnten, um die Schmalwand lückenlos zu schließen.

Verfahrensbedingt erzeugt das Rüttelschmalwandverfahren erhebliche Vibrationen im Untergrund. Dies führte auf einer Hofffläche zu Setzungen der Asphaltoberfläche im Bereich der Schmalwandtrasse. An einer parallel zur Trasse verlaufenden öffentlichen Straße wurden Risse in der Asphaltdecke festgestellt, die später repariert werden mußten. Die Auswirkungen der Vibrationen waren auch an einem an der ungünstigsten Stelle ca. 20 m entfernten Tanklager zu spüren. Durch Schwingungsmessungen an den nächstgelegenen Tanks konnte allerdings nachgewiesen werden, daß eine Gefährdung für Betonboden und Tanks jederzeit ausgeschlossen war.

Insgesamt wurden von September bis Dezember 1988 ca. 760 lfdm. Schmalwand mit einer mittleren Tiefe von 8,60 m hergestellt. Weiterhin wurden auf dem Gelände selbst 5 Zellen mit den Abmessungen 5 x 10 m durch Schmalwände gegeneinander abgeteilt bzw. eingeschlossen. Diese Zellen dienen als Versuchszellen für ein mikrobiologisches Forschungsprojekt.

6. Prüfungen nach der Fertigstellung

Bei Dichtwandarbeiten stellt sich immer wieder die schwierige Frage, wie das Bauwerk den Qualitätsforderungen entsprechend abgenommen werden kann. Neben den in Kapitel 5 erläuterten Materialeigenschaften steht die Dichtigkeit der Wand als Ganzes und damit ihre Funktionstüchtigkeit im Vordergrund. Auf dem Pintsch-Gelände wurden dazu die folgenden vier Prüfungen durchgeführt:

Im südlichen Bereich des Geländes wurde ein Teil der Schmalwand bis in ca. 3 m Tiefe freigelegt und visuell geprüft. Fehlstellen waren in diesem Bereich nicht vorhanden. Es wurde eine Schmalwandstärke von ca. 12 - 18 cm ermittelt.

In einer der mikrobiologischen Versuchszellen wurde anhand eines lokalen Absenkversuches die Durchlässigkeit der Schmalwand in-situ zu $7,4 \cdot 10^{-8}$ bis $2,5 \cdot 10^{-8}$ m/s ermittelt.

Das gesamte hydraulische System innerhalb und außerhalb der Schmalwand wird durch über 103 Grundwassermeßstellen überwacht, die z.T. mit kontinuierlich aufzeichnenden und speichernden Meßwertgebern ausgerüstet sind. Im Rahmen der Sanierung wird in 7 Brunnen auf dem Gelände verunreinigtes Wasser entnommen und nach Reinigung wieder versickert. Durch regelmäßige Ableitung von Überschußwasser in den naheliegenden Main wird sichergestellt, daß der Innenwasserspiegel immer mindestens 10 - 30 cm unterhalb des Außenwasserspiegels liegt. Hierdurch wird ein Schadstofftransport nach außen vermieden.

Wasserbilanzrechnungen an diesem System ergeben unter Vernachlässigung der Unterströmung der Schmalwand über den gesamten Schmalwandbereich, also auch unter Einbeziehung möglicher Fehlstellen einen mittleren k_f -Wert von 10^{-7} - 10^{-8} m/s. Dies entspricht somit einer mindestens 1000-fachen Verminderung des ursprünglichen k_f -Wertes des Terrassenkieses von 10^{-4} m/s.

Die vorgenannten vier Kontrollen waren im Falle Pintsch deshalb ausreichend, weil im Schutze der Schmalwand das Grundwasser abgesenkt und gereinigt wird und damit die Wand nur eine temporäre Aufgabe erfüllt. Beim Einsatz von Schmalwänden zur dauerhaften Abkapselung von Altlasten empfiehlt sich der Bau einer doppelten Wand im Abstand von ca. 1,0 m, die mit Querschotten in eine Vielzahl von Zellen unterteilt wird [5]. In jeder einzelnen Zelle kann dann über Jahrzehnte durch Absenkung die Dichtigkeit kontrolliert werden.

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VERWENDUNGSMÖGLICHKEITEN DER BUDAPESTER MÜLLVERBRENNUNGSASCHE (-SCHLACKE) IN STÄDTISCHEN TIEFBAU

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ZUSAMMENFASSUNG: Das Institut für Verkehrswissenschaften hat die technischen Vorschriften der Verwendung der Müllverbrennungsasche im Strassenbau ausgearbeitet. Der Autor ist der Meinung, dass die Müllasche auch in Stättischen Tiefbau - als Gründungen, bzw. als Verbesserungs- und Isolierungsschichten - wirtschaftlich verwendbar ist.

EINLEITUNG

Im ersten (nördlichen) Müllverbrennungswerk von BUDAPEST entsteht jährlich etwa sechzig tausend Tonnen Müllverbrennungsasche (bzw. Kehrrichtschlacke, im weiteren : Müllasche). Es ist die Vorbereitung der Zustandebringung des zweiten (südlichen) Werk im Gange.

Material- und Energieersparungsforderungen machen der - aus technischen und wirtschaftlichen Gesichtspunkten - optimale Benutzung der Müllasche notwendig.

Das Institut für Verkehrswissenschaften (KTI) hat die Verwendungsmöglichkeiten der Müllasche im Strassenbau ausführlich untersucht. Hier wurden erstens die diesbezüglichen schweizerischen, französischen und westdeutschen Erfahrungen studiert, dann wurden laboratorische Versuchsserien durchgeführt. Aufgrund der ermutigenden Ergebnissen hat das Institut den Bau und die regelmässiger Beobachtung einiger Versuchsstrecken angeregt. Mit der Auswertung der so erhaltenen günstigen Erfahrungen hat das Institut die technischen Vorschriften der Verwendung - für die folgenden Benützungsgebiete - ausgearbeitet:

- mechanische Stabilisierung: Trag-, Schutz- und Verbesserungsschichten, Dämme,
- zementstabilisierte Tragsschichten,
- heissgemischte bituminöse Tragschichten.

Aufgrund der günstigen Erfahrungen im Strassenbau kann man auf einige wirtschaftlichen Verwendungsmöglichkeiten auch im städti-

schen Tiefbau folgern.

AUFBEREITUNG DER MÜLLASCHE

Der Beim Verbrennungsprozess anfallende Rückstand wird als Rohasche bezeichnet. Für die Verwendung im Strassen- und Tiefbau muss die Rohasche aufbereitet werden. Die Aufbereitung umfasst eine Verminderung des Wassergehaltes - durch Lagerung in einer Zwischen-deponie während 2-3 Monaten - und eine Aussortierung der Metalle und weiterer groben Bestandteile.

Die Aufbereitungsanlage besteht aus einer geeigneten Kombination von Förderelementen, Magneten und Vibrationssieben.

Die Aufbereitung und der Einbau von Müllasche ist in Schutzzonen um Trinkwasserefassungen und Quellen ausgeschlossen.

ZUSAMMENSETZUNG UND EIGENSCHAFTEN DER MÜLLASCHE

Die budapester Müllasche besteht aus der folgenden Materialien:

50 - 60 % Schlacke,

5 - 15 % hydraulische Flugasche (mit einer Puzzolanaktivität von 69, und einem aktiven CaO-Gehalt von 10 - 11%),

20 - 30 % Glas- und Porzellanstückchen,

1 - 3 % Metalle.

Seine wichtigsten Oxid-Komponenten (in Klammer die Werte der EWG) sind, wie folgt:

SiO ₂ :	35 - 44 %	(42 - 58 %)
Al ₂ O ₃ :	9 - 12 %	(6 - 18 %)
Fe ₂ O ₃ :	7 - 12 %	(8 - 18 %)
CaO :	13 - 19 %	(9 - 12 %)
MgO :	2 - 3 %	(1 - 2 %)
Na ₂ O :	0,1 - 0,9 %	(3 - 8 %)
SO ₃ :	4 - 9 %	(0,7 - 6,4 %)

pH Wert: 7,6 - 10,7

Kornverteilung:

unter 0,1 mm: 5 - 15 % (der Flugaschegehalt),
" 2,0 mm: 30 - 50 %,
" 20,0 mm: 80 - 100 %

Sowohl durch die PROCTOR-, wie die Walzenverdichtung werden die grösseren Teilchen der Müllasche teilweise zerkleinert.

Die PROCTOR-Verdichtungswerten:

- maximales Trockenraumgewicht: 1700 - 1800 kg/m³
- optimaler Wassergehalt: 12 - 18 %

Die gut verdichtete Müllasche ist praktisch wasserundurchlässig, also ist sie als Filter- oder Sicker- (Drän-) Material unbrauchbar.

Die Tragfähigkeit können wir mit dem CBR-Wert charakterisieren, dessen Angaben in allgemeinen zwischen 60 und 80 % liegen. Dieser Wert steigt sich in der Zeit, weil sich die hydraulische Flugasche langsam bindet.

Die aufbereitete Müllasche ist mindestens mit einem natürlichen (unklassifizierten) Sandkies gleichwertig.

VERWENDUNG OHNE BINDEMITTEL

Das am besten verbreitete und einfachste Benützungsgebiet ist die mechanische Stabilisierung.

Die Kornverteilung der Müllasche muss die Vorschriften der Mechanischen Stabilisierung - mit der Berücksichtigung der Zerkleinerung der grösseren Teilchen während der Verdichtung - erfüllen.

Die max. 30 cm dichten losen Schichten sind mit Gummirad- oder Vibrationswalze zu verdichten - bis zum Erreichen des vorgeschriebenen Verdichtungsgrad (bzw. Tragfähigkeit).

Die solchen Tragschichten bewähren sich in den Befestigungen der budapester Probestrecken als genügend tragfähig. Nach dreijährigen Kontrollmessungen haben wir keine signifikante Grundwasserverschmutzung beobachtet.

Im städtischen Tiefbau ist die aufbereitete Müllasche als spezielle Dämme und Verbesserungsschichten zu verwenden.

VERWENDUNG MIT BINDEMITTELN

Die aufbereitete Müllasche können wir mit Zugabe von 5 - 6% Hochofenschlacke-Portlandzement sehr günstig stabilisieren. Seine einaxiale Druckfestigkeit erreicht 10.000 kPa.

Solchen Stabilisierungen sind als einfache Magerbetongründungen im Tiefbau anwendbar.

Aus Mischung von Müllasche und sandigen Kies können wir mit Dosierung von 5 % Bitumen heisse bituminöse Tragschicht herstellen.

Die Verwendung der Bitumenemulsion als Bindemittel hat die folgenden Vorteile:

- die nasse Müllasche soll nicht ausgetrocknet werden (Energieersparung),
- die eingebaute Mischung mit einem Müllaschegehalt von 70 % - mit von 4,5 % Bitumen (in Form einer Emulsion) - hat eine Anfangsstabilität von 7,5 kN,
- diese Stabilität vergrössert sich 2,5-mal nach drei Monaten, weil die hydraulische Flugasche bindet sich langsam (Doppelbindung).

Diese bituminösen Mischgüter sind in Tiefbau als Isolierungsschichten benutzbar.

VORTEILE DER VERWENDUNG

Mit einer aufbereiteten Müllasche können wir den natürlichen Sandkies ersetzen.

Die Müllasche ist besser verdichtbar, als Sandkies und gebrochene Gesteine.

Eine Mischung von Müllasche und natürlichem Sandkies hat wegen des eckigen Schlacken- und Glasstückchengehalts der Müllasche eine grössere innere Reibung, als die von Naturkies.

In den bituminösen Mischungen ersetzt der Flugaschengehalt den übrigen notwendigen Kalksteinmehl (Füller).

Der hydraulische Flugaschengehalt der Müllasche ist sehr vorteilhaft, weil:

- sich die mechanische Stabilisierung langsam verfestigt,
- bei der Stabilisierung mit Zement eine geringere (5 %) Bindemitteldosierung nötig ist,
- sich bei Dosierung von Bitumenemulsion eine Doppelverbindung abspielt: die Stabilität erhöht sich langsam.

STRENGTHENING THE FOUNDATION OF A DWELLING HOUSE AGAINST ADDITIONAL SETTLEMENT

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1. INTRODUCTION

Foundation of different structures on weak soil is always a serious engineering problem. Observations of buildings constructed in such conditions indicate that still proceeding, often nonuniform settlement is observed even after many years.

Such behaviour of structures impedes and often makes impossible their safe exploitation. In such cases the problem of strengthening the soil or transmitting the loads to deeper lying bearing layers appears. In this paper such a problem is presented on the example of a dwelling house in Gdańsk which was built on fill.

2. STRUCTURE AND SETTLEMENT OF THE BUILDING

The dwelling house was built in 1962. It's dimensions were: length 73,6 m and width 12,9 m. The building's structure consists of 3 segments divided by expansion joints of respective length: segment A - 31,2 m, segment B - 23,7 m, and segment C - 18,7 m. The building is a 5 - story structure with transverse alignment of bearing walls which are spaced 5,05 m and 2,85 m apart (fig.1). The bearing walls and walls of cellars are made of poured concrete, and their thickness is 0,25 m and 0,4 m. The longitudinal external walls are made of aerated concrete blocks. The building is founded at 2 m depth on a reinforced concrete plate of 0,4 m thickness.

Some time after construction of the house, nonuniform settlement, of segment A mainly, deflection of walls from vertical and slight damage of the building were observed. Since 1982 systematic measurements of settlement

of wall verticality were carried out. In 1986 the deflection of walls from vertical reached 150 mm to 185 mm (fig.1).

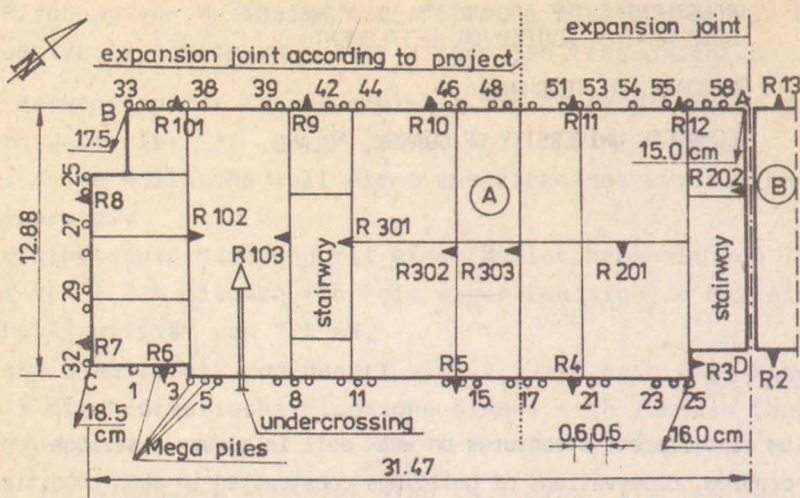


Fig. 1 Building foundation layout (segment A).

The reference measurement for settlement evaluation (series I) was made in December 1982. The rate of settlement during the following years is shown in fig.2. Until September 1988 (series XII), settlement at the corners of the building reached $R7 = 13,3$ mm, $R3 = 8$ mm, $R8 = 10,2$ mm, $R12 = 6,4$ mm respectively.

It should be mentioned that three other buildings of the same type and with the same structural system have been built in the direct neighbourhood, and these buildings show uniform settlement. The problem building differs by the location of expansion joint only. This resulted in asymmetry of the building and caused a nonuniform distribution of loads.

From calculations the following nonuniformity of stresses below the foundation plate (fig.1) was obtained

$$\frac{\sigma_C}{\sigma_B} = 1,05 ; \quad \frac{\sigma_C}{\sigma_A} = 1,32 ; \quad \frac{\sigma_C}{\sigma_D} = 1,25 ; \quad \frac{\sigma_D}{\sigma_A} = 1,06$$

This distribution agrees qualitatively with measurements of vertical and horizontal displacements of the building. Additionally, constant increase of settlement is caused by variable spatial stiffness in vicinity of

expansion joints, variations of groundwater level and decomposition of organic particles in the zone of ground water level variability.

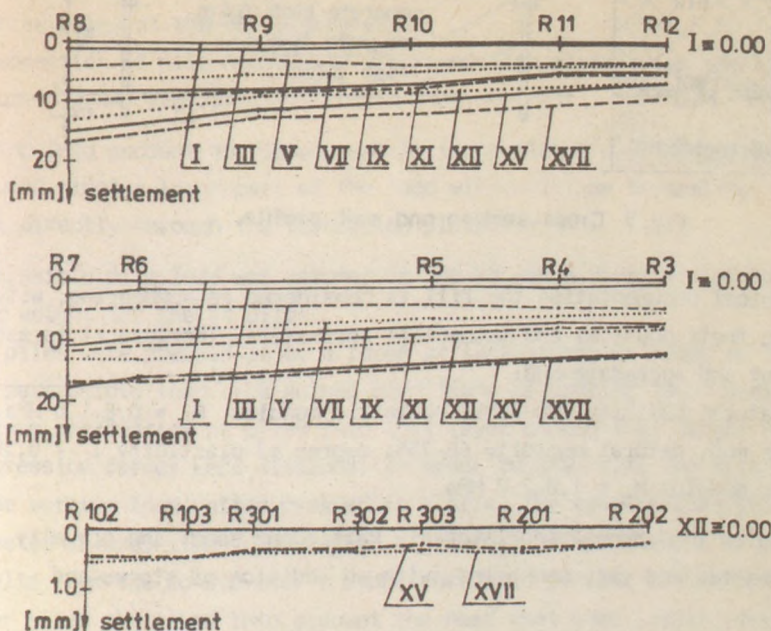


Fig. 2 Measured settlements.

Though over 25 years have passed since construction, continuous, growing and nonuniform increase of settlements was observed, which more and more impeded the use of the building. In this situation it was decided to strengthen the foundation in order to slow down the settlement and to allow straightening the building. Protecting and strengthening works were started in September 1988.

3. GEOTECHNICAL CONDITIONS

Between the surface of the terrain and the depth of about 5 m the whole area is formed very nonuniform mineral and organic fill (crushed brick, fine sands, sand with humus, aggragate mud, ceramics). Below is a thin layer (0,3 to 0,5 m) of Holocene material, consisting of aggragate mud and peat mainly. These soils are weak and subject to large deformation, fig.3.

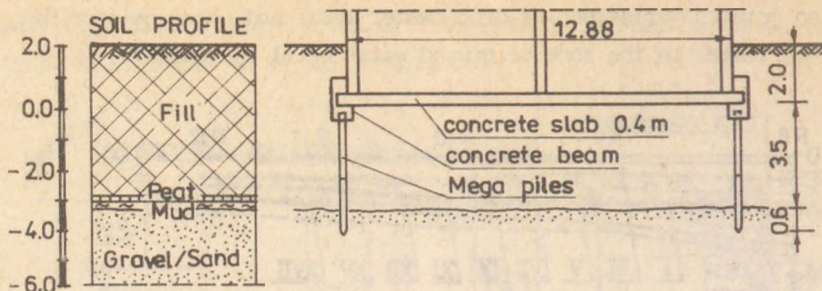


Fig.3 Cross-section and soil profile.

In geotechnical documentation the fill is considered as nonbearing, without giving their physical and mechanical parameters. Therefore it is assumed for peat and aggrade mud:

- peat, natural humidity 180-300%, edometric modulus $M_0 = 0,8-1,0$ MPa,
- aggrade mud, natural humidity 60-75%, degree of plasticity $I_L = 0,50$, edometric modulus $M_0 = 1,8-2,0$ MPa.

Directly below this nonbearing layer are Paistocene sandy and gravelly soils, compacted and very compacted, with an addition of stones and aggregate.

At 10 ÷ 12 m below terrain surface is compacted fine sand with clay laminations.

Analysis of archival geotechnical documentation shows temporal oscillations of ground water level. The groundwater table is at about 5 m below terrain surface, and water table oscillations reach 1,5 m.

4. STRENGTHENING OF FOUNDATIONS AND PROTECTION OF BUILDING

In the described above situation, the object may be protected in several ways. The main factors considered in this case are the type of structure of structure of the building, geotechnical conditions, method of founding and existing technical and technological potential of the contractor.

After analyzing several variants, it was finally decided to strengthen the building's foundations by Mega type pressed piles. The piles were pressed in by a special hydraulic device, constructed at the Piling Works Contractor, eng. Klaus (Katowice). The following were assumed:

- prefabricated reinforced concrete piles in 1 m and 2 m sections of

25 cm diameter, pressed in along the external wall after prior local removal of the reinforced concrete foundation plate,

- placing the piles along the building's perimeter, fig.1, in groups of three piles at the transverse walls,
- connection of pile heads with the foundation plate by an additionally formed beam, fig.3.

Taking into account the many years of the building's existing, it was also assumed, that a large part of the load will still be transmitted to the soil directly through the foundation plate.

Permissible pile load was assumed at 300 kN which gives 51% of total building weight for the 58 piles.

The piles were pressed-in by a force at least twice as large as the assumed permissible load. The actual conditions of bearing layer position and minimum penetration of piles into that layer caused that larger values of compression forces were attained. In order to stabilize the settling curve under working load, after pressed in a pile, the same maximum load was repeated several times. The number of repetitions of loading of each piles results from the constructor's experience with similar soil conditions at other sites and takes into account the need that some larger settlement must occur under the west wall of the building in order to achieve in time straightening of the building. Actual parameters, obtained during construction, are following:

- for the south and east walls, piles no. 1-32 (fig.1), average compression force - 694 kN, average number of maximum force repeats - 14, pile penetration below foundation plate bottom - 4,2 to 4,8 m, pile penetration into bearing layer 0,6 to 0,8 m;
- for the west wall, piles no. 33-58, average compression force - 786 kN, average number of maximum force repeats - 3, pile penetration below foundation plate bottom - 4,0 to 4,5 m, pile penetration into bearing layer 0,6 m.

The data show that the piles are relatively short and only slightly penetrate into the bearing layer. Their actual bearing capacity was evaluated basing on three test loadings. As an example, the settlement curve for pile no. 24 is shown in fig. 4. The pile length was 4,3 m, penetration into bearing layer - 0,6 m, maximum pressed-in load - 600 kN, number of maximum force repeats - 15. From the analysis of the settlement curve it may

be stated, fig.4, that even loads exceeding 2,5 times the working load are very well transmitted by the pile. A similar characteristic was obtained for all test piles. Test pile no. 39, for which the number of maximum load repeats was only 5, showed - in agreement with assumptions - a slightly larger settlement.

As it was mentioned earlier, since 1982 settlement was monitored outside the building. Taking into account the implemented technique of supporting the building along its external perimeter, before starting construction works additional datum points were established along the building's longitudinal axis. Examples of settlement results obtained for all the walls are shown in fig.2. Measurement series no. XII, September 1988 is treated as the zero (reference) measurement before the strengthening. Measurement series XV, January 1989, represents they state directly after the work was finished. The assumed technological procedure ensured the settlement of the building of 3 to 4 mm during the whole period of work.

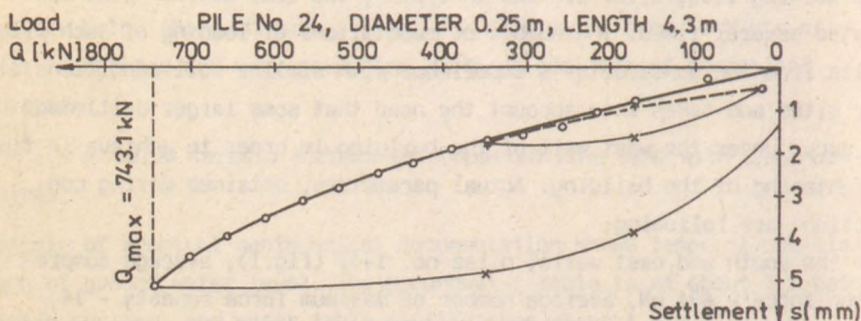


Fig.4 Load settlement curve, pile no 24.

Last measurements were carried out in March 1990, i.e. 1 year after finishing work (series XVII). On the eastern side of the building displacement has practically stopped and settlement is of the order of 0,2 - 0,4 mm in 1 year. In agreement with assumptions, larger settlement (of 1,2 to 2,1 mm) occur on the west side of the building, fig.2, which will lead to slow straightening of the house.

Datum points placed along the centre wall show uniform settlements, proportional to settlement of external datum points. This indicates that loads are correctly transmitted by the foundation plate to the soil and that the building is not excessively bent.

5. CONCLUSIONS

During the exploitation of a 5-story dwelling house, founded on a concrete slab placed on a 4 m thick mineral and organic fill with aggregate mud and peat laminations, nonuniform settlement and tilting was observed even after 25 years.

In order to protect and straighten the building, short, reinforced concrete, type Mega pressed-in piles were used which penetrated into the bearing layer.

Measurement of settlement of the whole object during and after the strengthening works and test loading of the piles confirmed that the concept of supporting the building along its perimeter was correct.

Such a method of supporting the object significantly lowered costs and shortened the time of the construction works and made them much easier (no work in cellar level was performed). During all construction work the building was normally exploited and there was no need of removing tenants from the house.

Foundation reinforcing of surrounding buildings around construction sites in urban areas by using the jet grouting technology

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INTRODUCTION

An office building was built on the site 12. Vadász street (Fig. 1.). The excavation level of the working pit was generally about the level of foundation soles of the surrounding buildings. Along the north-west side the new foundation had to be placed actually under the level of foundation of the neighbouring buildings.

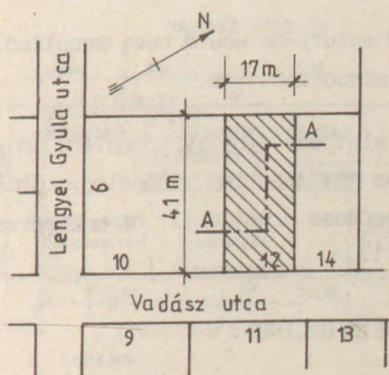


Fig. 1. Layout

According to the soil investigations down to 1,5-2 m a backfill with rubbles of ancient buildings, then yellow sandy gravel or gravelly sand layers were found as characteristic soil types (see Fig. 2.).

CHOICE OF THE FOUNDATION REINFORCING TECHNOLOGY

In order to increase the safety against the limit state of subsoil failure, either deepening of existing foundation levels (e.g. by means of a step-by-step underpinning of old walls by grouting, etc.), or to cut a continuous "sheet pile like" diaphragm wall or pile wall along the existing foundations came into question.

The step-by-step underpinning procedure had have needed strong labour and time-consuming efforts, and for the grouting symmetrically located, inclined boreholes must have been created partly through the foundation walls, which moreover, needed extended working room in the neighbouring cellars, more than those actually had. Because of these problems the above mentioned methods were rejected.

Stabilizing diaphragm walls or r/c pile walls might have been suitable, but these were also refused because their regarding technologies would have needed the prior removal of some subsurface wall and stone remains just laying in the track line of those. Furthermore, it would have needed a high amount of backfill.

All of the above mentioned solutions would have resulted in a significant increasing of cost and construction time.

The offer of Hídépítő Vállalat was finally accepted, which suggested an adequate application of the jet grouting technology. This solution became the most economical one, because practically no underground preparation works were necessary for it.

DESIGN OF THE FOUNDATION REINFORCING

The detailed design prescribed 3,0 to 3,5 m long jet grouted columns of 80 cm diameter. Studying Fig. 2 it is to be seen, that at locations having a deeper level of new foundation than that of surrounding one, two rows of steel bar reinforced jet grouted soil columns were planned.

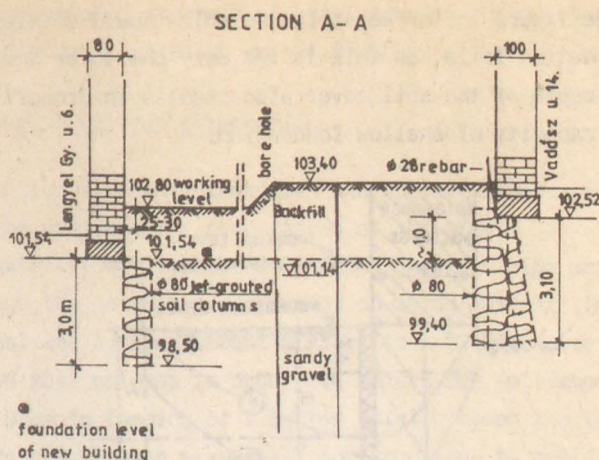


Fig. 2. Section A-A

These columns were distributed touching, or penetrating into, one another, adequate to the stability requirements (Fig. 3.).

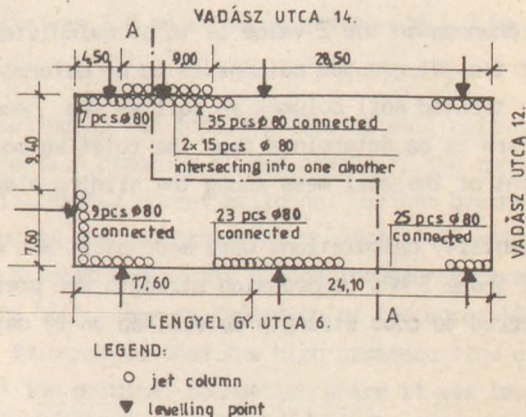


Fig. 3. Spacing of jet-grouted piles and levelling points

The stabilizing effect of the jet grouted soil columns prevents the formation of a dangerous sliding plane marked by 1 on Fig. 4. The sliding plane marked

by 2 on the same figure is the possible one which could develop if the reinforced foundation fails, so this is the only one to be analysed. The increasing depth of the soil cover also results in proportional increasing of the bearing capacity of shallow foundation.

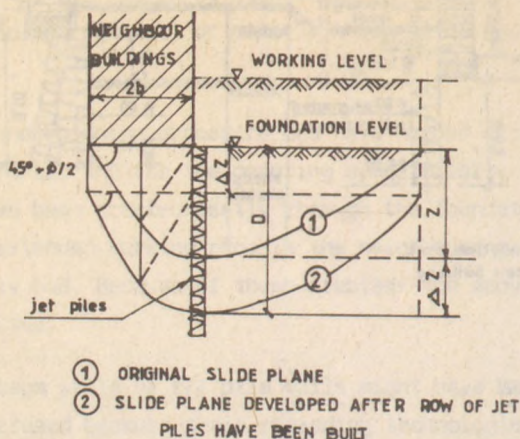


Fig. 4. Effect of jet piles on developing of slide planes

According to this phenomenon the Z value is to be calculated and the necessary depth of the jet grouted columns is to be determined. The shearing stress loading the treated soil columns as well as the necessary shear strength of these are to be determined from the rotative moment which acts against the movement of the soil mass along the sliding plane marked by 1.

In our case the stability calculations were made as it was mentioned above, and as a result of those 5 MPa compression strength was prescribed for the grouted soil, referred to cube strength determined on 28 days old cubes of 20x20x20 cm.

To meet design requirements, 8 soil column tests were prepared by jet grouting. The grouting mixture had an 1:1 water-cement ratio and contained some bentonite up to 1,7% of the actual cement weight. The test piles (grouted bodies) were excavated and both axial and radial cylindrical samples of 15 cm diameter were taken from those. The average value of compression strength obtained from these test samples figured 14.79 MPa. No characteristic difference was observed between axial and radial compression strength values.

(The average compression strength of axial samples showed 14,69 MPa while the result of the radial samples was 14,95 MPa.)

EXECUTION OF THE FOUNDATION REINFORCING

On the construction site an automatized and containerized mixing plant type GM-7 of Soil Mec was used for preparation of the grout.

The grouting material was forwarded by GEOASTRA 4T250 high pressure pump to the soil through the jet-monitor, mounted on an SM 305 drilling rig. The grouted material was a water-cement mixture to 1:1. Bentonite was added only to the inclined soil columns in order, to retard the settlement of the grout itself and to promote forming of a better joint between the treated soil columns and the brick-stone mixed wall construction, to avoid any settlement of the reinforced foundations during excavation of the new foundation pit.

On the average 300 kg of cement was used for the treatment of a one meter long soil column. About 30% of this amount filled out the unexpected underground holes.

The monitor was rotated by 15-20 rpm and the lifting speed of it was around 30-35 cm/min in the gravel and 20-25 cm/min in the sand areas. The suitable diameter of the jets was 2.0 mm in sandy gravel and 1,8 mm in sand or silty sand soil.

During the jet grouting of the inclined treated soil columns it happened two times that the injection pressure caused cracks of the concrete floor pavement of the cellar at 14. Vadász street building. Further cracks were avoided by short interruptions of the grouting process. At that locations where the grouting holes were located directly beside the foundations it was important to take care of the high pressure injection of the upper 50 cm of the columns, because sometimes it happened that the high pressure flow of the grout moved out some pieces of the original foundation where it was found in a very bad condition.

During jet grouting the horizontal and vertical displacements of the surrounding buildings were observed to prevent the development of dangerous situations.

Finally it was determined that all the settlements of the surrounding buildings remained less than 10 mm, which wasn't dangerous at all.

SUMMARY

The works detailed here proved the greatest advantage of the jet grouting technology which was in this case the following:

It succeeded to carry out a suitable foundation reinforcing in a sandy and gravel soil mixed with building rubbles, without any subsurface preparation or excavation activity and without causing any loosening of the soil and any damage of the existing buildings located close around the new foundation pit. The applied technology also ensured absolute stability of the walls of the foundation pit during the whole building construction period.

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Verfestigung des Tiefkellers und Fundaments des Gebäudes unter
Budapest, I, Petermann biró 4

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Einleitung

Der Burgberg im Teil Buda von Budapest ist ein ausragender Wert, eine weltberühmte Denkmalgesamtheit. Der Schutz und die Instandhaltung davon beansprucht eine gesteigerte Aufmerksamkeit.

Das Höhlensystem unter den historischen Gebäuden und Wohnhäusern des Burgbergs, sowie die wässrigen Versorgungsleitungen in ihrer Umgebung verfügen über eine wesentliche Wirkung auf den Zustand der Oberflächenanlagen.

In der ersten Hälfte der 80-er Jahre hat man die Erneuerung der Wohnhäuser der Budaer Burg, der Zivilstadt vorgenommen, im Lauf deren wurden mehrere Wohnhäuser rekonstruiert. Der Grund der Beschädigungen konnte in mehreren Fällen zum Fehlerhaftwerden der wässerigen Versorgungsleitungen oder zur Zustandsverderbung des Tiefkellers unter dem Gebäude zurückgeführt werden.

Im Rahmen der Rekonstruktion der Wohnnausgesamtheit fand auf Grund des Auftrags von FIMÜV die Untersuchung, Projektierung, dann die Verfestigung des Gebäudes im Titel statt. Die Untersuchungen, weiterhin auf Grund dieser die Baupläne wurden von FTV 1982-83 ausgeführt.

Bekanntmachung des Gebäudes

Das Gebäude ist im nördlichen Teil des Burgviertels, in der Nähe des Platzes Bécsi Kapu, an der Ecke der Strassen Petermann biró und Országház zu finden. Das einstöckige Wohnhaus, geschlossen gebaut, mit einem Innenhof wurde um 1820 gebaut. Die jetzige Form wurde nach mehreren kleinen Umgestaltungen, mit der Eliminierung der Kriegsschäden 1950 erreicht.

Das Gebäude ist ein teilweise unterkellertes, einstöckiges Bauwerk mit hohem Dach, darunter befindet sich in der Tiefe von etwa 13-14 m ein Tiefkeller.

Das Gebäuden steht auf Streifenfundamenten. Der Überbau ist vom Längswandsystem, die Wände sind aus Ziegeln, die Decken sind am Erdgeschoss auf einer Strecke bömisches Kappengewölbe, an den anderen Teilen und am Stöckwerk eine Plattendecke. Der nordöstliche Teil des Gebäudes wurde zur Zeilt der Untersuchungen (1982) stark beschädigt, es waren auf Differentialsetzungen hinweisende Risse sichtbar.

Untersuchungen

Der Burgberg ist ein Terrasseninsel, durchschnittlich bis zur Höhe von 155-160 m ausgehoben und mit einem Süßwasserkalkstein- Überzug bedeckt. Die Grundmasse des Berges ist überwiegend ein Budaer Mergel mit einer Neigung in südwestlicher Richtung und untrgeordnet - im nördlichen Gebiet des Berges ein Ton von Kiscell, der sich einer Strukturlinie entlang berührt. Auf die - zum gleichen Niveau gewitterte - Oberfläche der zweiar-tigen Gesteinmasse wurde eine sandige, kiesige Schicht mit einer Stärke von 0,5 - 1,5 m und von Flusswasser-Herkunft gelagert. Die Deckschicht des ganzen Berges ist Süßwasserkalkstein von der Thermalquellen-Aktivität abgelagert, die Stärke erreicht die 10 - 15 m. Die Kalksteinoberfläche konnte im ursprünglichen Zustand in der Nähe der Oberfläche sein aber es hat sich im Lauf der mehrhundertjährigen Bauarbeiten stellenweise eine mehrere Meter starke Auffüllung geformt.

Nördlich von der Linie des Platzes Disz - unter der Bürgerstadt - sind - laut den Forschungen - die zeitweilig tätige Quellen durch schon im Lauf der früheren Tätigkeit sich präzipitierten Schichten aufgebrochen, mit dem Zustandebringen von Kleineren - grösseren Gängen, Höhlen- und Schlotsystemen.

Von der hydrologischen Hinsicht haben durch die Risse und Spalten der Kalksteinschicht in die Masse des Burgbergs von der Oberfläche bzw. aus den Versorgungsleitungen hineinsickernde Wässer eine Bedeutung. Eine Zusammenhängende Wassermenge ist nur im Spaltensystem des Budaer Mergels zu finden.

Laut den Grund- bzw. Bodenaufschlüssen ausgeführt im Lauf unserer Untersuchungen ist die Beschichtung und der Zustand des Untergrundes sehr hete-

rogen (sehr locker, ausgehöhlt, bzw. dicht). Die Oberfläche ist durch eine Sandauffüllung mit Gebäudeschutt der Stärke von 0,7 - 4,3 m gedeckt, worunter auch in variierender Stärke Schichten aus Sand mit Sandmehl, bzw. Sandmehl (stellenweise schlammig) findbar sind. Die Mehrheit der aufschliessenden Bohrungen hat in sehr unterschiedlichen Tiefen (0,7-9,0 m) die Oberfläche des Süsswasserkalksteins erreicht.

Einige Streifenfundamente stehen stellenweise auf sehr tragfähigen und dichten, bzw. auf tragfähigen und mitteldichten Schichten, hie und da direkt auf der Kalksteinschicht. An diesen Gebäudestrecken haben sich allerdings keine Schäden gemeldet. Anderswo war der Untergrund sehr locker und ausgehöhlt. Die Gebäudeschäden haben sich selbstverständlich hier gemeldet.

Es wurde kein Grundwasser in den Aufschlussbohrungen erreicht. Die starke Durchnässung beobachtet in den einzelnen Fundamentaufschlüssen lasste sich auf das Fehlerhaftwerden von Versorgungsleitungen folgern.

Unter dem Gebäude ist ein Tiefkeller zu finden. Das obere Zweidrittel des Treppenuntergangs ist mit Ziegelgewölbe gesichert, das untere Eindrittel hat eine freie Kalksteinoberfläche. Im Lauf unserer in situ Besichtigungen haben wir beim Treffen von Gewölbe und Kalkstein einen sehr intensiven (1,0 - 1,5 l/Minute) Wasserfluss erfahren. Zur Feststellung der Herkunft des Wassers haben wir eine Wasserprobe genommen, deren Chemische Analyse hat eine Herkunft aus der Druckleitung wahrscheinlich gemacht. Das gefärbte Wasser geleitet in die Kanäle des Gebäudes erschien im Keller nicht. Auch andere indirekte Beobachtungen haben einen Rohrbruch wahrscheinlich gemacht (Beobachtung des Wassergebrauchs, Rauschen, usw.)

Auf Grund obiger Untersuchungen wurde es festgestellt, dass der Untergrund des nordöstlichen Teils des Gebäudes sehr locker, stellenweise ausgehöhlt ist, was mit grosser Wahrscheinlichkeit vom grossmengenigen Wasser aus dem Rohrbruch verursacht wurde. Die am Gebäude beobachteten Beschädigungen wurden an diesem, auf sowieso einer Auffüllung stehenden Gebäudeteil ausgeformt. Die Risse und die Durchfeuchtung wurden auch in einigen Zweigen des Tiefkellers beobachtbar.

Unseren annähernden Berechnungen gemäss entstand - wegen der Auflockerung, bzw. Aushöhlung des Untergrundes - auch mit behutsamer Abschätzung eine

Extra-Setzung von 40-50 mm - an der beschädigten Gebäudestrecke. Dieser Setzungsunterschied war genug zur Ausformung der Setzungsrisse. Wir haben es an unseren mehrmaligen lokalen Besichtigungen erfahren und es wurde auch von den versetzten Gipsiegeln bewiesen, dass zur Zeit der Untersuchungen (Februar -Mai 1982) der Setzungsprozess noch nicht anhielte, die Risse haben sich augenscheinlich erweitert.

Es wurde von uns vorgeschlagen, dass zusammen mit der Ausbesserung des Versorgungsleitungsfehlers, der die Aushöhlung des Untergrundes verursacht, im Interesse des Anhaltens der Setzung des Gebäudes und der Behütung des Zustands des Gebäudes, die Fundamente des Gebäudes und die beschädigten Tiefkellerstrecken verfestigt werden müssen.

Verfestigungen

Allen Fundamentverfestigungsarbeiten müssen von der Überprüfung, bzw. Rekonstruktion der wässrigen Versorgungsleitungen vorbeugt werden, d.h. die Durchfeuchtung und Auflockerung des Untergrundes muss eliminiert werden.

An der Gebäudestrecke auf einer Auffüllung gegründet wurde eine Fundamentverfestigung, und zwar mit einer höhlenausfüllenden Bodenverfestigung vorgenommen. Die Bodenverfestigungsarbeiten wurden von der Fa BAV ausgeführt. Die Erdmasse zwischen der unregelmässigen Kalksteinoberfläche und der Gründungsebene wurde injiziert. Die Injektierung wurde von der Stärke der zu injektierenden Bodenschicht abhängig von unten nach oben in 2 - 4 Phasen durch ein Stahlrohr mit offenem Ende in 2-4 Phasen durchgeführt. Das Stahlrohr war nicht perforiert. Unter den einzelnen Phasen wurden die Injektierrohre ausgewaschen und der Phase gemäss 1,3-1,6 m nach oben gezogen, dann nach dem wiederholten Einbau und der Verfestigung der Halsdichtung wurde die Injektierung auch auf diesem höheren Niveau durchgeführt.

Die Zusammensetzung des Injektiermaterials war je 1 m³ die folgende:

1000 l	Wasser
250 kg	Hüttenportlandzement 350 (aus Vác)
55 kg	F2 Bentonit (plastifizierend und Sedimentation-hindernd)
100 kg	Sand mit einer Kornabmessung von 0,5-2,0 mm.

Auf Wirkung des Injektierungsdrucks (6-10 bar) ist der Boden in Richtung

des kleineren Bodenwiderstands aufgerissen und das Injektierungsmaterial hat die umgebenden Höhlen ausgefüllt, damit den Untergrund unter den Gründungskörpern verdichtend.

Als Ergänzung haben wir auch die Bodenschicht der Stärke von etwa 2 m unter der Fussbodenkonstruktion verfestigt. Der Injektierungsdruck war hier natürlich wesentlich kleiner, d.h. 2-3 bar. Die Materialzusammensetzung stimmte mit der früheren überein.

Die Bodenverfestigung kann - von der Natur folgend - schwer kontrolliert werden, das Ergebnis kann nur indirekt abgewogen sein, deswegen haben wir eine Arbeitsleitung des Projektants und die ausführliche Protokollaufnahme des Injektierungsprozesses vorgeschrieben.

Es wurde von uns auch gefordert, dass vor den Injektierungsarbeiten auch die beschädigten Kellerzweige verfestigt werden müssen.

Die Verfestigung wurde von der Sohlenbreite der Zweige abhängig mit einem Gewölbebogen gemauert aus einreihigen Betonformsteinen VA 20 und VA 25 durchgeführt. Wegen den engen Abmessungen wurde das beschädigte alte Ziegelgewölbe, natürlich beim Schutz einer provisorischen Unterstützung (Verbau mit Doppelbölzung mit vorabgesteckten Schwarten) entfernt. Die Hinterhöhle hinter der fertigen Mauer, bzw. dem Gewölbe muss durch die vorherig versetzten Injektierungsröhre mit Zementmörtel ausgefüllt werden.

Die oben beschriebenen Verfestigungs- weiterhin die Wiederherstellungs-Arbeiten des Überbaus wurden im Lauf von 1984-1986 fertiggestellt.

Zusammenfassung

Die oben erwähnte Aufgabe war ein gutes Beispiel dafür, dass die wichtige Mehrheit der Gebäudeschäden kann mit dem im Boden befindlichen oder dorthin kommenden Wasser in Verbindung gebracht werden. Im gegenwärtigen Fall hat das Druckleitungswasser den Gleichgewicht des Untergrundes aufgelöst, weil die Höhlen in der Tiefe haben das Auswaschen der feineren Bodenkörner und damit die Auflockerung des Untergrundes ermöglicht.

Am Gebäude sind gegenwärtig (1990) kleinere Haarrisse sichtbar.

Ein Teil dieser ist an der Stelle der früheren Beschädigungen sichtbar.

Diese Risse haben sich aber nicht geöffnet. Es kann angenommen werden, dass das Gebäude erst nach der Wiederherstellung des Überbaus in Ruhestand

kam. Diese Beschädigungen beanspruchen keinen Eingriff, ihre systematische Beobachtung ist ratsam.

Im Zusammenhang mit dem Gebäude wurde keine Setzungsmessung vorgehommen, weil die Untersuchungen haben die Notwendigkeit des Eingriffs eindeutig gemacht. Dem gegenüber wäre es zweckmässig gewesen, vor allem zur Kontrolle des Erfolgs der Gründungsverfestigung, eine Setzungsmessung durchzuführen.

Eine andere wichtige Erfahrung ist, dass bei Verfestigungsarbeiten solchen Charakters schon im Lauf der Projektierung eine Zusammenarbeit mit der Baufirma zustande gebracht werden soll. Im Lauf der Bauarbeit eine häufige Werkleitung des Projektants ist notwendig. Die enge Zusammenarbeit der Baufirma und des Projektants ist sehr wichtig, weil die Bodenverfestigung nur diese Weise beruhigend und mit entsprechendem Erfolg durchgeführt werden kann.

ZUM MECHANISMUS DER RÜCKSCHREITENDEN EROSION UNTER DAMMEN UND STAUWERKEN

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1 EINLEITUNG

Rückschreitende Erosion findet unter Dämmen und Stauwerken statt und äußert sich auf der Unterwasserseite zunächst durch sog. Quelltrichter (Bild 1 und 2). Aus diesen strömt Wasser nach oben aus, und im Falle kritischer Verhältnisse werden größere Mengen Bodenmaterial, und zwar zumeist Feinsand, ausgeworfen. Der erodierte Auswurf bildet unterhalb des Dammes progressiv einen zum Oberwasser hin gerichteten unterirdischen Kanal, dessen Wasserführung mit zunehmender Länge wächst und dessen Erosionswirkung ebenfalls steigt. Der Damm kann im Spätstadium in diese Erosionshohlräume hineinsacken, so daß dann die Krone überspült und der Damm endgültig zerstört werden kann, auch wenn das Hochwasser ursprünglich nicht das Niveau der Dammkrone erreicht hat (HAVLICEK

Bild 1: Erodierender Quelltrichter am Rhein



Niveau HHW

Dammprofil, verzerrt

rech. Energielinie ohne "Störung" durch Gellichter

Vorland

Binnenland

Deck-schicht

Aquifer

Bemessungslänge L_R

möglicher Erosionskanal

Bereich möglicher kritischer Quellschuttsysteme

Bereich un-kritischer Quellschuttsysteme

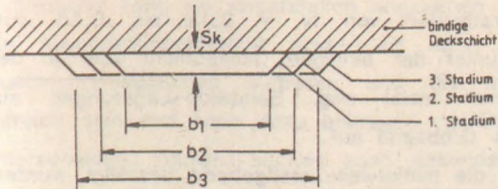
Die Kontrollgradientenmethode nach TSCHUGAJEW (1965) findet aufgrund ihrer Einfachheit in der heutigen Praxis eine breite Anwendung.

2 ERGEBNISSE

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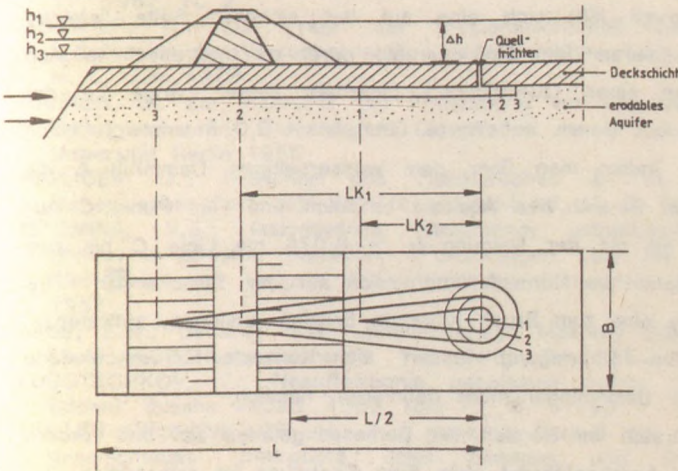
- Die rückschreitende Erosion unter Dämmen und Stauwerken (Bild 2) ist an die Existenz einer kohäsiven bzw. gegen Einstürzen des Kanals ausreichend stabilen Deckschicht gebunden.
- Der Querschnitt der Erosionskanäle entspricht nicht, wie auch heute noch vielfach angenommen, dem von Röhren. Es handelt sich vielmehr im mittleren Stadium, d. h. vor dem Durchbruch des Kanals zum Oberwasser, um einen quasi trapezförmigen Querschnitt, dessen Tiefe stets etwa eine gleichbleibende Größe S_K für alle Versuchsstadien und auch für unterschiedliche Versuchsmaßstäbe aufweist (Bild 3).

Bild 3: Kanalquerschnitte in verschiedenen Versuchsstadien



- In der Draufsicht haben die Erosionskanäle eine quasi spitzwinklige Form (Bild 4).
- Innerhalb der vorgenannten geometrischen Konfiguration verändert sich der Kanal nach jeder Gefälleerhöhung so lange, bis für das jeweilige Stadium der spitzwinklige Grundriß hergestellt ist. Im Frühstadium, d. h. solange die Kanalspitze weniger als etwa die halbe Distanz $L/2$ zwischen Quelltrichter und dem freien Oberwasser erreicht hat, kommt die Erosion nach jeder Gefälleerhöhung zunächst wieder zum Stillstand, der erst nach einer weiteren Gefälleerhöhung überwunden wird.

Bild 4: Geometrie der Erosionskanäle



- Aus der Tatsache, daß die Kanaltiefe S_k auch bei unterschiedlichen Maßstäben stets in der gleichen Größenordnung bleibt, sich die sonstigen Kanalabmessungen mit dem Gefälle nahezu linear vergrößern, ergibt sich über besondere Herleitungen, daß zumindest bis etwa $L/2$ die Fließgeschwindigkeit im Kanal etwa gleich bleibt, obwohl mit ΔH die Wassermenge zunimmt. Unter den genannten Voraussetzungen scheinen die experimentell ermittelten Ergebnisse insoweit größenordnungsmäßig im Proporz zum Längenmaßstab bzw. zur Niveaudifferenz ΔH auf die Natur übertragbar zu sein.

- Die bisherigen statistisch-empirischen Verfahren gehen davon aus, daß die gradientenbezogenen Standsicherheitskriterien jeweils auf nur eine Hauptbodenart zu beziehen sind. So geht die Methode von TSCHUGAJEW für feinen Sand generell von einem sog. Kontrollgradienten von $I_k = 0,12$ bis $0,16$ aus. Demgegenüber treten in der Natur unter der bindigen Deckschicht aber in den erosionsgefährdeten, oberen Bereichen meist sog. Sandwich-Lagerungen aus Grobschluff, Feinsand und Mittel- bis Grobsand auf.

Wie bereits frühere Untersuchungen, die mittlerweile weitgehend bestätigt wurden, ergaben, gelten für Feinsande, die in gröbere Sandwichlagen eingebettet sind bzw. von gröberen Böden unterlagert werden, bedeutend niedrigere Kontrollgradienten bis herab auf $I_k = 0,075$. Hierbei spielt das Verhältnis der geometrischen Schichtdicken der einzelnen Böden zueinander eine wesentliche Rolle. Nach unseren Erfahrungen sollte man aber nach worst-case-calculation stets davon ausgehen, daß in der primären Erosionszone ungünstige Schichtdickenverhältnisse vorliegen.

- Es kann davon ausgegangen werden, daß der obengenannte Kontrollgradient $I_k = 0,075$ für den Beginn einer rückschreitenden Erosion im Normalfall nirgends unterschritten wird. Insoweit läßt sich eine auf der sicheren Seite liegende Bemessungslänge L_R definieren (Bild 2), innerhalb derer es theoretisch möglich ist, daß durch Auftreten eines Quelltrichters innerhalb dieser Länge L_R der mittlere Gradient unter dem Damm auf Werte über $I_k = 0,075$ ansteigt. Nach Bild 2 ergibt sich L_R , indem man über dem wasserseitigen Dammfuß A die Energiehöhe h_1 aus dem Bereich des Aquifers bestimmt und vom Punkt C aus nach der Brunnenseite hin mit der Neigung $I_k = 0,075$ die Linie C bis zum Punkt B führt. Dann können im Normalfall nur noch auf der Strecke $\overline{AB} = L_R$ kritische Quelltrichter, die eine zum Bruch führende Erosion auslösen, auftreten.

- Mittels der vorgenannten Überlegung lassen sich Konzepte für verschiedene Gefährdungsklassen eines Dammbinnenlandes definieren, nämlich:

Klasse A: Quelltrichter treten im Bereich der Bemessungslänge auf und fördern Sand. Die Situation kann bruchgefährdet sein. Eine Sanierung ist erforderlich.

Klasse B: Keine Quelltrichter im Bereich der Bemessungslänge. Treten sie dennoch auf, so ergibt sich Klasse A. Eine Sicherung gegen das Auftreten von Quelltrichtern bzw. besondere Beobachtung ist erforderlich.

Klasse C: Es treten Quelltrichter binnenseitig der Bemessungslänge auf, die Sand fördern. Der Sandauswurf entstammt im wesentlichen dem Bereich der primären Erosionslinse (MÖLLER-KIRCHENBAUER 1978, MÖLLER-KIRCHENBAUER und HANSES 1981, HANSES 1985), jedoch kommt es nicht mehr zum progressiven Weiterwandern des Kanals über das Anfangsstadium hinaus. Beobachtung der Situation ist empfehlenswert.

Klasse D: Es treten binnenseitig der Bemessungslänge Quelltrichter nahezu ohne Sandauswurf auf, die nur noch Wasser ohne Feststoffanteile fördern. Diese Situation kann als ungefährlich angesehen werden.

Ein Klassifikationskonzept obiger Art kann in Flußbereichen, in denen jahrelang Quelltrichtersituationen auftreten und die man folglich zu kennen glaubt, sehr hilfreich sein und kann dazu beitragen, gefährliche von ungefährlichen Bereichen zu unterscheiden. Insoweit können auch besonders zu untersuchende und besonders zu sanierende Bereiche unterschieden werden, was den gezielteren Einsatz der meist begrenzten Erkundungs- und Sanierungsgelder erlaubt.

Die Ergebnisse unserer Arbeiten konnten im vorliegenden Rahmen nur knapp behandelt werden. Wesentlich ausführlichere Berichte erscheinen demnächst.

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Verformungen von Bewehrte-Erde-Bauwerken

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1. Einleitung

Die Bewehrte Erde stellt ein in der Geotechnik relativ neuartiges Verfahren zur Verbesserung der bodenmechanischen Eigenschaften dar. Es ging dabei anfangs primär um die Erzeugung einer künstlichen Kohäsion. Die Veränderung der Verformungseigenschaften wurde als sekundärer Effekt meist nur positiv vermerkt. Im Falle von analytischen Untersuchungen mit Hilfe der Methode der finiten Elemente (FEM) gelang es, die Verformungsvorgänge abhängig von gewählten Stoffmodellen mehr oder weniger zutreffend zu beschreiben /1/ /2/ /3/. Diese Untersuchungen sind jedoch wegen ihres hohen Aufwandes für baupraktische Anwendungen wenig geeignet.

Mit diesem Beitrag sollen Näherungen vorgestellt werden, die quantitative Aussagen über die Nutzungsfähigkeit bewehrter Erdbauwerke gestatten und mit denen die Plausibilität von FEM-Berechnungen kontrolliert werden kann. Außerdem ermöglichen diese Näherungen eine kritische Einschätzung von modelltechnisch beobachteten Effekten des Verformungsverhaltens /4/ /5/.

2. Künstliche Querisotropie

Ein elastischer Halbraum, der richtungstreu mit Bewehrungen durchsetzt ist, kann näherungsweise als künstlich querisotrop betrachtet werden. Das Modell der Querisotropie, analytisch bereits von JELINEK /6/ untersucht, wurde auch zur Berechnung der Spannungsausbreitung in einem Halbraum aus Bewehrter Erde benutzt /7/. Charakteristisch für die künstliche Querisotropie

ist jedoch, daß die Schubverzerrungen unabhängig von der Dehnsteifigkeit der Bewehrung sind.

Die Gleichgewichts- und Verträglichkeitsbedingungen an dem in Bild 1 dargestellten differentiellen Element führen für den ebenen Fall zur Bildung einer Bipotentialgleichung. Anhand einer Grenzwertanalyse für das Spannungsfeld kann man zeigen, daß sich die Vertikalspannungsverteilung nur in einem kleinen Intervall in Abhängigkeit von der Bewehrungsdehnsteifigkeit ändert. /7/.

Wie in Bild 2 ersichtlich, nähert sich das Verhältnis der inneren Horizontalspannung $\sigma_{yq}^{(1)}(z)$ zur Vertikalspannung $\sigma_{zq}(z)$ mit zunehmender Dehnsteifigkeit der Bewehrung dem Ruhedruckbeiwert. Die Verteilung der Bewehrungskraft $T(z)$ ist der Spaltzugverteilung im Stahlbeton ähnlich. Der Aufbau von zusätzlichen inneren Horizontalspannungen vermindert so die deviatorische Beanspruchung des Verfüllmaterials. Eine hinreichend zutreffende Setzungsberechnung für Gründungen auf aufgefülltem nichtbindigem Material ist folglich erst bei genügender Steifigkeit der Bewehrung im Lasteintragungsbereich möglich.

Die in Bild 2 dargestellte Charakteristik der Spannungsverteilung bestätigt die Anwendbarkeit des von SZALATKAY /8/ vorgeschlagenen Verfahrens zur Setzungsberechnung, aber auch die praktische Regel, wonach es ausreicht, nur den Bereich möglicher Plastizierungen zu bewehren.

Als Näherung kann die innere Spannung nach Gl. (1) ermittelt werden.

$$\sigma_{yq}^{(1)}(z) = \sigma_{zq} \frac{\nu(1+\nu)}{\frac{E \Delta H}{D_B} + 1 - \nu^2} + \sigma_{yq} \frac{E \Delta H}{E \Delta H + (1 - \nu^2) D_B} \quad (1)$$

mit:

- E - Verformungsmodul der Verfüllung
- ΔH - vertikaler Bewehrungsabstand
- D_B - Dehnsteifigkeit der Bewehrungslage je m
- ν - Querdehnzahl der Verfüllung
- σ_{yq}, σ_{zq} - Zusatzspannungen infolge der Auflast q nach BOUSSINESQUE.

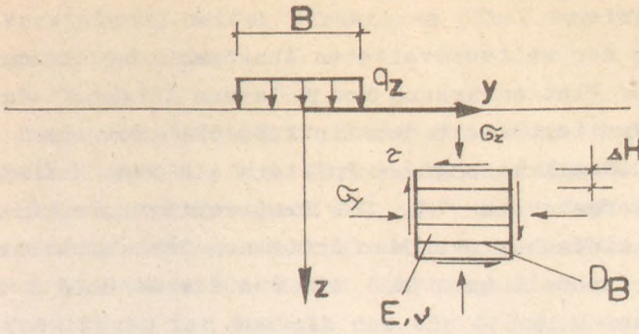


Bild 1 Differentielles Element in der künstlich-
querisotropen Halbebene

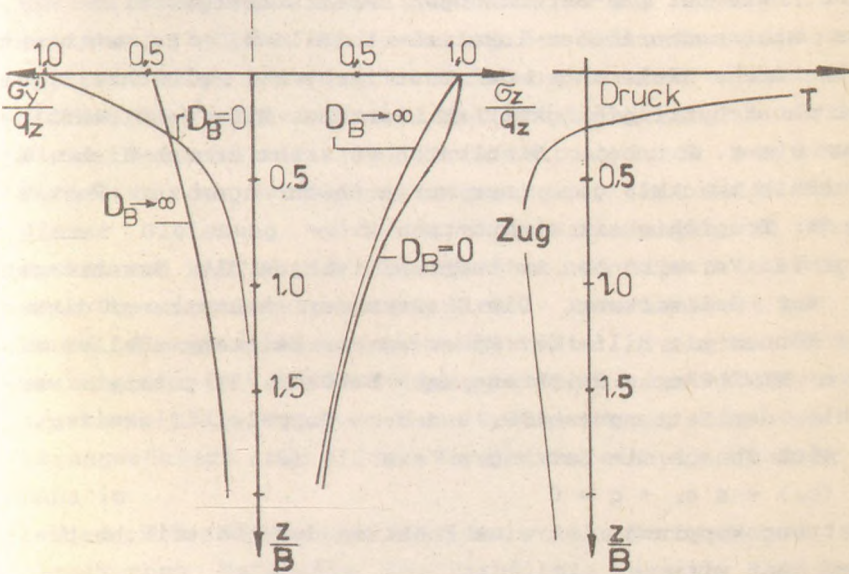


Bild 2 Spannungsverteilung- und Bewehrungskraft in der
Achse einer Streifenlast

3. Gründungspolster

Zur Überprüfung der weitverbreiteten Annahme, daß horizontale Bewehrungen zur Plattenwirkung von Polstern führen, wurde im Interesse der Variierbarkeit der Einflußgrößen bei geringster Rechenzeit die Idealisierung des Polsters als Stab auf elastischer Bettung vorgenommen /7/. Die Programmierung berücksichtigte ein sehr einfaches, von der örtlichen Schersicherheit γ abhängiges Stoffmodell nach Gl. (2) und die im Bild 3 dargestellte Struktur.

$$E_t = E_0 (1 - 1/\gamma) \quad (2)$$

Mit der inkrementellen Berechnung konnte das nichtlineare Verformungsverhalten des Polsters trotz der Annahme der linearen Bettungsreaktion und der Hypothese vom Ebenbleiben der Querschnitte nachgewiesen werden. Die Plastizierungsbereiche sind, wie bei FEM-Berechnungen bereits festgestellt /1/, in den Lastrandbereichen lokalisiert (Bild 4). Es entsteht eine elastische Säule unter der Lastfläche bei gleichzeitigem Verlust der Schubtragfähigkeit am Lastrand. Eine gewölbeartige Struktur wie z. B. beim Stahlbeton entsteht danach nicht. Es ist deshalb zweckmäßiger, nur kurze Bewehrungen zur Verbesserung der Tragfähigkeit einzusetzen.

Die großen Verzerrungen am Lastrand zwingen die Bewehrungen jedoch zur Seilwirkung. Die Grenzen der Ausnutzung dieses Effekts können mit Hilfe der Näherung des Halbraummodelles als ergänzte WINKLERSche Halbebene nach BALDAUF /9/ gezeigt werden. Mit der Bettungszahl c_z und der Koppelsteifigkeit c_p ergibt sich danach die Setzung s aus Gl. (3).

$$s''(c_p) - s c_z + q = 0 \quad (3)$$

Die Bettungskopplung ist eine Funktion der Lastflächenbreite B und kann mit

$$c_p \approx 0,352 B E \quad (4)$$

abgeschätzt werden /7/.

Eine ähnliche Kopplung ergibt sich auch, wenn die Oberflächenmembran mit der WINKLERSchen Bettung verbunden wird. Es ergibt sich eine zur Gleichung (3) ähnliche Differentialgleichung, wenn in dem in Bild 4 dargestellten undehnbaren Membranstreifen eine konstante horizontale Kraft T_H vorausgesetzt wird.

Die Vereinigung beider Strukturen führt zur Bildung der folgenden Gleichung:

$$s'' (c_p + T_H) - s c_z + q = 0 . \quad (5)$$

Die Membrankraft bewirkt somit eine Vergrößerung der Verzerungssteifigkeit. Die Verzerrungssteifigkeit des Halbraumes ist jedoch entsprechend Gl. (4) unter Bedingungen des Hochbaus unvergleichbar höher als die Größenordnung jener Kräfte, die von Membranen überhaupt übertragbar sind. Die Nutzung der Membranwirkung ist deshalb nur für Gründungen auf sehr weichem Untergrund bzw. bei sehr schmalen Lastflächen erfolgversprechend.

4. Verbund zwischen Bewehrung und Boden

Die Verbundbedingung der künstlichen Querisotropie ist bei kurzen Bewehrungen nicht erfüllbar. Es muß wegen der Scherver-schiebung bei der Mobilisierung der Reibungsspannungen zumindest mit einem elastischen Verbund gerechnet werden. Eine nach ROSMAN /10/ aufgestellte Differentialgleichung berücksichtigt diese Besonderheit. Bei der Ableitung wurde vom elastischen Verhalten der Verfüllung ausgegangen. Mit Hilfe der Lösung dieser Gleichung wurden Vergleichsrechnungen zu Modellversuchen von MARTIN /11/ durchgeführt. In Bild 5 ist die relativ gute Übereinstimmung ersichtlich. Bemerkenswert ist, daß die durch die gemessenen Bewehrungskräfte erzeugten Horizontalspannungen in der Verfüllung annähernd den Ruhedruckzustand ergaben. Die Annahme der Größenordnung für den Verankerungsschlupf war in der Vergleichsrechnung offenbar zu ungünstig.

Wird die Bewehrungskraft des vollen Verbundes zu der des elastischen Verbundes ins Verhältnis gesetzt, ergeben sich Faktoren, mit denen die Nachgiebigkeit des Verankerungsschlupfes bei Berechnung nach der Theorie der künstlichen Querisotropie näherungsweise berücksichtigt werden kann /7/.

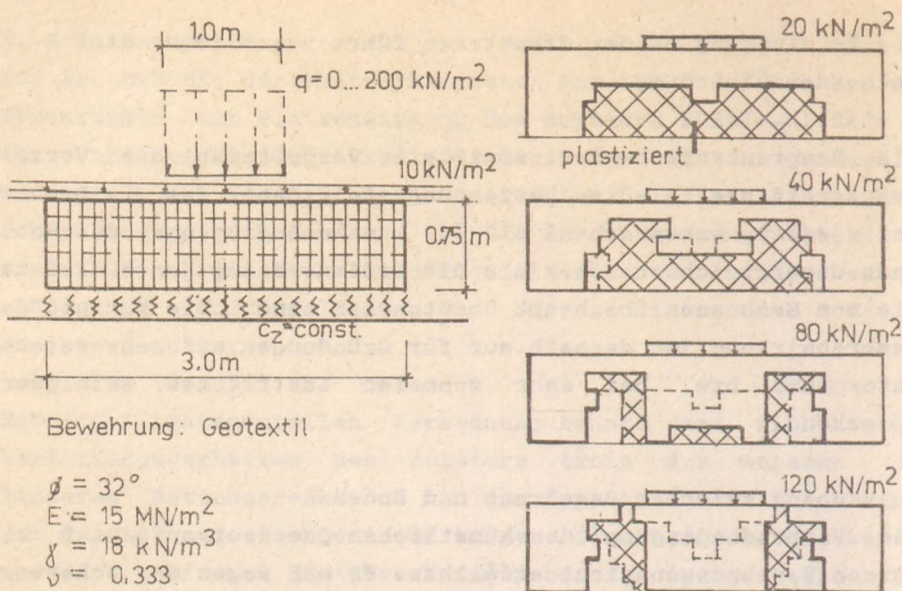


Bild 3 Strukturierung des Polsters als Stab auf elastisch Bettung und Plastizierungsbereiche

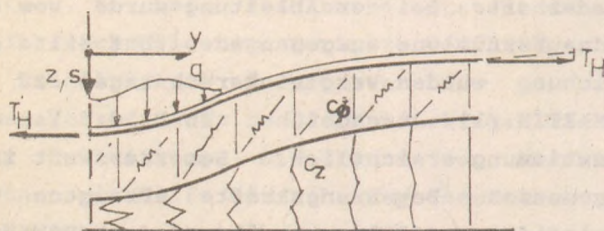


Bild 4 Membranwirkung der Bewehrung im Polster

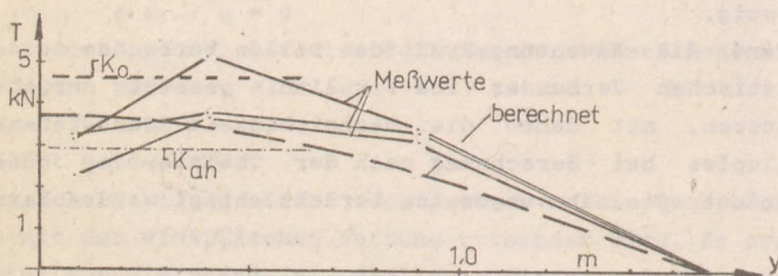


Bild 5 Bewehrungskraftverteilung

5. Stützbauwerke

Die Horizontalverformungen von Stützbauwerken sind nicht nur aus Verankerungsschlupf und Dehnung der Bewehrungen reproduzierbar. Auch der Hinterfüllungsdruck hat an der Verformung maßgeblichen Anteil.

Die von SZALATKAY /12/ vorgeschlagene Methode, wonach das Stützbauwerk als ein vom Hinterfüllungsdruck belasteter schubelastischer Kragträger betrachtet werden kann, wurde mit Hilfe von Analogiebetrachtungen zum elastischen Halbscheibenstreifen sowie einer FEM-Analyse von GIROD/TEICHMANN /1/ überprüft und hat sich bestätigt. Danach entstehen ca. 75 % der Horizontalverformungen nicht aus Biegung. Wird die Vertikaldehnung der Luftseite des Stützbauwerkes z. B. durch eine dehnsteife und rückseitig rauhe Außenhaut verhindert, werden bis zu 76 % der Biegeverformungen kompensiert. Die Verformungen sind also primär eine Folge von Schubverzerrungen. Die Analogiebetrachtung ergab ferner, daß der wirksame Querschnitt des Ersatzstabes der Struktur annähernd der Bauwerkshöhe entspricht.

Die Untersuchung des Einflusses der Außenhaut auf den Spannungs- und Verformungszustand ergab, daß dieses Bauwerkselement bisher in seiner Wirkung unterschätzt wurde. So wird bei genügender Rauigkeit die Vertikalspannung so weit reduziert, daß die Bewehrungskräfte an den Anschlüssen geringer sind als im Bauwerksinnern, ohne daß zwangsläufig Gleitflächen vorhanden sein müssen /7/. Die Verhinderung von Relativverschiebungen zwischen Außenhaut und Verfüllung ist andererseits auch entscheidend für die Zuverlässigkeit der Bewehrungsanschlüsse. Auflagerbänke sollten deshalb auf der Bauwerksoberfläche so gelagert werden, daß die Außenhaut sich mitsetzen kann.

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A Technical Effective, Economical and Ecological Structure
for supporting of an Urban area in Sofia

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I. GENERAL NOTES

Both the general reconstruction of the northeast in-out arterial road of Sofia boulevard "Botevgradsko shosse" and the enlargement of "Poduene" station in the north direction, required carrying out a project for a retaining wall, supporting the railway and having immense dimensions for such an urban area.

The retaining wall is about 460 m in length, the 225 m of which in eastern direction, and the other 235 m are westward and along with the subway within the station region form the entrance to the central part of Sofia.

The retaining wall is situated in a straight line so that the top of the wall is horizontal and the foundations come after the terrain. The height of the wall varies from 6.5 m in its highest part to 1.2 m in its lowest part. The foundation is in sand-clay layer with organic particles.

The structure of the wall allows some deformations of the foundation without any effect on the walls face.

Structural precautions are to be taken against the water access to the foundation-drain and plastic clay screen (fig.1).

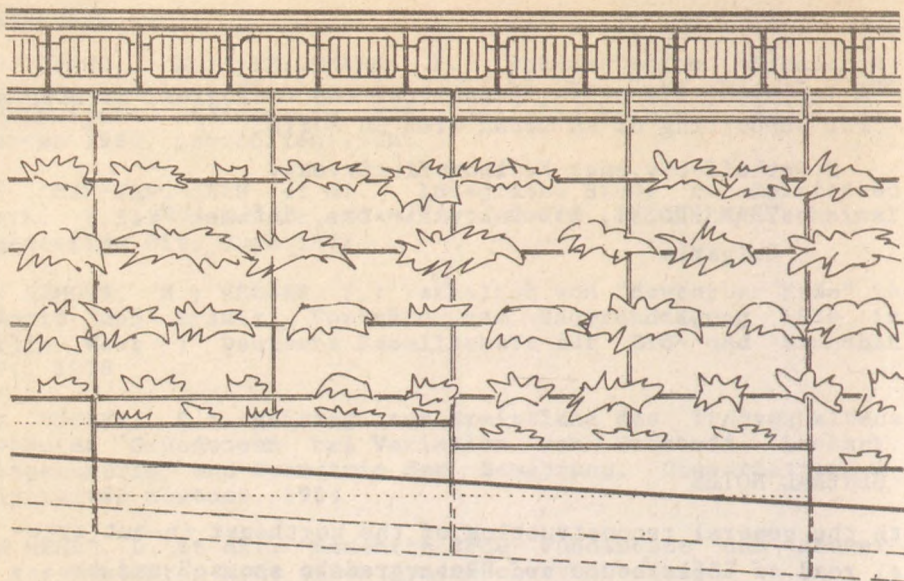
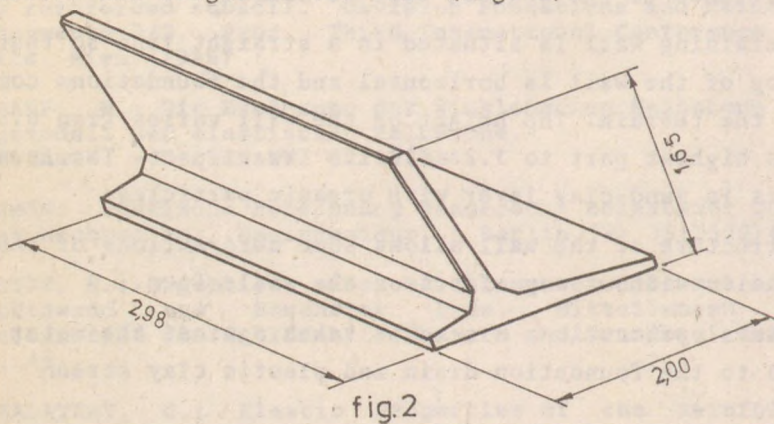


fig.1

II. STRUCTURAL SOLUTION

The wall consists of foundation cast in situ and prefabricated reinforced angle shaped elements, which are fitted in a stepped way. Every element has dimensions of 200 x 165 x 300 centimetres and weight of 4.7 tons (fig. 2).



The space between the elements is well filled up with embankment from compacted bank gravel which serves as a foundation to the upper element that is laid out (fig. 3).

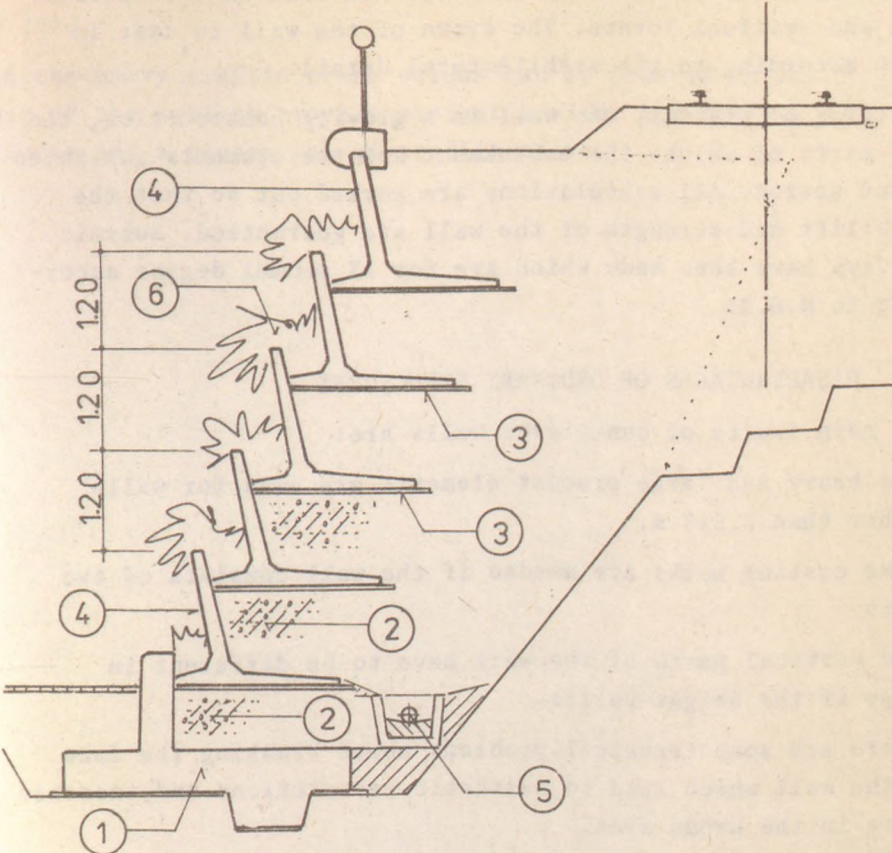


fig.3

- | | | | |
|---|-------------------------------|---|-----------------------------------|
| 1 | foundation cast in situ | 2 | bank gravel |
| 3 | poor concrete | 4 | prefabricated reinforced elements |
| 5 | drain and plastic clay screen | 6 | shrubs |

The highest part of the wall consists of 5 elements.

Poor concret is cast in situ on the embankment under every range of elements. The elements are not cast in the horizontal and vertical joints. The crown of the wall is cast in situ according to the architectural detail.

In terms of statics, the wall is a gravity construction, the two parts of which- the embankment and the elements guarantee joint action. All calculations are worked out so that the stability and strength of the wall are guaranteed. Seismic surveys have been made which are for IX seismic degree according to M.S.K.

III. DISADVANTAGES OF ORDINARY STRUCTURES

The main faults of cantilever walls are:

- too heavy and large precast elements are used for walls higher than 2.5+3 m.
- some casting works are needed if the wall consists of two parts
- the vertical parts of the wall have to be different in shape if the height varies.
- there are some technical problems about breaking the face of the wall which lead to aesthetic restrictions and increase noise in the urban area.

IV. ADVANTAGES OF THE NEW STRUCTURE

The New Structure offers the following main advantages:

- the structure can be built up of identical elements in spite of the variable height of the wall.
- the elements are comparatively small in shape- 200 x 165 x300 cm and in weight- 4.7 tons, which allows the use of all kinds of lifting machinery.
- casting works on the site are not needed
- the inclination of the wall to the embankment reduces the

efforts from the ground pressure so that the concrete and reinforcement are reduced by 20%.

-the drain materials for the embankment are also reduced by 20%.

-in the heavy traffic areas shrubs can be planted on the face of the wall to provide easthetic view, ecological effects and a protection of traffic noise.

THE PHOSPHOGYPSUM COMPOSITE AND ITS APPLICATION

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The research on utilization of the waste phosphogypsum from fertilizer industry have been recently carried out in the Institute of Hydroengineering of Polish Academy of Sciences in Gdańsk /1/. The observations indicating a possibility of using the phosphogypsum as a good coagulant agent in the process of silicatization of soluble sodium silicates were the reason for these works.

The interest in the phosphogypsum was also caused by the intention of utilization of so onerous waste for the environment. The purpose of the research was to describe the technology of the production of the material which would be useful and at the same time to remove its noxious properties.

The properties of the new received material show that it can be widely applied as a suitable building material for sealing the storage and waste reservoirs and for stabilizing the existing dumps of the phosphogypsum and also for protecting the environment from these dumps.

This material is called The Phosphogypsum Composite. It is a compacted mixture of solid materials such as the phosphogypsum, the fly-ash, the hydrated lime and the soluble sodium silicates in the quantities which allow for the maximum compaction of these mixtures.

2. THE MATERIALS AND THE TECHNOLOGY

The phosphogypsum, the fly-ash, the hydrated lime and soluble sodium silicates are the materials that are being used for the production of the Phosphogypsum Composite. Only the phosphogypsum from Gdańsk Fertilizer Factory was used in this research. The technology of the production of the fertilizer in this factory caused that the main waste product / above 75% / is the dihydrate calcium sulfate.

The phosphogypsum used in these investigations was preliminary modified in two ways. In the first, the phosphogypsum was preliminary dried in the room temperature and in the second, the phosphogypsum was heated in the 200°C. The heating in this high temperature caused the transition of the dihydrate of the calcium sulfate / $\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$ / to the mixture of two components: a halfhydrate of the calcium sulfate and to an anhydrous form - the anhydrite / $\text{CaSO}_4 \cdot \text{III}$ /.

The heating of the phosphogypsum in 200°C causes also the series of the changes in its physico-chemical properties, for example in the pH value and the solubility in water. The table 1 shows the comparison of the selected physico-chemical properties of the two mentioned above types of the phosphogypsum /2/.

Table 1

Property	Unit	The phosphogypsum dried in room temperature/20°C/	The phosphogypsum heated in 200°C
pH value		2.5	3.5
Solubility in water in 20°C	g/dm ³	2.0	8.9
Content of CaSO_4	%	79.1	93.8

The phosphogypsum dried in the room temperature plays in the process of the silicates setting in the Composite a role of practically filling material. Only the small quantities of the dissolved acids may play role of a coagulant. However, the phosphogypsum heated in 200°C actively participate in the formation process of the crystallic structure of the new material.

Two kinds of fly-ashes were used as a second component of the Composite: the sulfate-calcium fly-ash from the brown coal and the alumino-silicate fly-ash from the hard coal.

These two fly-ashes have quite different chemical characteristics /3/. The hydrated lime was applied as a modifier. It was used for the adequate alkalization of the Composite. The forming of the Composite consists in the accurate mixing of the dried, pulverized solid components, in adding the adequate solution of the soluble silicates, and then in remixing and finally in compaction of the mixture. The solutions of the soluble sodium silicates were applied in the densities ranging from 1070 kg/m³ to 1300 kg/m³. For the economic reason the optimum solution has the density equal to 1070 kg/m³. All presented results refer to the Composites formed with such a solution. The physico-mechanical were determined for two sizes of cylindric samples with the diameter and the height equal 5 cm and 8 cm respectively. The samples were stored both in the constant maximum humidity / constant humidity chamber / and in the suitable conditions for the gradual drying / the laboratory room /.

3. THE MAIN PHYSICO-MECHANICAL PROPERTIES OF THE PHOSPHO-GYPSUM COMPOSITE.

Having in mind the practical application, the static water-stability, resistance to frost, unconfined compressive strength / Rs / and the coefficient of permeability are the main physico-mechanical properties of the Phosphogypsum Composite. The unconfined compressive strength is the most interesting property and at the same time it shows in the

most vivid way the efficacy of the Composite silicization. The influence of the initial content of the Composite components on that strength presents a good example of the above. The content of the Composite influences the rest of the above mentioned properties i.e. static water stability, resistance to frost and the coefficient of permeability, to the very small degree. All samples, for which the unconfined compressive strength is presented below, exhibited high static water-stability and resistance to frost and their coefficient of permeability varied from 10^{-9} to 10^{-11} m/s.

A type of the phosphogypsum and the fly-ash used for the Composite, a quantitative ratio of the components, the method of sample storage, chemical properties, pH value, the total alkalinity of the mixture of the solid components these are the most important agents influencing the mechanical properties of the Phosphogypsum Composite. The figures 1-4 show these relationships.

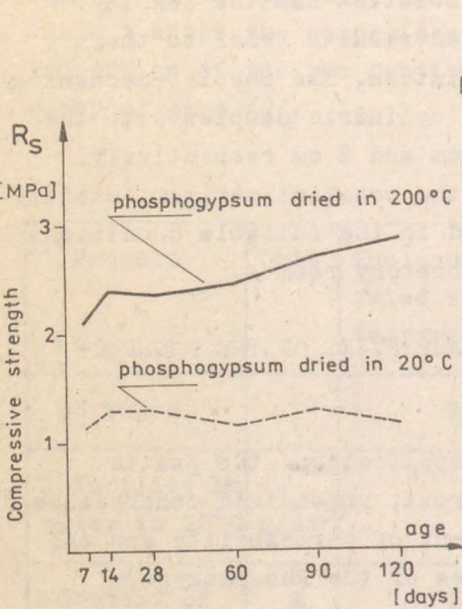


Fig. 1

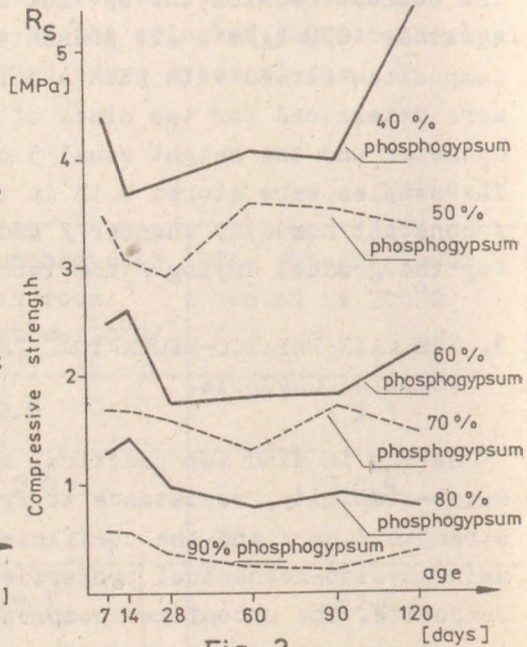


Fig. 2

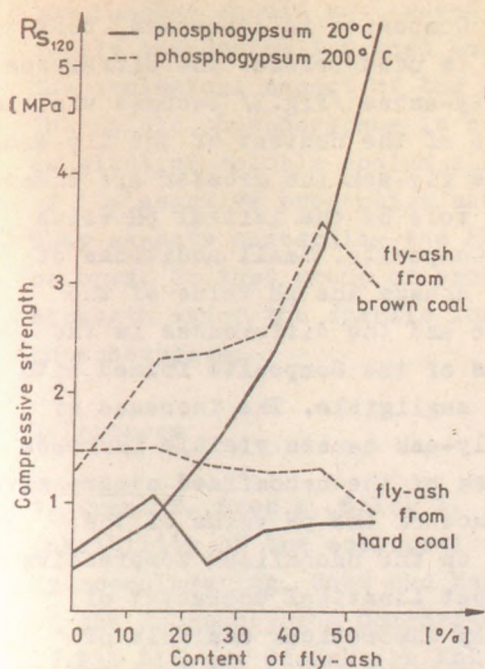


Fig. 3

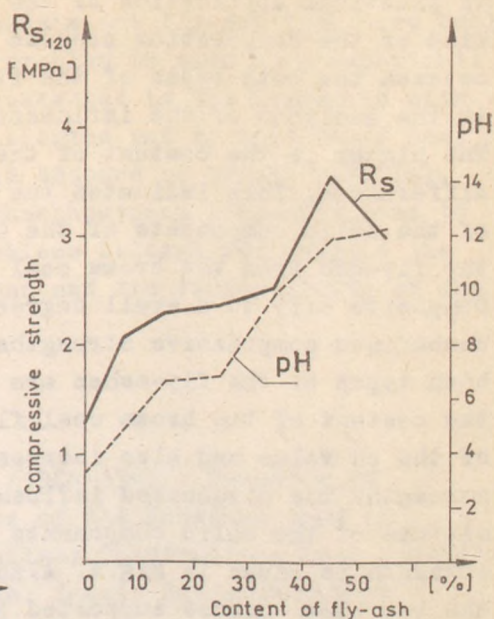


Fig. 4

Many important conclusions from both a scientific and a practical points of view, may be drawn from the research. The drying of the phosphogypsum in 200°C has visible influence on the properties of the Composite. The Composite which contains this type of the phosphogypsum has the unconfined compressive strength two times higher than that with the phosphogypsum dried in 20°C / Fig.1 /.

When the content of the phosphogypsum increases the value of the unconfined compressive strength decreases. The content of 40% of the phosphogypsum / Fig.1 / is regarded as the optimum content. These remarks concern both the phosphogypsum dried in 20°C and the phosphogypsum heated in 200°C. The unconfined compressive strength depends on the pH value. The content of the phosphogypsum in the Composite greater than 60% causes the decrease of the pH value of the mixture of solid components below pH=7, and in a consequence changes the character of the coagulation process of the soluble sodium silicates.

If practical application of the Composite is concerned, that kind of the coagulation process is undesirable. The difference between the both types of the fly-ashes /Fig.3/ becomes visible in the analysis of the influence of the content of the fly-ash. The higher is the content of the fly-ash the greater are these differences. This indicates the role of the initial pH value of the solid components of the Composite. Small additions of the fly-ash from the brown coal change the pH value of the Composite only to a small degree and the differences in the unconfined compressive strengths of the Composite formed with both types of the fly-ashes are negligible. The increase of the content of the brown coal fly-ash causes visible increase of the pH value and also increase of the unconfined compressive strength. The discussed influence of the pH value of the mixture of the solid components on the unconfined compressive strength is shown in Fig.4. Almost identical character of the two shown curves supported by theoretical analysis of the phenomenon permits to state without any doubt that the presented argumentation is correct.

4. THE FINAL CONCLUSIONS

The Phosphogypsum Composite satisfies the requirements for the new building material. It is characterized by the sufficient strength, small water permeability and also exhibits good static water stability and resistance to frost. As a result of the combined physico-mechanical and chemical tests it was proved that the Composites with the pH value between 9 and 12 have the acceptable parameters of strength. This parameter is a basic factor in prediction of the physico-mechanical properties of the Composite. To receive the required pH value of the mixture of the solid components, the addition of the modifier should be often made. It must be the material of the type of hydrated lime / the presence of the Calcium ions seems to be necessary /. A water content is another parameter / not mentioned up to now / that plays an important role in the Composite. The weight loss as an effect of the evaporation of the water during the storage of

the samples should not exceed 25% . This remark concerns mainly the Composites that contain the fly-ash from hard coal. The ecological aspect of the problem is also very important. The acidic phosphogypsum is connected in the Composite with an alkaline soluble sodium silicates and by this loses many of its negative properties and becomes an inert material. Many aspects concerning the Phosphogypsum Composite seem to be open. To that group of problems belongs for example the strength under the dynamic load and the radioactivity of the phosphogypsum.

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2. Quant B.,Knop M.,Sułek Z. /1989/. The influence of the type of the phosphogypsum and the fly-ash on the physico-mechanical properties of the Phosphogypsum Composite.
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Kriterien zur bodenmechanischen Bewertung der Basisabdichtung
von Mülldeponien auf kompressiblem Untergrund
(Zusammenfassung bisheriger Forschungsergebnisse)

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ZUSAMMENFASSUNG: Die bei Modellversuchen gewonnenen Erkenntnisse über das Dehnungsverhalten von Lösslehm- und Tonböden mit und ohne geotextiler Bewehrung werden mit Ergebnissen großmaßstäblicher Feldversuche auf kompressiblem inhomogenem Untergrund verglichen.

Die Vergrößerung der Dehnfestigkeit bei eingelegtem Geotextil soll bestimmt werden, um die infolge ungleichmäßiger Setzung entstehende Reißgefahr in mineralischen Basisabdichtungen (oder anderen setzungsgefährdeten Bodenkonstruktionen) zu verringern. Ziel der laufenden Untersuchungen ist es, die Verformungs-Risiken des Baugrundes unter mineralischen Basisabdichtungen durch wissenschaftlich begründete Ausführungsvorschläge zu minimieren.

SUMMARY: Nowadays experimental evidence and experience suggest that the performance of earth fill structures can be enhanced with the use of geotextiles. The deformation of a cohesive soil fill can, even under severer conditions, be limited via proper compaction practice in conjunction with the use of suitable geotextile reinforcing layers, thereby maintaining design shear strength levels.

This paper reports on an ongoing study which address the influence of geotextile reinforcement on the deformation behaviour of clay and loess liners. The ultimate goal of this study is to develop guidelines, based on sound scientific research, for the design, construction and site control of clay liners supported by compressible subsoils.

1. Einleitung:

Auf Grund von unregelmäßig gelagertem Baugrund werden z.Bsp. der Basisabdichtung von Mülldeponien unterschiedliche Setzungen aufgezwungen, die zum einem im extremen Fall der lokalen Karstbildung, Dolinen etc., zum anderen durch nadelähnlich

aufgehende Felshorizonte zur Zerstörung des Dichtungssystems führen können. Das Paket der Basisabdichtung wird über dem geöffneten Hohlraum bzw. der Felsnadel stark durch Biegung deformiert. Die dabei auftretenden Zugkräfte können weder von der eingebauten mineralischen Dichtungsschicht noch von der Kunststoffdichtungsbahn (zerstörungsfrei!) aufgenommen werden (s. Bild 1).

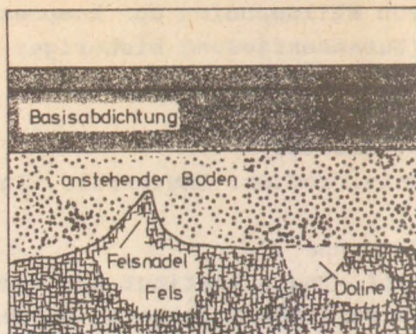


Bild 1: Prinzipskizze Deponieuntergrund

2. Kleinmaßstäbliche Biegeversuche

Im Zuge der Beurteilung von bindigem Bodenmaterial für Deponieabdichtungen wurde eine spezielle Versuchsanordnung entwickelt, um Aufschluß über die Biegezugfestigkeit und die Grenzdehnung bindiger Böden im ungerissenen Zustand zu erhalten (s. Bild 2).

Die Biegezugfestigkeit soll Aussagen hinsichtlich der Tragfähigkeit, die Grenzdehnung hinsichtlich der Verformbarkeit des Bodens ermöglichen. Die Ermittlung dieser Kenngrößen wurde sowohl an ungestörten als auch an künstlich verdichteten Bodenproben durchgeführt. Bei diesem Versuch wurde eine quaderförmige Bodenprobe (21,5 x 5 x 5 cm) über eine dünne Kunststoffplatte durch drei

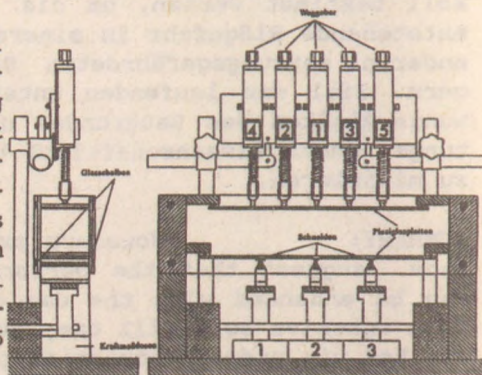


Bild 2: Versuchsanordnung

unabhängige Schneiden (Idealisierung als Einzelkräfte) belastet. Weggeber (Genauigkeit 1/100 mm) erfassen die Durchbiegung des Quaders in den 1/6 Punkten des Auflagerabstandes ($l_0=20$ cm). Die Belastung durch die Schneiden wurde über Kraftmeßdosen registriert (Genauigkeit 1 ± 2 N). Die Meßwerterfassung erfolgte über einen Meßwertverstärker, der die Daten an einen PC weitergab. Bei beiden Böden wurden Biegezugversuche unter Variation des Wassergehaltes - ausgehend von w_{opt} - vorgenommen, wobei der Wassergehalt stufenweise um $\Delta w=2\%$ auf der nassen Seite der Proctorkurve erhöht wurde.

Unter der Annahme der konstanten Krümmung k_0 der neutralen Faser wurde ein konstantes Biegemoment M erzeugt, und über die geometrische Beziehung die Grenzdehnung $\epsilon_{max} = z/R_0$ ermittelt: $\epsilon_{max} = 0,025 \text{ m} \cdot k_0$ [m⁻¹] mit $z = h/2 = 2,5$ cm.

Die Biegezugrandspannung β_z berechnete sich zu

$$\beta_z = \frac{M_{\max}}{W} = \frac{M_2}{I_g} \cdot z$$

mit I_g = Trägheitsmomente der Quader und Platten.

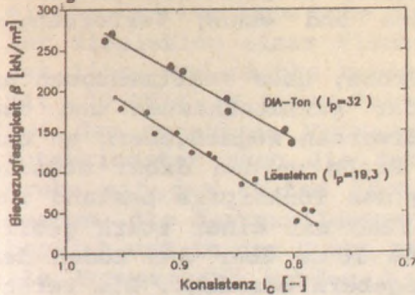


Bild 3: $\beta_z + I_c$ -Verlauf

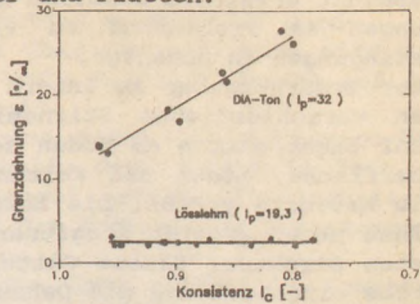


Bild 4: $\epsilon_{\max} + I_c$ -Verlauf

In den Bildern 3 und 4 wurden über der Konsistenz der Böden die Biegezugrandspannung β_z und die Grenzdehnung $\epsilon_{\max}$ aufgetragen. Wie vermutet, reichen schon so geringe Grenzdehnungswerte wie 0,3 % (Lösslehm) bzw. 2,7% (Ton) aus, um erste Risse im Randbereich der Probe hervorzurufen. Versuche, die Bodenproben mit einer Bewehrung zu versehen und ebenfalls auf Biegung zu untersuchen, scheiterten bisher an den kleinen Abmessungen der Proben und am fehlenden Reibungsverbund Geotextil+Boden.

3. Großmaßstäbliche Biegeversuche (Ton mit und ohne Geotextil)

In einem Versuchskasten (250x60x60 cm, s. Bild 5) mit absenkbarem Bodenblech wurde ca. 1,7 to künstlich aufbereitetes Tonmaterial (Quader mit den Maßen 30x27x10 cm, Tonanteil >40%) lagenweise eingebaut und mit leichtem Gerät abgerüttelt.

Die Seitenwände des Versuchskastens bestanden aus 2cm starkem Plexiglas, um die erwartete Rißbildung beobachten zu können. Um Reibungseinflüsse auszuschalten, wurden zwischen Probe und Wänden doppelt geschmierte PVC-Folien eingelegt. Die Oberfläche des Tonblocks wurde mit 100 kN/m² konstant belastet und die Belastungseinrichtung so ausgelegt, daß diese der späteren Verformung der Probe folgen konnte. Die Konsolidationsdauer vor dem Versuch

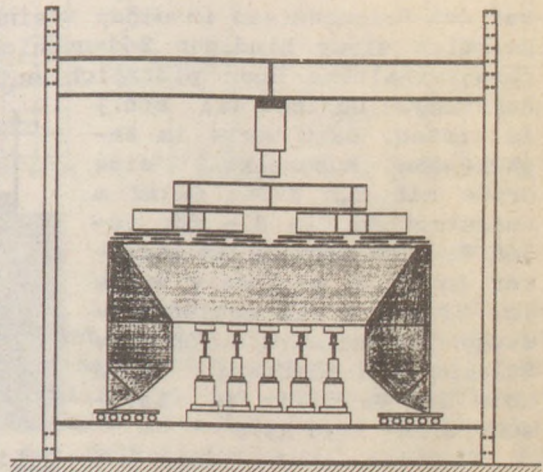


Bild 5: Ton-Biegeversuch

betrug 48 Stunden. Während des Versuchs wurde das Bodenblech stufenweise (in Intervallen alle 12 Std.) so ausgelenkt, daß die Probe kreisförmig durchgebogen wurde. Die seitlichen Auflagerkästen, in denen das Bodenblech auflag, wurden dabei am Fußpunkt ebenfalls seitlich ausgelenkt, um Querschnittsverformungen am Probenrand zu vermeiden und ebene Verformungsbedingungen zu erhalten.

Über Erddruckgeber im Innern der Probe, über Kraftmeßdosen an den verschieblichen Stirnseiten der Auflagerkästen und den fünf Lagerpunkten am Boden sowie diversen Wegmeßgebern an der Oberfläche, Boden und Seiten der Probe wurden dabei ständig die Meßwerte erfaßt. Die Bewehrung des Tonkörpers bestand aus einem hochzugfesten Geogitter aus PEHD mit einer stark geöffneten Struktur. Dieses Gitter wurde 10 cm über dem Boden der Probe eingebaut und mit Dehnungsmeßgebern bestückt. Die Veränderung der dabei erfaßten Meßwerte geben Aufschluß über die Bewehrungswirkung des Systems. Die Horizontalspannungen in der Probe sanken zum Geogitter hin ab, und die erforderlichen seitlichen Haltekräfte waren geringer (s. Bild 6). Finite Elemente Berechnungen bestätigten die Entlastung der Zugzone beim bewehrten Modell und die Konzentration der Zugspannungen in der Geotextilebene. Für die Berechnungen wurde ein nichtlineares Stoffgesetz verwendet, das große Deformationen zuläßt.

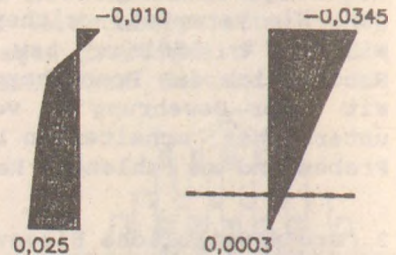


Bild 6: Horizontalspannungen, (FE-Analyse) in MN/m^2

4. Großmaßstäbliche Feldversuche

Bei dem Versuchsfeld in einem Steinbruch kam es darauf an, die Reaktion einer bindigen Bodenschicht hinsichtlich ihres Setzungsverhaltens über plötzlich entstandenen Hohlräumen (Verkarstung, Dolinenfall etc.) zu testen. Dazu wurde im anstehenden Muschelkalk eine Grube mit den Maßen $6 \times 3 \times 2$ m ausgebrochen, in die für jeden Versuch Lößlehm in lockerer Lagerung eingebaut wurde (s. Bild 7). Die setzungserzeugende Last wird durch eine Belastungseinrichtung 2×2 m über der Grube in 10 Lastschritten á 10 kN/m^2

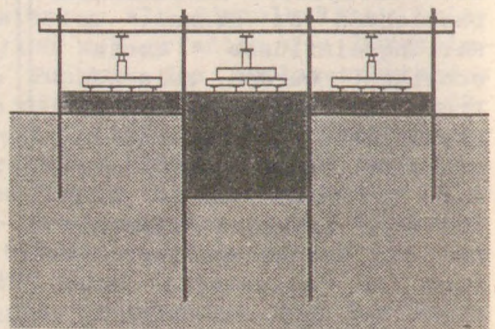


Bild 7: Systemschnitt des Feldversuchsstandes

aufgebracht, zwischen jeder Belastungssteigerung wurde ein Zeitraum von ca. 6 Std. zum Abklingen der Setzung eingeschaltet.

Die geotextile Verstärkung der Lösschicht wurde in einer Breite von 2m quer über die Grube gelegt, wobei das Geotextil mittig in eine 50cm hohe Überschüttung eingebettet wurde.

Zur Simulation einer Flächenlast über dem Geotextil wird dieses links und rechts neben der Grube mittels Pressen auf 4 m^2 mit 100 kN/m^2 im Boden "eingespannt".

Bei den genannten Versuchen werden neben den Setzungen über Potentiometer auch die Setzungsmulde des Geotextils über der Grube mit dem System "Schlauchwaage" erfaßt, die induzierten Lasten, die Geotextildehnung und der Erddruck im Verankerungsbereich und in der Grube gemessen.

Als Vorversuche wurden 3 Versuche ohne geotextile Bewehrung durchgeführt. Es zeigte sich, daß bei Maximallast der Lößboden in der Grube um ca. 50 cm komprimiert werden konnte, die Setzung erfolgte hier ohne die charakteristische Muldenform. Die Lastplatten stanzten sich tief in den Lößlehm ein. Im Bereich des Grubenrandes zeigten sich bis zu 5 cm breite Oberflächenrisse.

Die bereits abgeschlossenen Versuche mit zwei Geogittern zeigten im Gegensatz zum unbewehrten Feld eine deutliche Verringerung der Oberflächenrisse im Bereich des Grubenrandes (Stelle des größten Biegeradius), eine Reduzierung der max. Setzung des Belastungsfeldes um ca. 30 % und eine flache, gleichmäßige Setzungsmulde ohne kritische Krümmungsradien(s. Bild 8). Die Geotextildehnung zweier Geogitter (Grubenrand) zeigt Bild 9.

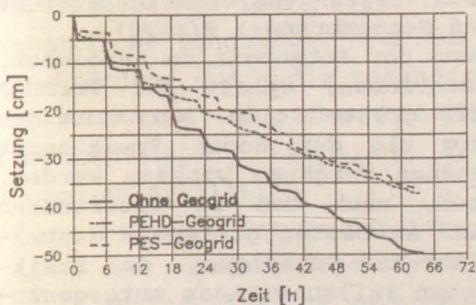


Bild 8: Setzungsverläufe

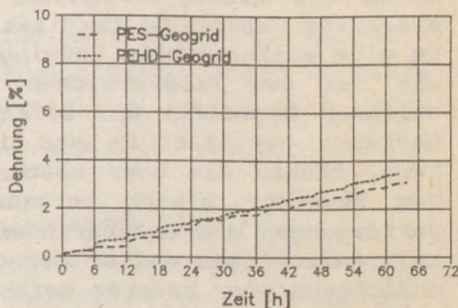


Bild 9: Geotextil-Dehnung

Insgesamt werden 2 Geogitter, 1 Gewebe und 2 schwere Vliese auf ihre Bewehrungswirkung untersucht. Im Anschluß an die Feldversuche werden auch diese Versuchsergebnisse mit FE-Berechnungen verglichen. Parallel zu den genannten Feldversuchen laufen Kastenscherversuche in einem $30 \times 30 \text{ cm}$ Scherkastengerät zur Ermittlung der Reibungsparameter zwischen den genannten Geotextilien und Lößlehm.

5. Schlußbetrachtung

Geotextile Bewehrung kam bisher zumeist nur bei Böden zur Anwendung, die unter anderem auch sandig-kiesige Bestandteile besaßen, um einerseits die Verzahnungswirkung in den Öffnungen des Geogitters voll mobilisieren zu können, andererseits die Rauhgigkeit ihrer Struktur über Oberflächenreibungskräfte ins Geotextil einfließen zu lassen. Bei bindigen Böden lag bisher nur wenig Erfahrung vor, diese durch Geogitter zu bewehren. Sie besitzen aber, wie gezeigt, auch die Fähigkeit, sich wegen des hohen Feinkornanteils dicht in die Öffnungen des Geogitters einzudrücken und besonders gut mit dem Geogitter verkleben, wie ausgegrabene Stücke zeigten. Bei der Interaktion mit dem Geogitter können hier zusätzlich noch Kohäsionskräfte geweckt werden, wenn der Wassergehalt des Bodens nicht zu hoch ist.

Bei den angesprochenen Kastenscherversuchen zeigte sich, daß sich das Feinkorn auf der Oberflächenstruktur bestimmter Geotextilien derart festsetzt, daß die Scherfuge nicht zwischen Geotextil und Boden, sondern wenige Millimeter im Boden selbst lag.

Es existieren bereits Ansätze verschiedener Autoren, die Bodenparameter ϕ und c zur Berechnung des Verbundes mit div. Geotextilien einzubeziehen. In jüngster Zeit werden auch Bemessungstabellen erstellt, wo in Abhängigkeit von der Spaltenbreite, der Geotextildehnung, der Durchbiegung und der Auflast die Auswahl eines für den individuellen Fall zugeschnittenen Geotextils durchgeführt werden kann (z.B. Giroud, Bonaparte, Beech and Gross; "Design of soil layer-geosynthetic systems overlying voids"; Geotextiles and Geomembranes; 9/1990).

Um eine wirkungsvolle Verringerung der Setzungen zu erreichen, wie bei den Feldversuchen beschrieben, spielt der Verbund zwischen Geotextil und Boden die größte Rolle. Nur wenn das Geotextil vollflächig und innig mit dem Boden "reagieren" kann, können die oben beschriebenen Vorgänge wirksam werden. Dem Anwender bleibt vorbehalten, nach den zu erwartenden Verformungen des Untergrundes das am besten geeignete Geotextils auszuwählen und so der Gefahr der Schädigung der Basisabdichtung oder anderen gefährdeten Teilen wirksam entgegenzuwirken.

Sicherung eines historischen Bauwerkes
nach dem SOILCRETE-Verfahren

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1. ALLGEMEINES

Viele historische Bauten sind in ihrem Bestand gefährdet. Ursachen dafür sind häufig die exponierte Lage, der Bauplatz im historischen Kern einer Stadt oder die Situierung an Gewässern. Häufig wurde trotz ungünstiger Bodenverhältnisse nach heutigen Gesichtspunkten eine unzureichende Gründung gewählt. Oft haben aber auch äußere Einflüsse, wie Niveauänderungen, Verkehr oder vor allem auch die Veränderung des Grundwasserspiegels nachteilige Auswirkungen auf das Setzungsverhalten solcher Gebäude. Mit wenigen Ausnahmen konnten früher in solchen Fällen lediglich Erhaltungs- und Restaurierungsmaßnahmen gesetzt werden, da wesentliche Randbedingungen für eine wirtschaftliche Sanierung der Gründung durch die bekannten Möglichkeiten nicht sicherzustellen waren. Mit dem SOILCRETE-Verfahren steht eine Sanierungsmethode zur Verfügung, die folgende wesentliche Randbedingungen erfüllt:

- Schonung der Bausubstanz durch erschütterungs- und bewegungsarme Herstellung der Zusatzmaßnahmen.
- Kraftschlüssige Verbindung zwischen Altbestand und dem verbesserten Boden.
- Verbesserung des Lasteintragungsbereiches am Bauwerk selbst.
- Sanierung durch Nachgründung ohne Veränderung des statischen Konzeptes.

- Verwendung von natürlichen und erprobten, umweltfreundlichen Einbaustoffen wie Zement.
- Wirtschaftliche Ausführungsmöglichkeiten durch geringste Zusatzmaßnahmen.

2. SOILCRETE-VERFAHREN

Wesentlicher Bestandteil des Verfahrens ist ein Hochgeschwindigkeitsdüsenstrahl, welcher den anstehenden Boden durch seine Erosionswirkung löst. Mit einem Pumpendruck von etwa 300 - 700 bar werden Strahlgeschwindigkeiten von mehreren hundert m/sec. erreicht, die hydraulische Erosionsleistung ergibt sich in Abhängigkeit von Boden, Reichweite und der Energie des Düsenstrahles.

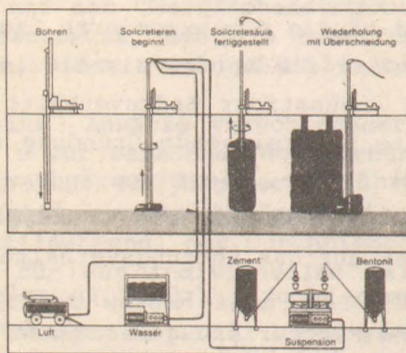


Bild 1

Im wesentlichen wird zunächst ein Bohrloch mit Gestängedurchmessern von 40- 89 mm hergestellt. Das Bohrverfahren wird den Bodenverhältnissen angepaßt. Nach Erreichen der Endtiefe wird der anstehende Boden über meist horizontal angebrachte Düsen am Bohrkopf aufgeschnitten. Die Bewegung der Düseneinrichtung, Umdrehung und Ziehgeschwindigkeit wird über das Gestänge entsprechend der Bauaufgabe und den bodenmechanischen Werten zuvor festgelegt. Die Energie des Schneidstahles wird bei der Fräs- und Spülarbeit verbraucht und der ausgefräste Boden teilweise über den Bohrlochringraum nach oben gespült. Gleichzeitig mit dem Fräsen wird der gelöste Boden mit Suspension aus Zement oder Bentonit-Zement vermischt. Es entstehen zylindrische oder scheibenförmige SOILCRETE-Elemente. Durch entsprechende Anein-

anderreihung dieser Einzelsäulen oder Scheiben sind beliebige, den jeweiligen Anforderungen entsprechende Körper herzustellen.

Grundsätzlich werden 3 Verfahren unterschieden:

- **SOILCRETE S (Single)**

Zur Verwendung gelangt ein Einfachgestänge, über welches unter Hochdruck Zementsuspension den Düsen zugeführt wird. Die Zementsuspension dient hier gleichzeitig als Schneidmedium. Durch das hohe spezifische Gewicht der Suspension entstehen Reibungsverluste, die die Eindringtiefe bei diesem System auf nur 0,4 bis 1,0 m Durchmesser beschränken.

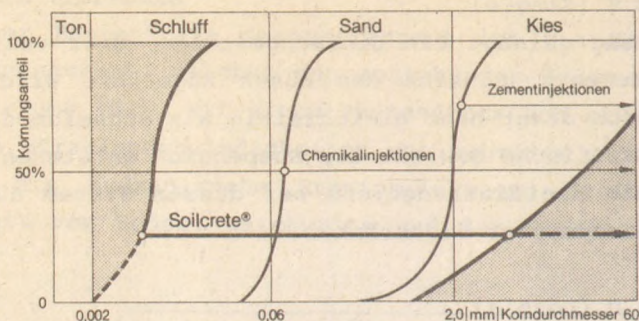
- **SOILCRETE D (Double)**

Die Ausführung erfolgt mit Doppelgestänge. Als Schneidmedium wird wiederum Zementsuspension verwendet. Über eine ringförmige Luftdüse wird der Zementstrahl luftummantelt, wodurch die Reibungsverluste der eintretenden Zementsuspension vermindert und die Reichweiten erhöht werden. Die erzielbaren Durchmesser liegen zwischen dem SOILCRETE S und dem nachfolgend beschriebenen SOILCRETE T Verfahren.

- **SOILCRETE T (Triple)**

Verwendet werden spezielle Dreifachgestänge, wobei die Vorgänge Schneiden und Vermörteln im Bereich des Düsenkopfes räumlich getrennt werden. Das Fräsen des Bodens erfolgt mittels Hochdruckwasserstrahl, der über eine Ringdüse zur Erhöhung des Wirkungsgrades mit Luft ummantelt werden kann. Die Vermörtelung erfolgt im selben Arbeitsgang, gleichzeitig über eine zweite, unterhalb der Schneiddüse angebrachte Zementdüse, sodaß für die Zementsuspension nur mehr Drücke von 20 - 40 bar erforderlich sind. Durch die räumliche Trennung des Schneid- und Zementierungsvorganges erfolgt einerseits eine homogene Vermörtelung innerhalb des Arbeitsraumes im Boden, auf Grund der geringeren Reibungsverluste sind außerdem mit diesem Verfahren die größten Säulendurchmesser herzustellen.

Üblicherweise ausgeführte Durchmesser der einzelnen Verfahren sind mit 0,8 bis 2,4 m anzugeben. Die erreichten Durchmesser liegen jedoch bei verschiedenen Bauaufgaben nach dem SOILCRETE-T-Verfahren bei über 3 m.



Anwendungsgrenzen des Verfahrens

Bild 2

Die derzeitigen Verfahrensgrenzen werden in Bild 2 gezeigt. Daraus ist ersichtlich, daß neben Kiesen und Sanden praktisch der gesamte Bereich des Schluffes verbessert werden kann. Bei Tonanteilen von 10 - 30 % ist beispielsweise das SOILCRETE-T-Verfahren noch einsetzbar, allerdings sind die wirtschaftlich erreichbaren Druckfestigkeiten deutlich geringer. Außerdem wächst der Aufwand für das Lösen des Bodens mit zunehmender Kohäsion, sodaß die Wirtschaftlichkeit gegenüber anderen Verfahren des Spezialtiefbaues bei höherem Tonanteil zu überprüfen ist.

Die erreichbaren Festigkeiten des SOILCRETE-Körpers hängen von der Bodenzusammensetzung und vom Zementgehalt ab. Üblicherweise werden für Kiese Druckfestigkeiten von 8 bis 18 MN/m², für Sande 5 bis 14 NM/m² und für Schluffe und Tone 5 bis 10 MN/m² erreicht. Die Wasserdurchlässigkeit des SOILCRETE-Körpers kann mit 10^{-7} bis 10^{-9} m/sec. hergestellt werden.

3. GRÜNDUNGSSANIERUNG DER KARTAUSE MAUERBACH

3.1 ALLGEMEINES

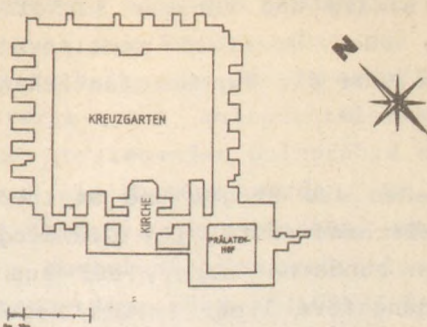


Bild 3

Auf einer rund 25.000 m² großen Grundfläche befindet sich als eines der hervorragenden Kulturdenkmäler Österreich die Kartause Mauerbach in unmittelbarer Nähe Wiens. Nach der Gründung im Jahr 1313 wurde der Bau mehrfach zerstört und in seiner jetzigen Form gegen Ende des 17. Jahrhunderts errichtet. Das ursprünglich im glanzvollen klassischen österreichischen Barock wiedererstandene Gebäude erfuhr in den letzten 200 Jahren seiner wechselvollen Geschichte eine beträchtliche Abnützung. In den letzten Jahrzehnten traten vermehrt stärkere Risse in den tragenden Mauern auf die eine grundlegende Sanierung des Bauwerkes erforderlich machten. Die vorhandenen Risse weisen eindeutig auf größere Setzungen hin, deren Ursachen in einer starken Änderung der Grundwasserverhältnisse und in einem Versagen der ursprünglichen Fundierung zu suchen waren.

3.2 GRÜNDUNGSVERHÄLTNISSE

Das Gebäude liegt im Bereich der Flyschzone des Wienerwaldes. Der Flysch besteht aus Sandsteinen und Mergeln und ist zum Teil stark rutschgefährdet. Im südwestlichen Teil des Bauwerkes sind die Sandsteine und Mergel des Flysches durch jüngere quartäre, zumeist locker bis mitteldicht gelagerte Bachablagerungen überlagert. Die umfangreichen Aufschlüsse haben gezeigt, daß unter

einer mehreren dm mächtigen Humusschicht, Anschüttungen, bestehend aus Ziegel, Schutt und Mauerwerksbrocken, Steinen und sandigen Schluff bis etwa 3 m Tiefe anstehen. Darunter folgt unterschiedlich sandig und toniger Schluff, steifer bis weicher, fallweise auch breiiger Konsistenz. Das Grundwasser befindet sich in Höhe der Fundamentunterkante, rund 3 m unter Gelände.

Die Mauern bestehen aus Ziegel und Steinen, also einem meist unregelmäßigen Mischmauerwerk. Die Gründung erfolgt auf einem darunterliegenden Fundamentkörper, der aus Steinmauerwerk besteht. Die Gründungsform liegt in der Regel als Flachgründung vor, fallweise sind Außenmauern auf einem Gewölbe aus Ziegel- und Steinmauerwerk gegründet, dessen Kämpfer auf einem Ziegelmauerwerk aufliegen. Bei Vorliegen ungünstiger Gründungsverhältnisse, wie weicher oder breiiger Konsistenz des darunterliegenden Schluffes, wurden sowohl derartige Gewölbegründungen als auch Steinmauerwerksfundamentkörper auf horizontalen Holzrosten abgesetzt, die wiederum auf mehreren Metern tiefen gerammten Holzpfählen gegründet waren.

Der Erhaltungszustand der Holzpfähle wurde an mehreren Schürfen relativ gut vorgefunden, in vielen Fällen sind jedoch auch vermorschte Pfähle bzw. Holzroste festgestellt worden.

Die Belastungen lagen in der Regel zwischen 250 und 350 kN/lfm Fundament im Bereich der Innen- und Außenwände. Im viergeschoßigen Bereich von Kirche und Lapidarium traten Laufmeterlasten von bis zu 900 kN/lfm auf. Besonders gefährdet waren die Innenpfeiler, welche Belastungen von 700 bis 750 kN aufgewiesen hatten.

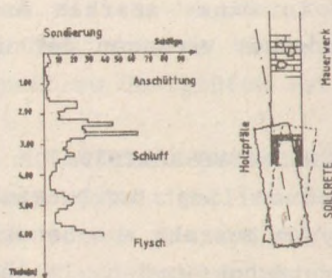


Bild 4

Zur Sanierung der bestehenden Gründung mußten die unterhalb der Fundamentsohle anstehenden Weichschichten überbrückt werden. Nach sorgfältiger Projektierung wurde die Ausführung einer Hochdruckbodenvermörtelung nach dem SOILCRETE-Verfahren vorgesehen. Damit war sowohl die Verbesserung des weichen, sandig tonigen Schluffes unterhalb der Gründungssohle als auch der Ersatz allenfalls verrotteter Holzpfahlelemente möglich. Im Falle von unverrottet vorliegender Holzpfähle sollten diese in den SOILCRETE-Körper eingebunden werden. Um die bestehenden Fundamente möglichst flächig zu unterfangen, waren große Säulendurchmesser für die Hochdruckbodenvermörtelung erforderlich. Dies führte zur Entscheidung, das SOILCRETE-T-Verfahren anzuwenden. Bei Säulendurchmessern von 100 bis 120 cm wurden Einzelsäulen in Abhängigkeit von der jeweiligen Fundamentbelastung in Abständen von 1 bis 2 m hergestellt. Die besondere Schwierigkeit lag in den beengten räumlichen Verhältnissen. Wegen der erforderlichen Abbindezeit der jeweils hergestellten SOILCRETE-Säule war ein Abstand von mehreren Metern bis zur nächsten ausgeführten Säule erforderlich. Dies bedingte ein laufendes Umstellen des Gerätes, was besonders bei schwieriger Zugänglichkeit des Aufstellpunktes sehr zeitaufwendig ist. Durch Einsatz wendiger und klein gebauter Geräte war dies auch unter den gegebenen Verhältnissen möglich.

Besondere Berücksichtigung bei der Bauausführung mußte ein weitläufiges, sich über den gesamten Gebäudekomplex erstreckendes, Drainagesystem finden. Auch wenn im Zuge der Hochbausanierung dieses zum Teil aus dem Mittelalter stammende Wasserleitungssystem erneuert wird, bestand das Bemühen, diese Leitungen während der SOILCRETE-Arbeiten zu erhalten. Dies konnte durch umfangreiche Beobachtungen während der Arbeiten und laufende Reinigungsspülungen im unmittelbaren und weiteren Arbeitsbereich sichergestellt werden. Wie dies häufig bei derartigen historischen Bauten der Fall ist, lagen Aufzeichnungen und Planunterlagen äußerst mangelhaft vor. Unbekannte Bauteile oder Hohlräume im Untergrund führen dann zu Überraschungen und Mehraufwendungen.

Für das Gebäude gefährlich sind insbesondere derartige unbekannte Hohlräume im Zusammenhang mit konzentrierten Einzellasten, wie Einzelstützen oder Gewölbestützen. Um die im Bereich der Kirche Mauerbach vorliegenden Stützenlasten schadensfrei zu unterfangen, mußten zunächst Tastbohrungen vorgenommen werden. Bei Antreffen von Hohlräumen, und dies war sehr häufig der Fall, wurden zunächst Auffüllinjektionen für die Vorstabilisierung vorgenommen. Die zumindest 4 SOILCRETE-Säulen für jede Einzellast wurden dann an jeweils 4 aufeinander folgenden Arbeitstagen unter größtmöglicher Vorsicht hergestellt, sodaß damit eine schadensfreie Unterfangung konzentrierter und nach neueren Erkenntnissen mangelhaft gegründeter Einzellasten möglich war.

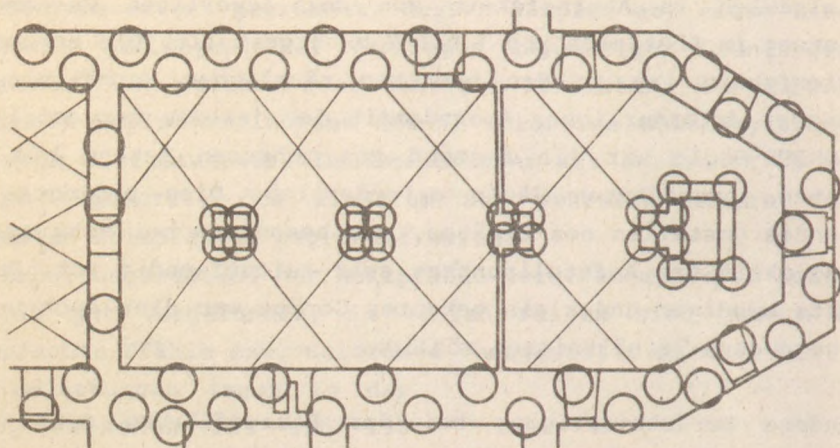


Bild 5

Bisher wurden die Nachgründungsarbeiten im Bereich des Prälatentraktes im Jahr 1988 und ein weiterer Abschnitt im Bereich der Kirche im Jahr 1989 abgewickelt. Die Sanierung des Kreuzganges mit den einzelnen Kartausen steht noch bevor. Die bei den bisherigen Ausführungen gewonnenen Erfahrungen werden es ermöglichen, auch die noch ausstehenden Sanierungsarbeiten trotz der schwierigen Verhältnisse erfolgreich nach dem SOILCRETE-Verfahren fertigstellen zu können.

Erfahrungen in rutschungsgefährdeten Neubaugebieten am Beispiel des Avas-Hügels in Miskolc

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Einleitung

Der etwa 4 km² große Avas-Hügel liegt im inneren Teil der Stadt Miskolc. Der Vorschlag, dieses, früher landwirtschaftlich genutztes Gebiet zum Wohnungsbau zu nutzen, stammt vom Ende der '60-er Jahre. Obwohl schon die ersten Gutachten [FTI 59/1287, FTV 65-1165/1-3] auf die Gefahr von Böschungsruutschungen hingewiesen haben, wurde mit der intensiven Bautätigkeit in den '70-er Jahren begonnen. Im Laufe der Ausbreitung der neuen Wohnsiedlung - parallel mit dem Fortschreiten der Bauarbeiten - wurden zahlreiche Baugrund-Untersuchungsbohrungen durchgeführt, um mit Hilfe der gewonnenen Daten die Konstruktion und die Gründung der geplanten Bauwerke entsprechend gestalten zu können. Trotz der gründlichen Vorbereitung meldeten sich schon während der Bauarbeiten Bodenbewegungen und bald waren auch Bauwerksschäden zu verzeichnen. Die wachsende Anzahl der Schadenfälle gab den Anlaß zur umfassenden Neuauswertung der Daten der etwa 1500, im Laufe der Bauarbeiten durchgeführten Untersuchungsbohrungen. Unter Verwendung sämtlicher, zur Verfügung stehender Daten wurde zur Darstellung der geologischen, hydrogeologischen Verhältnisse des Avas-Hügels eine Reihe von Spezialkarten angefertigt und es wurden auch neue Untersuchungen zur Standsicherheit der Böschungen vorgenommen. Aufgrund dieser Auswertung war es möglich, die zu Bodenbewegungen am meisten neigenden Teilgebiete abzugrenzen. Um weiteren Schäden vorzubeugen, wäre in diesen Bereichen der Ausbau eines Beobachtungsnetzes zur Verfolgung der eintretenden Bodenbewegungen angebracht.

Geologische und hydrogeologische Verhältnisse, geologischer Aufbau des Avas-Hügels

Der Avas-Hügel ist einer der den Ostrand des Bükk-Gebirges begrenzenden, etwas gegen SO gekippten, sanft hervorgehobenen tektonischen Blöcke. Seine Ausdehnung beträgt in N-S-licher Richtung etwa 2800 m, in O-W-licher Richtung 1400 m. Sein Querschnitt ist auf bezeichnende Art asymmetrisch: während die 1 km lange SO-liche Flanke eine durchschnittliche Neigung von 3-4° aufweist, hat die N-liche und W-liche Seite eine Neigung von mindestens 7°, die aber stellenweise auch 10-15° erreichen kann.

Am geologischen Aufbau des Avas-Hügels nehmen drei verschiedene stratigraphische Einheiten teil: auf miozänen vulkanogenen Bildungen liegt eine sedimentäre pannonische Schichtenfolge, die von quartären Ablagerungen bedeckt werden.

Die den Stufen Torton-Sarmat des Miozäns zugerechneten primären und umgelagerten Pyroklastika und vulkanogenen Ablagerungen rhyolitischer und andesitischer Zusammensetzung gelangen am nördlichen Hang des Hügels an die Oberfläche. Die Schichtenfolge fällt mit 5-12° gegen SSO ein [NME, 1979] und

Mächtigkeitkarte des Quartärs, die Flurabstandskarte des Grundwasserspiegels und Karten für die Grundwasserbeschaffenheit angefertigt. Durch die Zusammenschau all dieser Unterlagen konnten die besonders rutschungsgefährdeten Bereiche abgegrenzt werden (Bild 4.). Die örtliche Verteilung der eingetretenen Bodenbewegungen und Bauwerksschäden stimmt mit den Ergebnissen gut überein.

Zweitens wurde in drei ausgewählten Profilen mit Hilfe der Lamellen-Methode nach Janbu und Bishop die Standsicherheit der Böschung untersucht. Die Profile wurden in Hangrichtung an Stellen angesetzt, wo ein jähher Neigungswechsel der Oberfläche Pannon-Quartär und ein kleiner Flurabstand des Grundwasserspiegels eine besonders große Rutschgefährdung andeutete und in deren Nähe tatsächlich Bodenbewegungen aufgetreten sind. Entlang dieser Profile wurden sowohl für den Fall der Ausbildung von Kreis- wie auch für zusammengesetzte Gleitflächen Berechnungen durchgeführt. Bei der Festlegung des Modells wurde der erfahrungsmäßigen Tatsache Rechnung getragen, daß die Rutschungen meist entlang der Grenzfläche Pannon-Quartär vor sich gehen. Die Berechnungen haben - übereinstimmend mit den Feststellungen früherer Gutachten - zum Ergebnis geführt, daß die Böschung auch unter Voraussetzung eines real annehmbaren minimalen Reibungswinkels und Kohäsionswertes und unter den ungünstigsten hydrogeologischen Verhältnissen mit Sicherheit stehen müßte.

Die Erfahrung widerspricht aber den Berechnungen. Schon während der Bauarbeiten ereigneten sich zahlreiche Bodenbewegungen, und später traten an den fertigen Bauwerken - manchmal unreparierbare - Schäden auf. Aus den Berichten [FTI 76/1885, FTI 77/695a,b, FTV 80/1956, FTV 86/525-I-II, NME 1987, EME 1988, FTV 88/433-I-a-c] geht hervor, daß die Rutschungen in Regenperioden oder in ihrer Folgezeit aufgetreten sind.

Auf dem SO-lichen Hang des Hügels sind Zeichen der langsamen Talwärtsbewegung, des Kriechverhaltens der oberen Schichten (gekrümmte Baumstämme, schiefe Zäune und Lampenmaste) zu beobachten. Die Tatsache der langsamen Hangbewegung ist durch Inklinometermessungen bestätigt worden [NME 1987].

Aufgrund der Neubewertung der Bohrungen und der Analyse der erwähnten Karten konnten die rutschgefährdeten Teilgebiete in folgender Weise charakterisiert werden [NME 1989]:

- lokales Anwachsen der Mächtigkeit der Quartärschichten,
- Grabenartige, in Hangrichtung verlaufende Vertiefung der Oberfläche Pannon-Quartär, die höchstwahrscheinlich einen einstigen, verdeckten Erosionseinschnitt darstellt,
- kleiner Flurabstand des Grundwassers, das an diesen Stellen von pannonischen, in Hangrichtung einfallenden Sand- und Schluffschichten geringer oder mittlerer Wasserleitfähigkeit gespeichert wird, so daß diese Schichtenköpfe das Grundwasser unmittelbar auf die diskordant auflagernden Quartärbildungen zuleiten."

Durch den Vergleich der Berechnungsergebnisse und der praktischen Erfahrungen konnten folgende Feststellungen gemacht werden:

- die vorbehaltlose Anwendung der bekannten und allgemein akzeptierten, im Sinne der Statik korrekten Methoden zur Prüfung der Standsicherheit eignen sich nicht in jedem Fall zur Beurteilung der Stabilität von Böschungen.: Die tatsächlichen Verhältnisse nicht widerspiegelnden Ergebnisse der Berechnungen können aber nicht allein mit den den Methoden eigenen vereinfachenden Voraussetzungen erklärt werden, denn das berechnete Maß der Sicherheit (Sicherheitsgrad ≥ 4.0) ist sehr groß. Beim Zustandekommen der Rutschungen kann die während des Kriechvorganges auftretende und die durch die Austrocknungsrisse der durch große lineare Schrumpfung gekennzeichneten Tone bedingte Festigkeitsabnahme eine Rolle spielen, außerdem ist auch ein Aufleben fossiler Gleitflächen möglich. Da diese Faktoren in den Ausgangsdaten der bodenmechanischen Berechnungen (innerer Reibungswinkel, Kohäsion, neutrale Spannung, geometrische Parameter)

bildet den wasserundurchlässigen Untergrund der pannonisch-quartären Schichten.

Die 10-150 m mächtige, in SO-licher Richtung an Mächtigkeit zunehmende und mit 10-15° gegen SO einfallende pannonische Schichtenfolge besteht neben Tonen aus sandig-schluffigen, wasserdurchlässigen Ablagerungen. Sie sind in den küstennahen Bereichen des Pannonischen Sees entstanden und weisen eine Faserstruktur auf. Die einzelnen petrographischen Einheiten sind cm- bis m-dick, ihre horizontale Ausdehnung beträgt 10 bis 100 m [NME, 1979]. Die wasserleitenden Sandeinlagerungen enthalten zumeist gespanntes Grundwasser. Die Tragfähigkeit der Schichten ist ausreichend, das lineare Schrumpfungsmaß der Tone ist bautechnisch günstig (Bild 1.).

Auf der pannonischen Schichtenfolge liegt ein - meist brauner, fetter - Ton quartären Alters. Seine Mächtigkeit wächst gegen SSO an und erreicht bis zu 15 m. Diese Tonschicht mit hohem linearen Schrumpfungsmaß ist - wegen ihrer Schrumpfungsrisse - nicht wasserundurchlässig, durch sie ist eine Versickerung des Niederschlags möglich. Die bodenmechanische Beschaffenheit dieser Schicht ist aus bautechnischer Sicht ungünstig (Bild 2.).

Das in den pannonischen und quartären Schichten gespeicherte Wasser verfügt vermutlich über eine einheitliche Druckspiegelfläche, diese Vorstellung wird durch die Beobachtungen in entsprechend ausgeführten, an entsprechender Stelle errichteten Brunnen unterstützt. Nach den vorliegenden Daten folgt der Druckwasserspiegel ziemlich konsequent der Erdoberfläche, doch sind sprungartige zeitliche Wasserstandsänderungen - sogar in der Größenordnung von 10 m - möglich. Der durchschnittliche Flurabstand des Grundwasserspiegels beträgt von Null bis zu 15 m, auf dem SO-lichen Hang kann der Druckspiegel auch über Flur liegen (Bild 3.).

Das Grundwasser hat einen neutralen bis schwach alkalischen pH-Wert, eine Gesamthärte von 20-50 °dH und den Charakter eines Kalzium-Hydrogen-Karbonat- bzw. Sulfat-Wassers, und ist i.a. nicht betonaggressiv.

Menschliche Eingriffe im Untersuchungsgebiet

Im Zuge der allmählichen Bebauung der Hänge wurde der natürliche Zustand der Böschungen durch die Planierungsarbeiten gestört. In einzelnen Bereichen wurden bis zu 10 m mächtige, inhomogene, oft ungenügend verdichtete, nicht tragfähige Aufschüttungen aufgebracht. Für die einzelnen Gebäude wurden einige Meter tiefe Einschnitte errichtet, mit denen zuweilen auch das Grundwasser angezapft wurde.

Den Bauplänen gemäß wurden den Höhenlinien entlang streifenartig 100-200 m lange Wohnblöcke angelegt, deren tiefe Gründungen vermutlich die natürliche Strömung des Grundwassers stören.

Zur Erhöhung der Stabilität der Böschungen wurden fast alle Gebäude mit Drainageeinrichtungen umgeben. Diese Objekte geben bei unsachgemäßer Ausführung Möglichkeit zu konzentriertem Wasserzufluß in den Untergrund.

Zur Versorgung der in der Wohnsiedlung wohnenden 50000 Menschen wurde ein Trinkwasserleitungsnetz und ein Kanalisationsnetz gebaut. Eine besonders große Gefahrenquelle stellt jene, das Baugebiet durchquerende Wasserleitung großen Durchmessers (500 mm) dar [ESZAKTERV, 134/1973], deren eventueller Bruch große Wassermengen in die ungünstige bodenmechanische Eigenschaften aufweisende obere Schicht eindringen lassen würde. Mit zunehmender Alterung des Leitungsnetzes muß - wegen den immer größer werdenden versickernden Wassermengen - auch mit wachsender Neigung zu Bodenbewegungen gerechnet werden.

Standsicherheit der Böschungen

Zur Prüfung der Standsicherheitsverhältnisse der Böschungen des Hügels wurden zweierlei Ansätze gewählt.

Erstens wurden durch die neuerliche Zusammenfassung und Auswertung aller früheren Untersuchungsbohrungen geologische Karten bezüglich einzelner Tiefenintervalle, die Höhenlinienkarte der Oberfläche Pannon-Quartär, die



Bild 1.: Höhenlinienkarte der Oberfläche der pannonischen
Bildungen

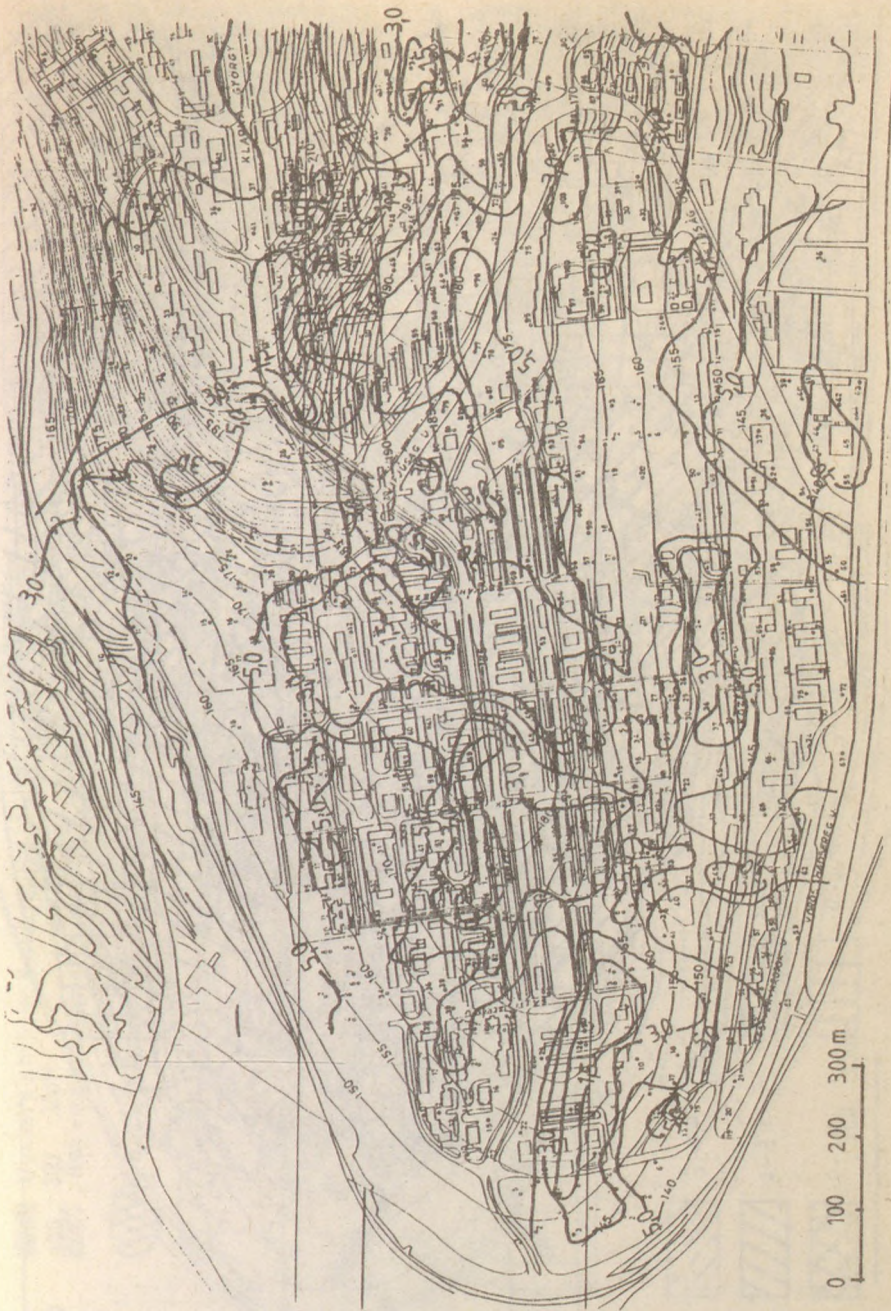


Bild 2.: Isolinien der Mächtigkeit der quartären Bildungen

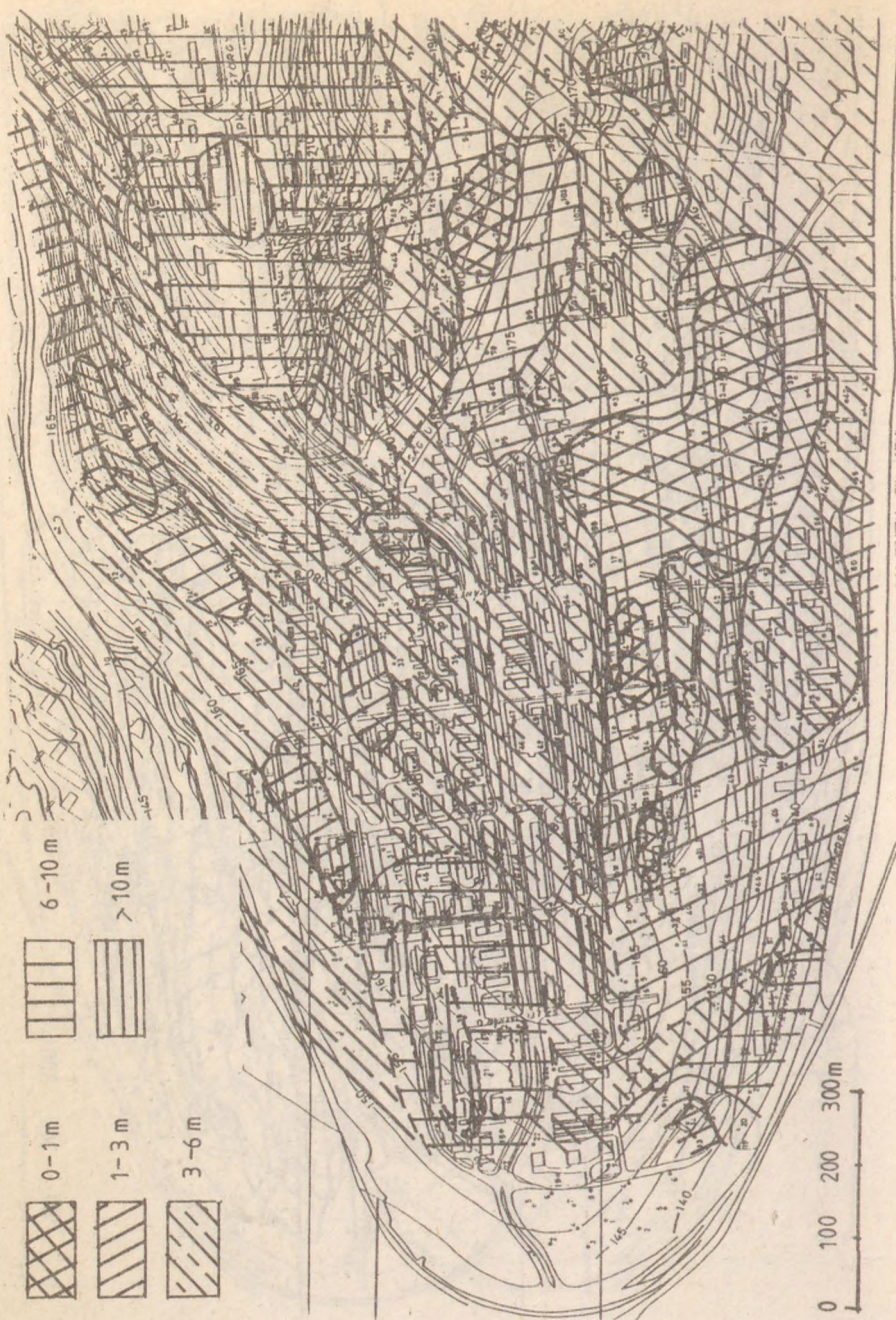


Bild 3.: Tiefe des Grundwasserspiegels unter der Oberfläche

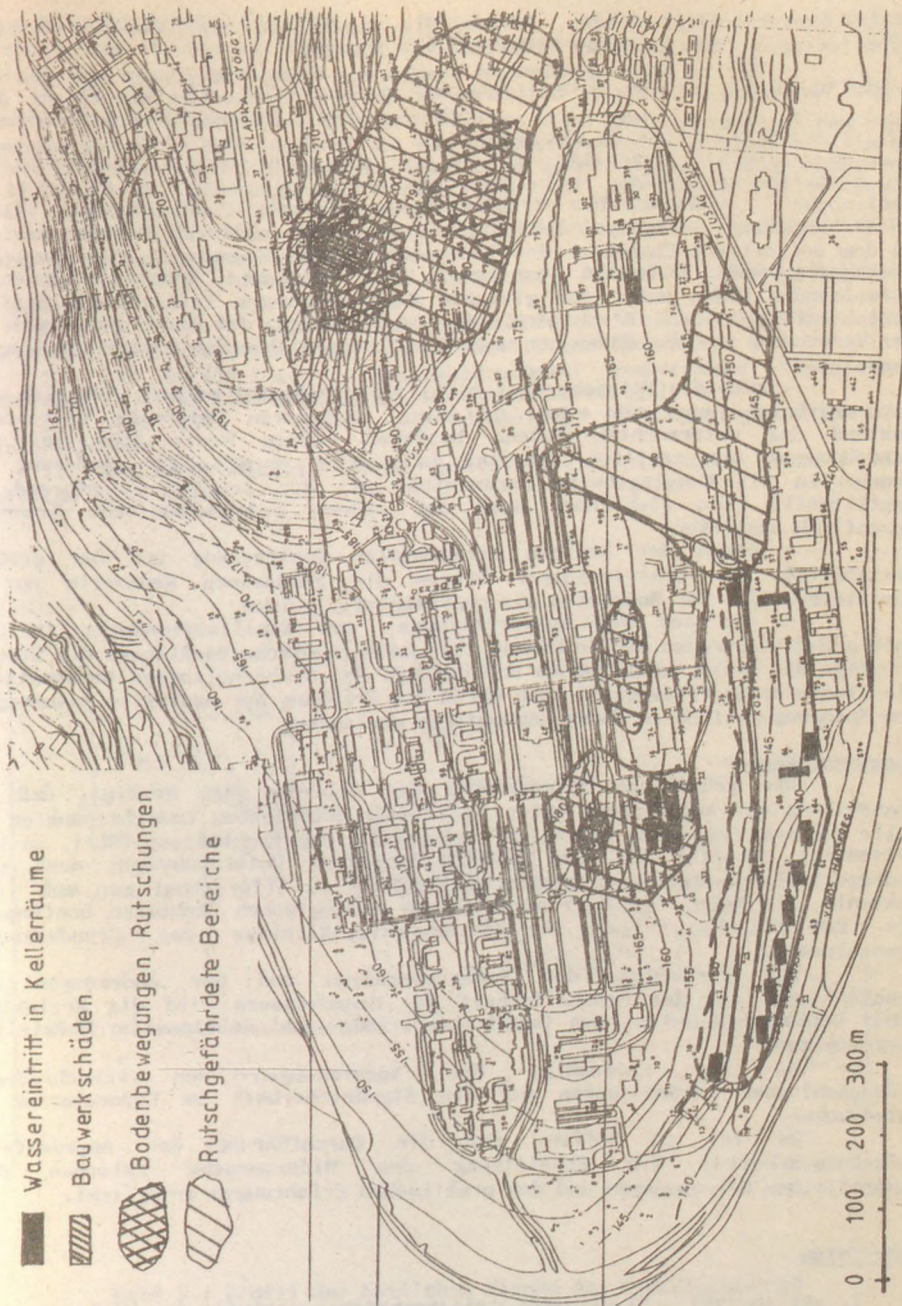


Bild 4.: Übersichtskarte der Bauwerkschäden

nicht berücksichtigt werden, können die Ergebnisse eine der Wirklichkeit nicht entsprechende günstige Situation vortäuschen.

- die im Labor gemessenen (meßbaren) Kennwerte können von den das wahre Verhalten der Böden bestimmenden in-situ-Kennwerten abweichen; denn wenn man im Labor mit möglichst ungestörten, kleinen Proben arbeitet, tut man es mit der Absicht, die Eigenschaften der in ihrem Maßstab um Größenordnungen abweichenden Materialien unter natürlichen Bedingungen zu bestimmen, welche Eigenschaften durch die bei den Laboruntersuchungen unberücksichtigt gebliebene Makrostruktur (Absonderungsflächen, Klüftung, feine Wechsellagerung) weitgehend beeinflusst werden. Ein weiteres Problem besteht in dem definitiven Charakter der bodenmechanischen Kennwerte, z.B. kann das bodenmechanische Verhalten von zwei, die gleichen Konsistenzkennwerte aufweisenden Böden unter den gleichen Randbedingungen (einwirkende Kräfte, Schichtenfolge, usw.) verschieden sein. Daraus folgt, daß das Problem auch in der Verwendung von die Böden nur ungenügend kennzeichnenden Kennwerten liegen kann.

- auch die Tatsache darf nicht vernachlässigt werden, daß sich die Laborergebnisse nur auf einen zu einem bestimmten Zeitpunkt bestehenden Zustand des untersuchten Bodens beziehen, diese Werte aber bei den Berechnungen als zeitlich konstant betrachtet werden. Die Änderung von Kennwerten wie Wassergehalt, Porenanteil mit der Zeit (z.B. während der Bautätigkeit) kann sich aber auch auf andere bodenmechanische Kennwerte wesentlich auswirken.

- wegen der kleinen horizontalen Erstreckung und der großen Variabilität der Gesteinskörper können die gemessenen Kennwerte nur in unmittelbar Nähe der Bohrungen als repräsentativ gelten.

- die aus dem Wasserleitungs- und Kanalisationsnetz in den Untergrund sickern Stoffe können über bodenchemische Reaktionen die Eigenschaften der Böden beeinflussen und dadurch den Sicherheitsgrad herabsetzen. Die einsickernde Flüssigkeit kann durch die Erhöhung der neutralen Spannungen den Spannungszustand im Boden ungünstiger gestalten.

Zusammenfassung

Die gründliche Untersuchung des Gebietes hat gezeigt, daß die Bearbeitung der Aufschlüsse unter rein bodenmechanischen Gesichtspunkten im Falle des Ávas-Hügels zu keinen zufriedenstellenden Ergebnissen führt, es ist notwendig, parallel mit den bodenmechanischen Untersuchungen auch die hydrogeologische-ingenieurgeologische Umgebung zu erforschen; man muß die Tektonik, die Makro- und Mikrostruktur der geologischen Einheiten bestimmen, die Beschaffenheit und die Strömungsverhältnisse des Grundwassers kennenlernen.

Zur Verfolgung der Bodenbewegungen und der Änderungen des Druckspiegels und der Beschaffenheit des Grundwassers wird die Errichtung eines Beobachtungsnetzes aus Grundwasserstands- und Inklinometer-Meßstellen vorgeschlagen.

Es ist notwendig, die Auswirkungen der verschiedenen bodenchemischen Veränderungen auf die Standsicherheit im Laboratorium zu untersuchen.

Es ist zu hoffen, daß die Durchführung der angedeuteten Forschungsarbeiten die Beseitigung des Widerspruchs zwischen den theoretischen Berechnungen und den praktischen Erfahrungen ermöglicht.

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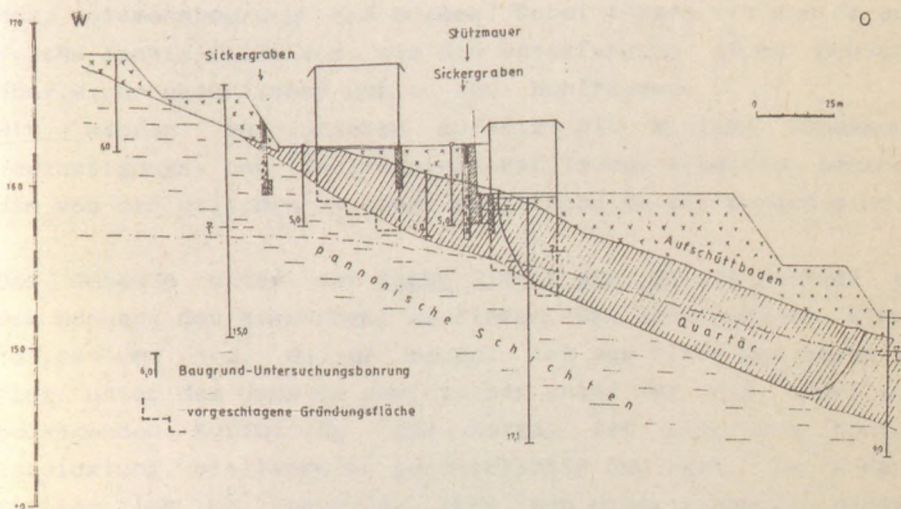


Bild 5.: Profil der östlichen Flanke des Avas-Hügels mit möglicher Rutschungsfläche

NEUARTIGE TIEFBAULÖSUNGEN BEI REKONSTRUKTIONSBAUTEN

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Die Tiefbauaufgaben von Rekonstruktionsbauten beanspruchen in Ungarn, besonders in Budapest, oft andere Lösungen als in Grosstädten allgemein. Neben den - meistens in Pest vorkommenden - Lückenbauten und den tiefen Baugruben auf dem Berggelände von Buda, ist die häufigste Tiefbauaufgabe der Rekonstruktionsbauten die Gründungsverstärkung, und zwar wegen der Volumenänderung des Bodens. Dabei kommen hie und da auch solche Spezialfälle vor, wie die Unterfangung eines Gebäudes über einer natürlicher Höhle, bzw. Hohlräumen.

Wir machen in unserem Aufsatz mit einigen Fundamentverfestigungs- und Baugrubenunterstützungs-Arbeiten bekannt, die von den üblichen Methoden abweichend gelöst worden sind.

Das Gebäude unter der Römer Floris Str. 50 ist in Buda, auf dem Südhang des Rosenhügel zu finden. Das Grundgestein dieses Teiles der sog. Budaer Mergel ist vom Alter her Eozän und liegt unter dem Gebäude unmittelbar unter der die Oberfläche bedeckenden Auffüllung. Der Mergel ist hier von toniger Entwicklung, stellenweise gesteinsartig und hart. An anderen Stellen ist er tonartig, aber von einer Formation grosser Festigkeit und ausgezeichneter Tragfähigkeit, die aber gegenüber Wasser sehr empfindlich ist. Infolge konzentrierten Wasserzuflusses zerfällt sie sofort. Die Stärke der Mergelformation beträgt nach geologischen Daten 50-60 m. Darunter befindet sich der sog. Nummuliten-Kalkstein, der auch vom Alter her Eozän ist und für Karstbildung sehr geeignet ist. In diesem Kalkstein befinden sich die durch das Thermalwasser entstandenen Höhlen des Budaer-Gebirges. Für

diese Höhlen sind grosse Räume und enge Verbindungskorridore charakteristisch, wobei von Fall zu Fall Spalten und Schlote auch in den Mergel, in die Nähe der Oberfläche hinauftragen. Das Dasein des Höhlensystems bedeutet hinsichtlich der Gebäude im allgemeinen keine Gefahr, die hinauftragenden Spalten aber können bei der Gründung Probleme verursachen.

Das Villen-Gebäude wurde um die Jahrhundertwende gebaut. Angeblich wegen einer Spalte oder eines Hohlraumes, gefunden im Laufe der Gründung, hat man die zwei Längswände auf grossen, die Einzelfundamente stützenden Gewölbe gegründet. Später hat man auch eine weitere Unterfangungskonstruktion eingebaut. Infolgedessen stand das Gebäude bis in die 60-er Jahre ohne Beschädigung, seitdem haben sich aber immer stärkere Risse gezeigt. Das aus dem kaputt gegangenen Versorgungsleitungen konzentriert abfliessende Wasser hat nämlich den Hohlraum unter dem Gebäude so erweitert, dass die ursprüngliche Unterfangungskonstruktion ohne Unterstützung blieb. Am Gebäude entstanden Beschädigungen in dm-Grössenordnung. Die originale Spalte war z.Z. unserer Untersuchung nicht mehr auffindbar, aber unter dem Gebäude existierte ein Hohlraum von mehreren Kubikmetern.

In Kenntnis obiger Tatsachen musste die neue Unterfangung die unbekannte Spalte und die Schichten unsicherer Tragfähigkeit völlig meiden.

Deswegen hat man neben den Tragkonstruktionen (Einzelfundamente, Stahlbetonträger) einen Kopfbalken gesetzt, der von mehrmals injektierbaren Ankern zur Seitenwand des Hohlraumes gespannt, dabei von schräg in die Seitenwand gebohrten und injektierten Stahlrohr-Mikropfählen unterstützt wird. Die Spannkraft wird durch die Anker 4-10 m weit vom Kopfbalken entfernt ins harte, unversehrte Mergel-Gestein übergeben. Die freie Seitenfläche des Hohlraums wird unter dem Kopfbalken von auf Torkretnetz gespritzten Beton stabilisiert. Der Untergrund des Hohlraums wurde in einer Stärke von etwa 4 m mit Injektierung stabilisiert. (Siehe Bild 1.)

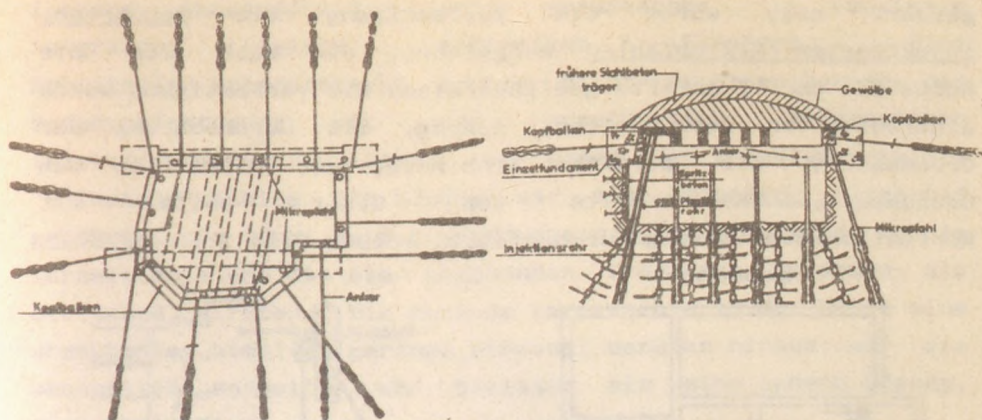


Bild 1: Verfestigung eines Gebäudes über einem Hohlraum mit Verankerung und Mikropfählen (Budapest, Römer Flóris Str. 50)

Die oben vorgeführte Arbeit war sowohl bezüglich der Problemstellung als in der technischen Lösung ohne Zweifel eine Spezialität. Häufig kommen aber Gebäudeschäden wegen der Volumenänderung des Bodens im Budaer Hügelgebiet vor. Der Oligozän Ton von Kiscell, die andere wichtige Grundsteininformation, und die obere verwitterte Schicht davon - wegen der Tonmineralzusammensetzung - neigt sehr zur Volumenänderung. Auf diesen Gebieten verursachen die natürlichen Schichtwasser, und zwar meistens die Wasser, die aus zugrunde gegangenen Versorgungsleitungen etc. stammen, - mit der Änderung der Sättigung des Bodens unter dem Fundament - massenhafte Gebäudeschäden.

Am Gebäude unter Fillér Str. 50/b haben sich seit Jahrzehnten Beschädigungen, die auf Gründungsfehler hinweisen, gezeigt. Diese konnte man früher auch mit Eingriffen wie Untermauerung, Mantelung, Einbau eines Stahlbetonholmes nicht eliminieren, so dass das Gebäude in 1984 lebensgefährlich geworden ist.

Da die Ursache der Beschädigung eindeutig die Volumenänderung des Bodens war, und die im allgemeinen akzeptierte Untermauerung in einer Tiefe von 2,0 m zu keinem Ergebnis

geführt hat, wurde die Verfestigung mit gebohrten, injektierten Mikropfählen ausgeführt, die auch für die Aufnahme der Zugkräfte geeignet sind. Die Verfestigung wurde einerseits von zwei Seiten, schräg, mit Durchbohrung der Grundmauer, mit direkter Einbindung der Pfähle in den Grundmauer, andererseits von einer Seite mit Kopfbalken-Schliessung durchgeführt. (Siehe Bild 2.)

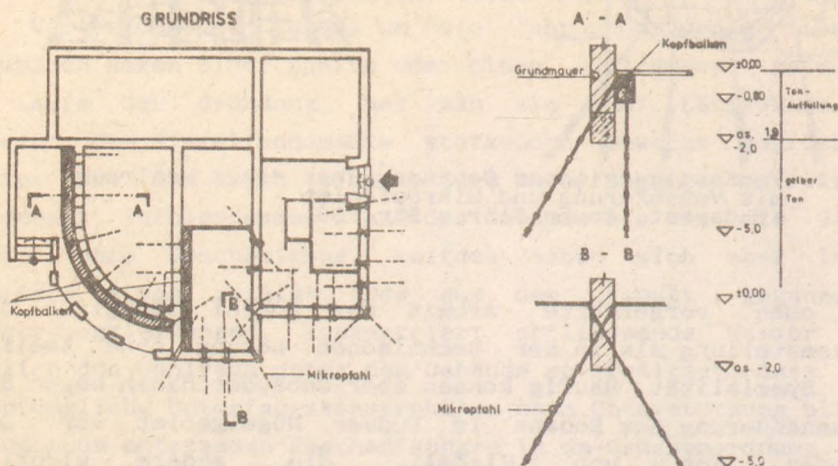


Bild 2: Gebäudeunterfangung bei einem wegen Volumenänderung geschädigtem Gebäude (Budapest, Fillér Str. 50/b)

Auf der Pester Seite wird bei Lückenbauten und tiefen Baugruben die am häufigsten vorkommende Lösung, die weltweit schon routinemässig angewandte Schlitzwand oder Bohrpfähle grossen Durchmessers, oft mit einer Verankerung kombiniert. Neuerdings wird das sog. Jet-Grouting Verfahren auch immer öfter angewandt. Hier erwähnen wir nur zwei solche Fälle, die man mit den obigen Verfahren nicht, oder nur wesentlich schwieriger durchführen können hätte.

In der Halle des Westbahnhofs in Budapest wurde ein 5 m tiefer Innenraum ausgestaltet. Innerhalb der Halle musste die Ausgrabung vertikal solch einer Keller-Seitenwand gesichert werden, wo bereits in unmittelbarer Nähe eine Böschung-Ausgrabung durchgeführt worden ist. Da der Boden eine

lockere Sandauffüllung war, verursachte die vertikale Ausgrabung einen sofortigen Einsturz. Eine Schlitzwand-Unterstützung konnte nicht in Frage kommen. Die Bodenverfestigung mit der ursprünglich vorgesehenen Injektierung war unlösbar, da wegen der Erdausgrabung die "Belastung" fehlte. Die Lösung war eine schräge, netzartig gebohrte, in drei Reihen angeordnete Wurzelpfahlwand, wo die Wurzelpfähle und die sie umgebenden Erdmassen zusammen als Stützwand wirkten. Die Methode verursachte in der Halle eine akzeptable, ziemlich geringe Störung; darüber hinaus war sie wesentlich schneller und billiger als eine andere Lösung. Siehe Bild 3)

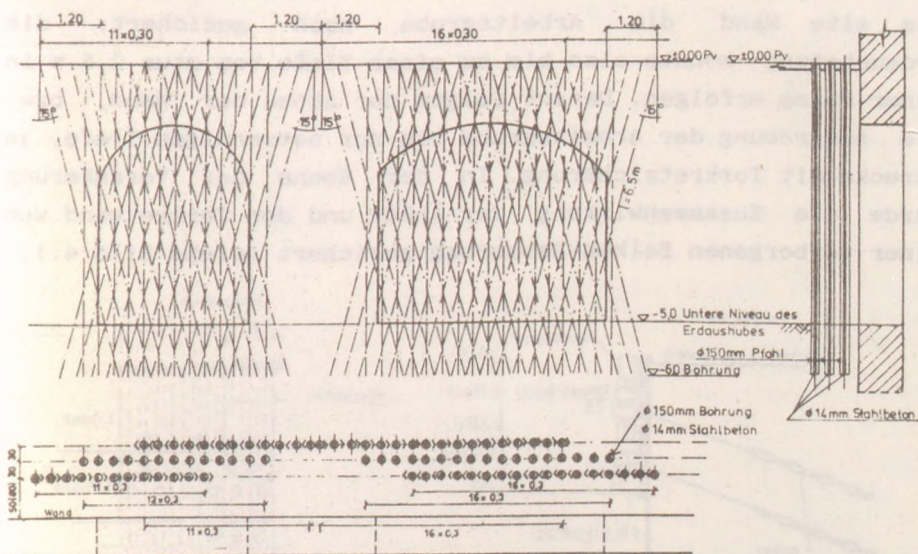


Bild 3: Baugrubenunterstützung mit netzartig gebohrten Wurzelpfählen in der Halle des Westbahnhofes Budapest

Die Residenz der Versicherungsanstalt Hungaria in der Vadasz Str. im 6. Bezirk von Budapest wurde auf einem Grundstück, umgeben von 4-5 stöckigen Häusern, mit einem 6,5 m tiefen Keller, gebaut. Von drei Seiten wurde die Sicherung der Gründung der existierenden Gebäude mit verankerten Bohrpfählen grossen Durchmessers gelöst. Auf der Strassenseite fiel die

Stein - und Ziegel-Kellermauer bzw. die Grundmauer der vorher hier stehenden Gebäude genau in die Ebene der projektierten Kellermauer bzw. des Fundaments. Die Unterstützung war also nur ausserhalb dieser alten Mauer möglich, wo es zur Ausgestaltung mit Pfählen grossen Durchmessers schon keinen Raum gab. Die Stützung der Arbeitsgrube wurde hier mit einer kombinierten Konstruktion, bestehend aus einem verankerten Mikropfahl und einer Torkretwand gesichert.

Die Mikropfahlreihe wurde ausserhalb der alten Wand, ca. 10 cm davon entfernt gefertigt, sie hat also nur etwa 25 cm vom Gehsteig weggenommen. Zur Zeit der Bohrung der Mikropfähle hat die alte Wand die Arbeitsgrube noch gesichert, die Erdaushub konnte also bis zu einer Tiefe von etwa 2,5 m in einer Phase erfolgen. Darauf folgte der Abbau der Wand, bzw. die Ausgrabung der Arbeitsgrube bis zur notwendigen Tiefe, je Strecke mit Torkretsicherung. In der Ebene der Verankerung wurde die Zusammenwirkung der Anker und der Torkretwand von einer verborgenen Balken-Bewehrung gesichert (siehe Bild 4.).

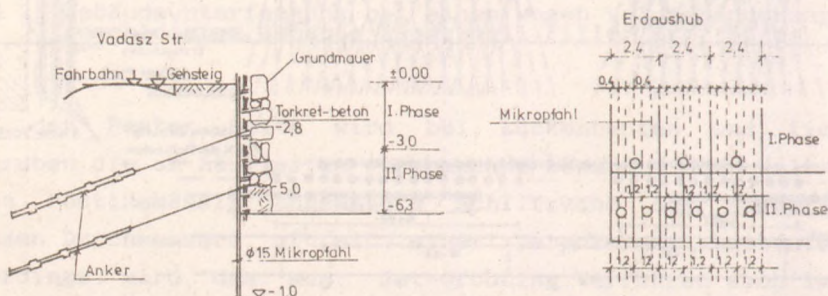


Bild 4: Baugrubenunterstützung hinter einer früheren Kellermauer mit verankerter Mikropfahlwand (Budapest, Vadász Str.)

SECTION 1: DEEP EXCAVATIONS

GENERAL REPORTER: PROF DR B. K. MAZURKIEWICZ

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Excavations are used normally as temporary measures which after finishing of the considered building or structure placed in an excavation are no more visible although they create considerable problems during construction stage, and their costs may reach 40 % of total costs of the whole undertaking (Stocker [14]). The methods or techniques used for the stabilization and protection of an excavation can be very complicated while in one place different types may be introduced to ensure a proper safety not only for the new structure but also for the existing structures or buildings in the neighbourhood.

During the last 20 or 30 years new developments have been introduced allowing on greater depths and better protection of excavation sheetings. Following structures can be mentioned, namely: sheet walls, soldier beam retaining walls, pile walls, slurry trench walls, prestressed ground anchors, soil nailing, underpinning, small bored piles and injection walls or slabs.

The sheet walls are very often used for the protection of excavations although their driving creates considerable problems due to big noise and vibrations. However, the last developments allow on a hydraulic forcing of sheets with forces up to 20,000 kN without noise and vibrations, giving thus a big chance for these structures in the future.

A large number of excavation protection structures consist of so called soldier beam retaining walls in which driven, bored or vibrated beams are the supports for horizontal sheets made of wood, steel or concrete. This system is still used, particularly when it can interact with an anchor system based for instance on prestressed ground anchors.

The pile walls are today very wide used due to the fact that the methods for execution of large diameter bored piles are improved and allow on protection of excavations of arbitrary shape in plane. Diaphragm walls made in the form of anchored pile walls with reinforced and intermediate piles allow to protect excavations of the depth up to 25 m. They can also be used as a part of the final structure.

In the recent time a wide use is made of the slurry trench walls in which also prefabricated reinforced concrete panels can be used. It seems today possible to have a large increase of this system due to introduction of a new excavation equipment, namely, special trench cutters which allow on excavation of 100 m deep trenches.

As an example of a slurry wall is in Fig.1 the 5200 square

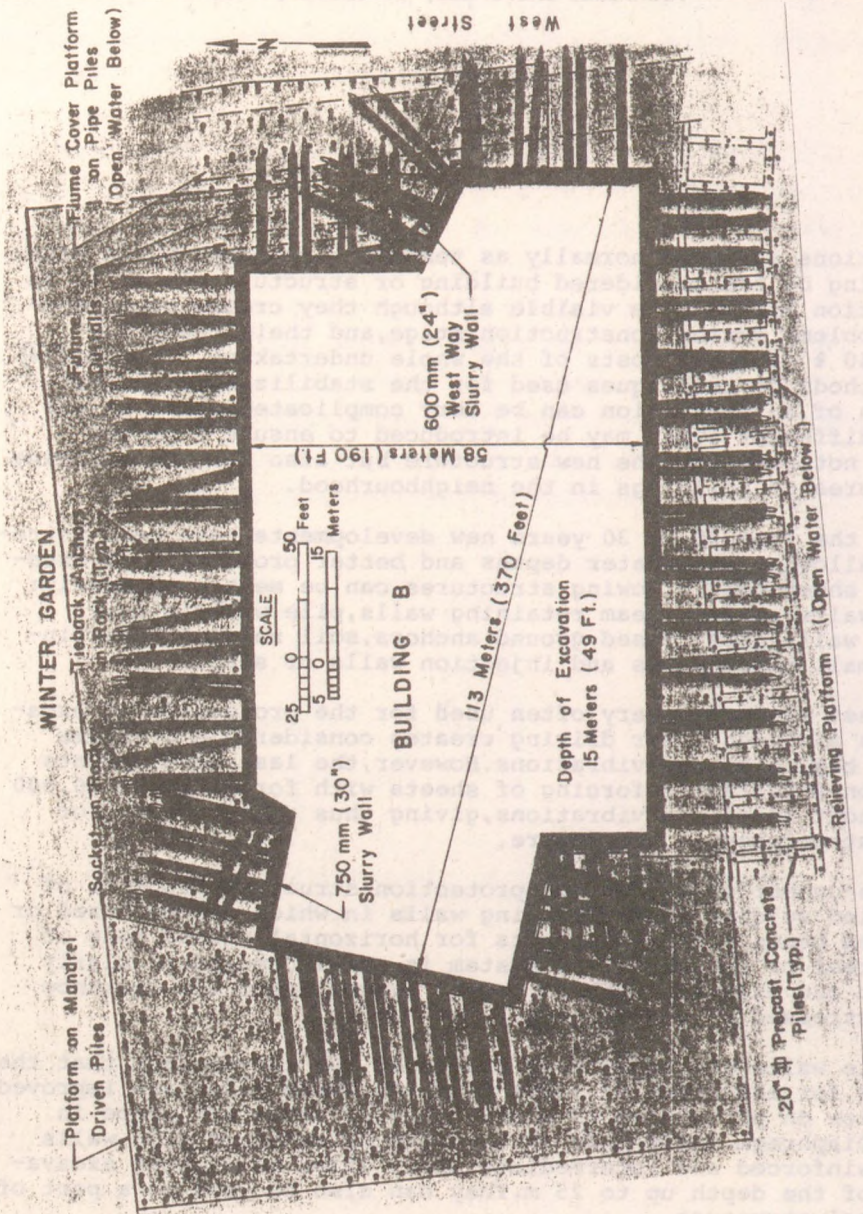


Fig.1 Excavation for the New York Financial Center Building B (TAMARO [15])

meter, 760 mm diaphragm wall given which was installed at the New York Financial Center Building B (Tamaro [15]) to plan dimensions of 113 meters by 58 meters and to a maximum depth of 15 meters. The wall was temporarily supported by tie backs anchored to rock ranging in capacity from 1,100 to 3,600 kN.

As mentioned all types of sheetings, particularly for deep excavations have to be tieback anchored if bracing can not be applied. Here the modern development of prestressed ground anchors has to be considered. These structures might be used as temporary or permanent one.

A very important excavation protection system, particularly in areas where existing foundations have to be supported is the underpinning made by injections or small bored piles. The introduction of jet grouting or high pressure grouting allows on the use of cement grout in all types of soils. Walls of arbitrary shape can be made under existing structures giving the chance through ground anchoring of a safe erection of the adjacent structure.

The introduction of injections give also the possibility to execute horizontal or vertical diaphragm walls and slabs cutting off the inflow of water to the excavation pit. This is a very important method, particularly in areas where the lowering of water level is from many points of view not recommended.

The last questions connected with the modern excavation protection methods concern the calculation methods of earth pressure, bearing capacity of piles, anchors and nails, as well as structure behaviour checking methods allowing on permanent delivery of information on the performance of excavation protection systems.

Papers delivered to the Conference concern many of the above mentioned structures and problems being a good contribution for further development of these very important elements of the foundation practice. The delivered papers can be characterized in this respect as follows.

The paper of RAYUK and MATVEYEVA [8] deals with the problem of braced excavations, namely, from the point of view of preservation of buildings located close to these excavations. Investigation has been made comprising measurement of deflections of the braced walls; stresses in the reinforcement; loads in the struts; settlements and lateral movements of the top of the bracing, and settlements of the building adjacent to the excavation. Experimental investigations of ground deformations caused by the movement of the retaining wall were also made in laboratory. Detail results from all investigations, presented in the paper, allowed on some general conclusions concerning mainly values and distribution of vertical and horizontal movements of the soil behind the wall, the knowledge of which is very valuable for the understanding of the work of a braced retaining wall.

Particularly important for the safety and stability of the bra-

ced walls is the knowledge of the earth pressure distribution and magnitude. This problem is studied in the paper of KOUDELKA [5] in which it is stated that the use of the Winkler's method for estimating pressures on retaining walls causes underestimation of forces and thus risk of failure. A new, so called Dependent Pressures Method, based both on the force and deformation conditions is proposed. A detail and critical discussion of this method allows to find a reliable and safe estimation of the action of a retaining wall. The method requires, however, some further investigations.

The paper of LAKATOS et al. [6] discuss techniques used for construction of subsurface structures for the Budapest underground lines, e.g.: cast in situ and prefabricated walls erected in an excavation protected by soldier beam retaining walls, bored pile walls with one intermediate pile and anchored soldier beam walls. Additionally open excavations were made using depression of groundwater by wellpoints while some slopes were stabilized by soil nailing which was also the subject of tests of its bearing capacity. The paper can be treated as a good case study while the loading tests of nails might be contribution to the understanding of stabilization of slopes using this method.

The planned underground construction in Bratislava concerns the paper of TURCEK et al. [11]. It deals with the question of estimation of the permeability of the subsoil in the vicinity of Danube to design the future excavation works. Tracer methods and pumping tests have been used allowing on the determination of the filtration characteristics (filtration velocities, filtration coefficient, underground water gradients, subsoil permeability, etc.). In addition an analysis was made of the possible underground flow of water contaminated due to expected use of chemical substances at the deep foundation pits. Hydrodynamic dispersion, adsorption, diffusion and half-life were considered.

The paper of BERMAN [1] concerns a wall protecting excavation and constructed by using bored 800 mm dia piles placed in two rows. The piles are bored cast-in-situ piles situated in a distance of two pile diameters along the row and one pile diameter between rows. Thus the gap between piles is about 0.44 pile diameter.

The main subject was the estimation through field and laboratory tests of the horizontal displacements of the wall loaded through a hydraulic jack at the level which corresponds to the position of the resultant of active pressure estimated according to the Coulomb's theory. The obtained results indicate that the horizontal displacement of piles of the second row (row from the supported earth) is 2 to 3 times higher than that of the first row (row from the excavation side). It was observed that the first row receives 20 to 30 % of the load and the second one 70 to 80 %. Further tests of pile rows connected by considerably stiff pile cap indicated equal displacements of pile rows, however, with increased bearing capacity in relation to the bearing capacity calculated according to the theory of elasticity.

It seems evident that the obtained results are influenced by the application of a rigid loading beam supported on piles of the second row. Thus the earth pressure normally acting on the piles of the first row could not be mobilized. May be tests made by mobilizing earth pressure, e.g. through excavation in front of the first row would deliver quite different results.

BLÜMEL and HARDER [2] describe bored pile walls with two or three not reinforced intermediate piles in a row used for excavation below ground water level. An analysis is made of the kinematic of the system in the case of two or three intermediate piles, as well as detail calculations of acting forces are presented. It is stated that due to the not sufficient friction forces required for satisfaction of the equilibrium conditions the system with two intermediate piles should not be considered at all while the system with three intermediate piles can be recommended if they can be acting in the form of vaults. The paper discuss some examples of bored pile walls introduced in the Federal Republic of Germany, and presents recommendations concerning the construction of these walls, particularly in difficult soil conditions. Generally the paper can be recommended for practical use when designing and constructing bored pile walls.

The paper of JANCHULEV et al. [4] concerns the use of precast reinforced concrete paneled cut off diaphragms during the construction of the underground in Sofia. It is foreseen that the cut-off diaphragms will be performed in alluvial gravel-sandy soils with a clay filler and underground water presence at different depths. To prepare the diaphragm wall construction investigation has been made of the bifunctional cement-bentonite slurries with chemical admixtures retarding the cement setting. Reological characteristics of the slurries and physico-mechanical characteristics of the hard-set product have been studied under laboratory conditions allowing on the estimation of the most appropriate slurry. Generally the paper presents a thorough description of a study which can be recommended for a wide use when estimating the slurry characteristics.

Deep slurry - walls are the subject of the paper of MANOLIU et al. [7] in which presented are cases of deep excavation protected by slurry-walls in urban and constrained areas, namely, excavations for underground tank (22.5 m deep), for foundation of a 120,000 kN jack (13.0 m deep), for underground structure of a lift testing tower (10.0 m deep) and for a tank for industrial polluted water (8.5 m deep). Generally the paper has to be recognized as a very valuable case study containing a full description of occurred design and construction problems of the very important excavation protecting structures.

DENEV and GERMANOV [3] consider in their paper the problem of decrease of stress effort of prestressed ground anchors due to creeping of soil. A theoretical background of this phenomena is presented allowing its quantitative estimation leading to important conclusions concerning the anchor length, anchor cross section and elasticity of steel in the anchor cable. The recommendation to use longer anchors but with smaller cross-section

tion, as well as smaller modulus of elasticity of anchor steel seems to be supported by the theoretical considerations given in the paper.

The problem of ground anchors, namely, their permanent checking is also the subject of the paper of WIETEK [13]. After a wide presentation of questions concerning a proper protection of prestressed anchors against corrosion, as well as a description of the anchor corrosion phenomena a report on the investigation of test anchors equipped with electrodes allowing on the measurements of the potential differences between the anchor steel and the electrode is given. A complex interpretation of all measurements allows to recommend the introduced method as a tool for determination of the state of the anchor during its construction and for checking if a corrosion takes place. In addition it allows to stop the corrosion through bringing an outside current to the electrode. Thus the whole system can be recognized as a good tool for anchor checking and for the anchor checking and for the anchor safety prediction.

The question of earth pressure is also analyzed in the paper of SHCHERBINA [9] namely, as the cohesive soil pressure estimated through centrifugal investigations. Assuming that the application of cohesive soils for backfilling the retaining walls will be increasing tests of cohesive soils lateral pressures on rigid and movable structures in the centrifuge were made using backfill with varying in different tests the density and moisture content. The test results allow the estimation of the lateral pressure coefficients for investigated kinds of non-uniform and combined backfills. They indicate also that the applied method can be valuable for the estimation of earth pressures in particular cases occurring in the engineering practice where the classical methods yield unsafe results.

The paper of STANCIU and SILION [10] concerns also the pressure of cohesive soils, however, under the influence of seismic forces. It presents a calculation method for the estimation of the nonlinear distribution of the active earth pressure on retaining walls being influenced by earth quake. Vertical and inclined walls might be considered according to the introduced nonlinear distribution of pressures and location of resulting forces. To try to introduce the method in practice it would be valuable to present a comparison between the results obtained using the existing methods and the proposed one.

A new approach to the problem of soil-structure interaction is presented in the paper of VILADKAR et al. [12] in which a study of the behaviour of an unsymmetrically loaded five storey two bay plane frame is made. An improved physical and constitutive modelling of the soil mass and structure is introduced by using hyperbolic stress-strain model for soil and a coupled finite-infinite, multi-element formulation for physical representation. As result of the analysis of the frame structure-combined footing - soil system subjected to both vertical and lateral loads a set of data concerning settlement, sway, contact pressures, base shear, bending moments was obtained, allowing on the estimation of the real behaviour of the designed structure.

The presented papers do not deal with one important problem raised today in the society, namely, the problem of preservation of the existing environment and the problem of environmental protection during the construction stage of the excavation and the final structure. Many questions can be asked among others if chemical grouts have to be still used; if ground water lowering should be accepted or substituted by other methods e.g. cut-off walls and slabs; if ground water flow should be stopped through a diaphragm wall; if very noisy pile and sheet wall driving or vibrating methods might be used, etc. The proper answer to these questions means considerable changes of the equipment used in the whole construction works, as well as changes in the design and construction methods.

During execution of excavations, particularly inside the town, a considerable problem arises when the soils are contaminated. It requires a fast excavation of these materials and their delivery to special waste storages. To avoid unpleasant surprises a special soil investigation from the point of view of its contamination has to be made in advance to allow the preparation of a proper excavation method and an adequate contaminated soil storage area.

It is quite evident that in the future the excavations will be made to larger depths to place underground storages, garages, power stations, etc. Recently only slurry trench walls can be made to the depth of about 100 m giving the possibility to follow the future requirements. It seems possible that other methods can be also improved to allow on larger depths, while the large ground water pressures might be taken by round or oval excavations in plane in which these pressures will cause compression of the excavation sheeting. Also the cement high pressure injection methods can be recognized as friendly for the environment and giving the opportunity to create cut-off walls for the water flow.

Concerning the future excavation protection methods from the point of view of the environment it is also probable that the soil nailing particularly for smaller depths to about 20 m will develop. It is recently used for ground strengthening in-situ, primarily for improvement of slope stability and support of excavations.

The "nails" are usually steel rods that are grouted into pre-drilled holes or driven using a percussion drilling device. Their length may be of the order of 50 % of the height of the excavation to be supported. The grouted bars can resist tensile forces that are limited by the lesser of the bar yield strength and the pull-out resistance provided by the friction that can be mobilized along the grout-soil interface. Shear and bending stress will be developed in the bars at magnitude that depends on the stiffness, of the bar-grout combination relative to the soil.

The problems connected with soil nailing concern the estimation of the ability of the soil to resist lateral loads acting trans-

versely to the nail axis not mentioning the determination of the size, spacing orientation and length of bars. The overall stability of the nailed soil mass can be analysed in the usual way i.e. assuming that the nailed soil mass represent a heavy retaining wall. It should be recommended to continue the investigation of the soil nailing method to increase the possible excavation depth and to find the optimum excavation wall protection method e.g. by its reinforcing and shotcrete lining. Concerning the bearing capacity of nails, tests as described in the paper of LAKATOS [6] should be continued.

Considering large diameter bored piles as friendly for the environment (low noise, cement as the main material) it seems possible to develop this method in the future, particularly when combining it with prestressed ground anchors installed with the progress of excavation. Here the proposal of BLÜMEL and HARDER [2] to use several not reinforced intermediate piles placed in the form of a vault which transfers the earth and water pressures on the reinforced piles and the ground anchors can be recommended. It is here proposed to use inclined rods drilled to a firm soil layer in which a prestressed ground anchor is executed. This proposal is very important particularly when the upper soil layers consist of very weak soils (e.g. fills). The future investigations should concern mainly the stability of pile walls, as well as their tightness particularly when lowering of ground water behind these walls can not be allowed.

The execution of deep excavation in urban areas requires underpinning of existing foundations. As mentioned before the jet grouting or the high pressure grouting seems to be today a method giving the possibility to obey all difficulties, however, for considerable small depths. Thus the investigations should concern first of all the possibility of increasing the excavation depth as well as the installation of ground anchors for the stability of the excavation wall supporting the underpinned foundation with its structure. An essential problem concerns here the erection of a tight membrane allowing on preservation of the water regime below the existing structure, as well as the protection of the structure against settlements and deflections. It seems correct to inject at least in two or three rows pressing injection rods at different inclination and in a distance allowing on the intersection of injected columns. Further investigations should concern mainly the grout content and grout strength, as well as the influence of soil conditions and parameters on the injection procedure. Of course a significant part of the research should concern the grouting equipment.

Other methods, mentioned at the beginning, are the subject of wide considerations particularly from the point of view of improvement of equipment used for driving of piles or sheet walls, for compaction of backfill and soils existing around the excavation, and for lowering of ground water level.

An important factor which has to be considered is the decrease of the period of excavation execution and the period of its utilization. It seems necessary to shorten these periods consid-

erable to avoid hindrances in the surrounding and to shorten the time when the building activities disturb the people living in the vicinity. This can be done exactly only through the above mentioned improvement of equipment, however, under one assumption, namely, that the new techniques will give the chance of a fast control of the work quality. This requirement is essential, particularly for the safety of the excavation protection.

The last question connected with excavations concerns the estimation of loads acting on different types of excavation sheetings and the estimation of the total stability of this type of structures. In many cases the problem of excavations can be also presented as the problem of slope stability. It is of course very difficult to connect with excavations all actually running investigations on distribution and magnitude of earth pressure. The same concerns the stability of excavation sheetings. However, many investigations e.g. in the field of harbour engineering (quay walls, retaining walls, etc.) indicate that the problems are still the subject of wide interest of scientists, and as indicated in the paper of KOUDELKA [5] and SHCHERBINA [9] require intensive continuation. The main task is of course to estimate the loads acting on the structure considering the full interaction between the excavation sheeting and the supported earth mass. Considering the modern approaches it is obvious that several generations of scientists will be able to investigate in the future these problems while the final satisfactory solution seems to be very far.

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SECTION 3: SURFACE SUBSIDENCE

INTERACTION BETWEEN SUBSURFACE CAVITIES, STRUCTURES AND BUILDINGS OBERFLÄCHENSETZUNGEN INFOLGE VON HOHLRAUMBAUWERKEN UND TIEFBAUTEN GENERAL REPORTER: DR L. MARTAK

Magistrat der Stadt Wien/Österreich

Eingrenzung des Themas und Zuordnung der eingereichten Beiträge

Den zunehmend enger werdenden Platzverhältnissen innerstädtischer Ballungszentren wird weltweit mit dem Bau von U-Bahnen, Tiefkellern und Hochhäusern entgegengewirkt. Diese Eingriffe in den Untergrund bestehen aus Entlastungen, die deutlich den Aushub einer Vorgängerbebauung übersteigen und aus Belastungen, die entweder weit unter dem Eigengewicht des ausgehobenen Bodens verbleiben oder stark darüber hinausgehen. In allen diesen Fällen werden die Verformungseigenschaften des Untergrundes für die benachbarte Bebauung und die Einbautensituation maßgeblich, was sich im Verein mit begleitenden Wasserhaltungen und Bodenverbesserungen als Oberflächensetzungen - in Ausnahmefälle auch als Hebungen - manifestiert. Diese Zusammenhänge bodenmechanisch zu charakterisieren ist das Thema des Generalberichtes.

In der zugänglichen Fachliteratur sind in den letzten Jahrzehnten eine Vielzahl von Fallstudien und Berechnungsvorschläge veröffentlicht worden, die von der Mitteilung bloßer Meßergebnisse über praxisorientierte rechnerische Abschätzungen bis hin zu rechenzeitintensiven FE-Analysen reichen. Auch die zum Thema eingesandten 11 Beiträge lassen sich in diese Gliederung einordnen. Fünf Beiträge beschreiben konkrete Baustellenerfahrungen, sechs Aufsätze stellen numerische Lösungsvorschläge vor. Die Arbeiten beziehen sich nicht nur auf Setzungen infolge Tunnelvortriebsarbeiten (5) und bei Gründungen sowie tiefen Baugruben (4), es wird auch in einem Beitrag über Grundwasserausgleich und in einem weiteren über Dammuntersuchungen im Bergsenkungsgebiet berichtet. Abgesehen von diesen beiden technisch interessanten Sonderfällen, erscheint es zweckmäßig das Thema einzugrenzen auf:

- Oberflächensetzungen aus dem Hohlraumbau (Tunnelvortrieb) und auf
- Setzungserscheinungen im Umfeld von Tiefbauten (tiefe Baugruben).

Gerade was die Setzungsprognose für ein konkretes Bauvorhaben betrifft, haben die numerischen Methoden in den vergangenen zwei Jahrzehnten unser Verständnis über Verformungen im Untergrund qualitativ wesentlich verbessern können, jedoch die quantitative Genauigkeit der Aussage bleibt infolge unzureichender Kenntnis und schwieriger numerischer Formulierung der Stoffgesetze weiterhin unbefriedigend. So erscheint es für einen Generalbericht nützlicher, die gesicherten Erfahrungen aus den Messungen auf den Baustellen phänomenologisch geordnet darzustellen und die Einflußfaktoren in ihrer gegenseitigen Abhängigkeit aufzuzeigen um für weitere Berechnungsansätze die physikalischen Grundlagen zu liefern.

OBERFLÄCHENSETZUNGEN AUS DEM HOHLRAUMBAU (TUNNELVORTRIEB)

Ein Tunnelvortrieb bewirkt für den verbleibenden Untergrund eine Neuorientierung seiner Einwirkungen und Widerstände. Je nach Entfernung vom Tunnelhohlraum und je nach Steifigkeit des Tunnelausbauwiderstandes (Spritzbetonschale, Extrudierter Beton oder Tubbing-Ausbau) durchläuft

der Untergrund von der vorhandenen hydrostatischen Erddruckbelastung bis zur vollen Ringdruckbeanspruchung alle Phasen einer dreiaxialen Spannungs- und Verformungsänderung, zu denen ihn seine bodenphysikalischen Eigenschaften in den gegebenen Entspannungs- und Wiederbelastungs- sowie Gestaltsänderungsgrenzen befähigen (Stoffgesetze). Der Masseverlust infolge des Tunnelhohlraumes im Verein mit dem Grundwasser und seinen Druckverhältnissen bewirkt Auftriebskomponenten, die sich dem neuen Belastungs- und Verformungszustand überlagern. Unabhängig vom Vortriebsverfahren - Schild oder NATM (Neue Österreichische Tunnelbaumethode) - verändert sich also der Beanspruchungszustand des Untergrundes und damit sein Verformungszustand, der bezogen auf bestimmte Beobachtungsebenen (Kellerebene, Einbautenebene, Geländeoberfläche, Geschoßebene in der Bebauung usw.) als räumliche und zeitlich veränderliche Setzungs-, in Ausnahmefällen auch Hebungsmulde gemessen werden kann. Ihre Form, die Tiefe, die Breite und der Zeitpunkt in bezug auf den Vortriebsverlauf usw. sind im wesentlichen abhängig von:

- den Untergrundeigenschaften
 - den Wasserhaltungsmaßnahmen
 - der Größe, Form und Anzahl der Vortriebe
 - der Art der Vortriebsmethode (Schild oder NATM) und Vortriebsgeschwindigkeit
- und nicht zuletzt von
- der handwerklichen Ausführungsqualität.

Untergrundeigenschaften

Der zeitlich unterschiedliche Verlauf der Setzungen von Schluffen und Tonen sowie schluffigen Feinsanden gegenüber den Sanden und sandigen Kiesen beim Tunnelvortrieb ist charakteristisch. Bezogen auf die Ortsbrustebene bzw. Ringschlußebene reagieren die Tone und Schluffe frühzeitig aber langsam, während die Kiese und Sande erst relativ knapp vor dem Vortriebsort, dafür aber schnell und vollständig die Verformung an die Geländeoberfläche weitergeben. Der zeitlich verzögerte Abbau der Porenwasserdrücke in den feinkörnigen Sedimenten erscheint dafür verantwortlich. Die grobkörnigen Sedimente bei Überlagerung > 2 bis $3 D$ (Tunneldurchmesser) reagieren durch die vollkommene Ausbildung eines starrkörperähnlichen Tragrings (Schutzzone) rund um den Vortrieb im Setzungsverhalten auf die Geländeoberfläche bereits gedämpft und nur bei geringen Überlagerungen $< 2 D$ zeigen sie schmale und tiefe Setzungsmulden. Zur Vorwegnahme von Setzungen bei kritisch erachteten Differenzsetzungen oder Gesamtsetzungen können Maßnahmen des Spezialtiefbaues wie Injektionen, Soilfracturing oder Jet-Grouting-Körpersetzungsmildernd eingesetzt werden. Der dadurch geschaffene Kunstboden wirkt außerdem setzungsverteilend und damit für die Setzungsmulde verflachend.

Wasserhaltungsmaßnahmen

Wasserhaltungsmaßnahmen beim Tunnelvortrieb werden aus der Lage der wasserführenden Schichten, ihrer Durchlässigkeitsverteilung (gespannt, halbgelastet oder frei) und ihrer Druckhöhe zum Vortriebsgeschehen bestimmt. In Sanden und Kiesen stellen sich bekanntlich flache, aber weit ausgreifende Absenkrichter ein, die je nach Mächtigkeit des Grundwasserleiters und seiner Steifezahl sowie des darunter liegenden feinkörnigen Grundwasserstauers zu Setzungen oder zu Hebungen führen können. Diese Verformungen manifestieren sich in der Regel mit flachen Mulden oder Hebungshügel. In feinkörnigen Grundwasserleitern (z.B. in gespanntes Wasser führenden Feinsanden) erfolgt durch die Auftriebsverminderung der

darüber befindlichen Untergrundzonen verstärkte Setzung insbesondere bei halbgespannten Leitern. Die Setzungsmulden werden steil und halten sich an den räumlich relativ engen Einfluß der Gravitations- oder Vakuumbrunnen.

Druckluft im Tunnelvortrieb wirkt bei feinkörnigen bis gemischtkörnigen Sedimenten als Wasserhaltung und Tunnelausbaustützung, wobei der freie äußere Grundwasserstand über den Ausströmungsort örtlich hügelartig angehoben wird (cm bis dm). Die Druckluft reduziert in feinkörnigen Böden die Setzungen des Vortriebs während ihrer Wirkungsdauer. Bei höherem Druck sind sogar örtlich Hebungen möglich. Nach Abschalten der Druckluft reagiert der Untergrund auf die verminderte Stützung seines Korngerüstes mit Setzungen, belastet allerdings dann eine bereits ausgehärtete Tunnelschale oder einen fertigen Tubbingausbau. Die Setzungsmulde über dem Tunnel bleibt kleiner als ohne Druckluft.

Größe, Form und Anzahl der Vortriebe

Je besser die Vortriebsform einer Stützlinie für die Untergrundeinwirkungen nahe kommt und je rascher die Zeit-Festigkeitsentwicklung des Tunnelausbaues erfolgt, desto geringer werden die Setzungen auf der Geländeoberfläche bleiben. Eine solche Profilanpassung an die mobilisierbaren Bettungsreaktionen des Untergrundes ist in der NATM in weiten Grenzen möglich, bei Schild- oder Preßvortrieben aber durch die Kreisform begrenzt. Große Querschnitte werden in der NATM durch Ulmenstollen, Kalottenvortriebe, Restausbrüche etc. unterteilt, wobei jede Profilerweiterung mit Setzungssprüngen (z.B. Strossensprung) bis zum Erreichen des Vollprofiles verbunden ist. Diese Setzungssprünge können durch provisorische in Spritzbeton ausgebildete Sohlgewölbe (z.B. Kalottengewölbe) wesentlich vermindert werden. Beim Schildvortrieb kann durch den Ortsbrustverbau bzw. die Ortsbruststützung (offenes Schneidrad, Suspensionsstützung, Erdbreistützung) die Setzung des Untergrundes in weiten Grenzen bis hin zu Hebungen bei zu starker Ortsbrustpressung verändert werden. Parallel laufende Vortrieb, Querschläge, Verbindungsstollen und Bodenpfeiler führen infolge der konzentrierten Beanspruchung des Untergrunds zu Setzungen, die bei Seichtlage (1 bis 2 D) in der Größe der Firstsetzung voll zur Geländeoberfläche durchschlagen. Bei Überdeckung $> 3 D$ verlaufen die Setzungen in einer gegenüber den einzelnen Firstsetzungen reduzierten Gesamtmulde über alle Querschnitte, wenn die gegenseitigen Abstände der Vortriebe $\approx D$ nicht wesentlich überschreiten. Die Breite der Setzungsmulde wird im allgemeinen durch die Tangenten an das oder die Tunnelprofile mit einer Neigung von $45^\circ + \gamma/2$ begrenzt.

Art der Vortriebsmethode und Vortriebsgeschwindigkeit

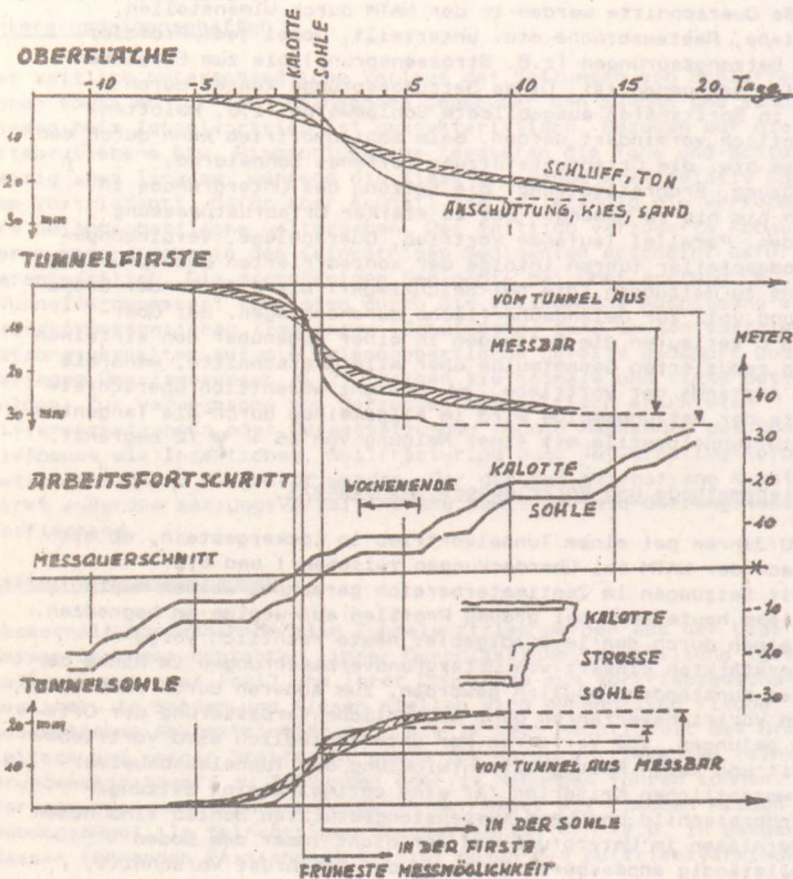
Während vor 20 Jahren bei einem Tunnelvortrieb im Lockergestein, ob mit Schild oder nach der NATM bei Überdeckungen zwischen 1 und 3 D prinzipiell mit Setzungen im Zentimeterbereich gerechnet werden mußte, lassen sich diese heute auch bei großen Profilen auf wenige mm begrenzen. Dies ist zum einen durch den im Stadtgebiet heute rechtlich vorgeschriebenen verstärkten Einsatz von Untergrundverbesserungen im Sinne der Schaffung eines Kunstbodens möglich geworden, zum anderen durch eine bei allen modernen Vortriebsverfahren ganz wesentliche Verbesserung der Ortsbruststützung gelungen. Vom Verfahren her unterschiedlich sind Vortriebsgeschwindigkeit und Zeit-Festigkeitsentwicklung des Tunnelausbauwiderstandes, die wesentlichen Kriterien für eine optimal kleine Setzungsmulde. Beim Erdbreischild und beim suspensionsgestützten Schild sind neben Vortriebshindernissen im Untergrund und der nicht immer dem Boden druckmäßig vollständig anpassbaren Stützung der Ortsbrust Vorschnitt,

Überschnitt, Konusform des Schildes und Ringspalt die Ursachen für Setzungen, die sich auch durch eine sofortige Ringspalthinterfüllung nur teilweise abfangen lassen.

Bei der NATM führt die dem Vortrieb und den Sonderstütz- und Sonder-sicherungsmaßnahmen an der Ortsbrust voreilende räumliche Entspannung des Untergrundes in Richtung Hohlraum zu den wesentlichen Verformungen und Setzungen. Eine Beschleunigung der Vortriebsgeschwindigkeit und damit eine Verkürzung der Ringsschlußzeit, was zu einer wesentlichen Ver-ringerung der Setzungen beitragen könnte, ist durch die notwendige Manipulationszeit zum Bodenabbau und zum Einbau der Bewehrung des Voll- oder Teilprofiles sowie durch die Zeit bis zur Erhärtung des jungen Spritzbetons eingeengt. Derzeit muß noch mit etwa 1/3 der Gesamtfirst-setzung vor dem Öffnen der Ortsbrust und mit etwa 2/3 der Setzungen während der Tunnelherstellung und danach gerechnet werden. Die Setzungsmulde an der Geländeoberfläche und an der Bebauung tritt dann je nach Überdeckungshöhe zeitlich verschoben und vom Ausmaß her gedämpft als Überlagerung aus den Einflüssen der Vorentspannung und den Maßnahmen des Tunnelausbaues (Ringspalthinterpressung, Ankerung, voreilende Verdichtung, Jet-Grouting-Gewölbe usw.) auf (Abbildung 1).

Abb.1

SETZUNGEN/HEBUNGEN BEI SEICHTLIEGENDEN TUNNELN IM LOCKERGESTEIN



Handwerkliche Ausführungsqualität

Abgesehen von den Fällen, in denen Hindernisse im Untergrund (Blockwerk, Fundamentreste, Baumstämme etc.) die Ortsbruststützung vorübergehend schwächen, sind ungünstige Setzungsverläufe mit baulichen Schäden in erster Linie auf Ausführungsmängel zurückzuführen. Weder beim Schildvortrieb, noch in der NATM sind Schlotbildungen zur Oberfläche, größere Setzungssprünge zwischen Kalotten- und Strossenvortrieben und für die Bebauung unvertretbar steile Setzungsmulden sowie Stützmittelverluste (Druckluft, Bentonitsuspension etc.) system- oder methodenbedingt, wenngleich diese auch oft als Rechtfertigungsgrund herhalten müssen. Handwerkliche Qualität ist in erster Linie eine Frage der Personalauswahl und des erforderlichen Geräteparkes und zwar nicht für die Vortriebsmaschinen selbst, sondern auch für die Vorsondierungen, Entwässerungsmaßnahmen und setzungsausgleichenden Spezialtiefbauverfahren. Die Kosten- Nutzenanalyse mag von Stadt zu Stadt eine unterschiedliche Wertung für Personal und Geräte ergeben. Sie entscheidet aber in Summe über den Vortriebserfolg und über die Verformungen an der Bebauung darüber und somit über die bevölkerungspolitische Akzeptanz des gewählten Tunnelbauverfahrens.

SETZUNGSERSCHEINUNGEN IM UMFELD VON TIEFBAUTEN (TIEFE BAUGRUBEN)

Setzungsmulden um tiefe Baugruben für Keller und Garagengeschosse, sowie um Gebäude mit hoher spezifischer Untergrundbelastung werden seit Jahrzehnten beobachtet und in der Fachliteratur systematisch dokumentiert. Es liegt darüber bereits ein umfangreicher Erfahrungsschatz vor, der phänomenologisch im folgenden skizziert werden soll.

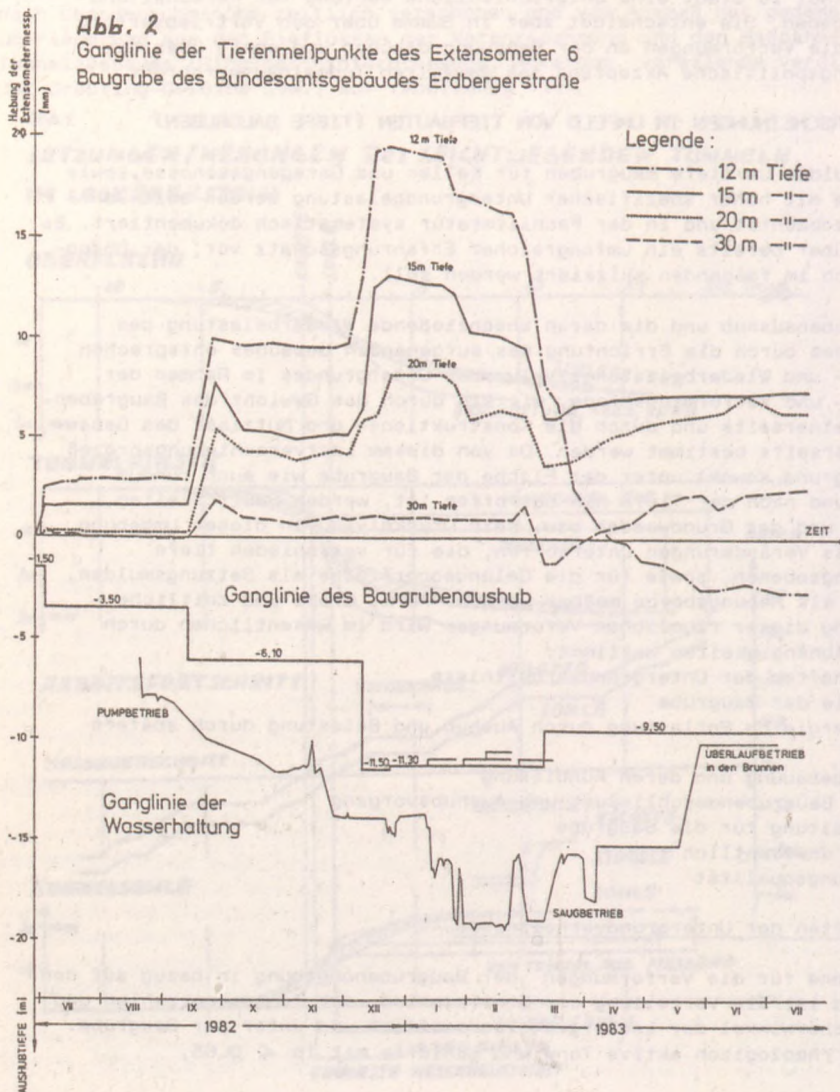
Ein Baugrubenaushub und die daran anschließende Wiederbelastung des Untergrundes durch die Errichtung des aufgehenden Gebäudes entsprechen einem Ent- und Wiederbelastungszyklus des Untergrundes im Rahmen der Spannungs- und Verformungspfade, wie sie durch das Gewicht des Baugrubenaushubes einerseits und durch die Konstruktions- und Nutzlast des Gebäudes andererseits bestimmt werden. Da von diesem Lastverschiebungsprozeß der Untergrund sowohl unter der Fläche der Baugrube wie auch rundum seitlich und nach der Tiefe hin betroffen ist, werden Baulichkeiten, Einbauten und das Grundwasser bzw. sein Druckniveau in dieser Umgebung gleichfalls Veränderungen unterworfen, die für verschiedenen tiefe Beobachtungsebenen, sowie für die Geländeoberfläche als Setzungsmulden, z.T. auch als Hebungsberge meßbar werden. Form, Größe und zeitliche Entwicklung dieser räumlichen Verformungen wird im wesentlichen durch folgende Abhängigkeiten bestimmt:

- Eigenschaften der Untergrundverhältnisse
 - Geometrie der Baugrube
 - Massenvergleich Entlastung durch Aushub und Belastung durch spätere Gebäude
 - Nachbarbebauung und deren Fundierung
 - Art der Baugrubenumschließung und Aushubsvorgang
 - Wasserhaltung für die Baugrube
- und nicht unwesentlich
- Ausführungsqualität

Eigenschaften der Untergrundverhältnisse

Entscheidend für die Verformungen der Baugrubenumgebung in bezug auf den Untergrund ist die Verteilung der Steifezahlen, der Konsistenzahlen und der Restscherwinkel der Lockergesteine seitlich und unter der Baugrube. Besonders rheologisch aktive Tone und Schluffe mit $I_p \leq 0.65$,

Es $< 10 \text{ MN/m}^2$ oder mit Restscherwinkeln $\varphi < 7^\circ$ können entweder beim Aushub relevante Kriecherscheinungen zeigen oder potenzielle Gleitflächen ausbilden, da der Untergrund hinter der Baugrubenwand neben einer Biegeverformung vor allem eine Schubverformung erleidet. Gleichgültig ob die Baugrube rückgeankert oder ausgesteift wird, mit zunehmender Aushubtiefe wandert der Untergrund verstärkt vertikal wie horizontal in Richtung Baugrube. Abhängig von der Anisotropie der Lockergesteine, wobei die feinkörnigen Böden wieder gegenüber den grobkörnigen Böden stärker verformungsmäßig reagieren, treten bei zunehmender Veränderung vom Ruhedruckzustand durch die seitliche oder vertikale Entlastung infolge wachsender Tiefe des Baugrubenaushubes exponentiell zunehmende Verformungen ein. Sie führen seitlich der Baugrube zu bleibenden Setzungsmulden und innerhalb der Baugrube zu Hebungsbergen, die aber durch eine spätere Gebäudelast wieder kompensiert werden können (Abbildung 2).



Geometrie der Baugrube

Je mehr der Baugrubengrundriß von einer geschlossenen Kreisform abweicht, um so aufwendiger werden die Sicherungsmaßnahmen gegen horizontale Verformungen und um so größer die Setzungsmulden außerhalb. Das durch den Aushub entstandene Massendefizit führt allerdings auch bei noch so starren kreisförmigen Aussteifungen zu Hebungshügel, die allerdings bei nachfolgendem Hochbau wieder verschwinden können. L- und T-förmige Baugruben mit einspringenden Ecken führen in diesen Bereichen trotz eckumschließenden Zugverankerungen zu örtlichen Hebungen, oder wenn die Verankerungen verabsäumt wurden, zu starken Setzungsmulden der Geländeoberfläche und an dort befindlichen Baulichkeiten. Bei der Tiefe nach abgetreppten Baugruben können an den einzelnen Geländesprung-sicherungen innerhalb der Baugrube infolge unterschiedlicher Sohlniveau-entspannungen waagrechte Risse (z.B. an Pfählen oder Spritzbeton-sicherungen) auftreten, die sich bei nachfolgender Belastung wieder schließen.

Massenvergleich Entlastung und Belastung

Je größer das Massendefizit durch den Baugrubenaushub wird, desto ausgeprägter werden die Setzungserscheinungen rund um die Baugrube. Diese werden allerdings um so geringer ausfallen, je steifer der Baugrubenverbau ist und je konzentrierter die Baugrubenränder von Hebungen erfaßt werden. Wenn auf dem Baugelände bereits Bebauungen bestanden haben, so kommt es erst dann zu merklichen Setzungstendenzen außerhalb der Baugrube, wenn die Entlastung infolge Baugrubenaushub das Gewicht des Vorgängerbaues überschritten hat. Für die Größe der Reaktionen ist wieder die tiefenmäßige Verteilung der Steifezahlen und der Konsistenzzahlen bzw. der Lagerungsdichte verantwortlich. Das Gewicht der nachfolgenden Bebauung bleibt bei mehrstöckigen Tiefkellern und mittleren Verbauungshöhen (< 25 m) in der Regel kleiner als das des Aushubes. Die seitlichen Setzungsmulden bleiben bei Wiederbelastung der Baugrubensohle bestehen, ja verstärken sich noch, wenn wie bei Hochhäusern, die Bebauung das Aushubgewicht deutlich übersteigt. Die Reaktionen treten bei kiesig sandigen Böden nahezu gleichlaufend mit dem Massendefizit oder Massenzuwachs auf, bei feinkörnigen Böden treten deutlich verzögerte Reaktionen infolge des verzögerten Porenwasserdruckabbaues auf.

Nachbarbebauung und deren Gründungstiefe

Je tiefer die Nachbarverbauung im Verhältnis zum Baugrubenaushub gegründet ist, desto weniger kommt es zu vertikalen Verformungen. Insbesondere bei Tiefgründungen (Schlitzwände, Pfähle etc.) ist diese hebungs- und setzungsvermindernden Wirkung zu beobachten. Erst bei schweren Lasten durch Hochhausbauten wird ein Durchstanzen des Untergrundes neben dem Nachbarbau in Form eines Setzungssprunges feststellbar (Abbildung 3). Ein steil nach außen verlaufender Setzungstrichter ist feststellbar. Flach gegründete Nachbarbebauung ist beim Baugrubenaushub besonders von Hebungstendenzen betroffen, wobei je nach Baugrubenverbau die außerhalb liegenden Setzungsmulden das Haus zeitweise in eine Verdrehung oder Zerrung bringen können, je nach dem ob eine Platten- oder Streifengründung vorliegt. Für Streifengründungen und Einzelfundamentgründungen sind Schubrisse in den Fensterstürzen und Brüstungsflächen solcher Nachbarbebauungen charakteristisch.

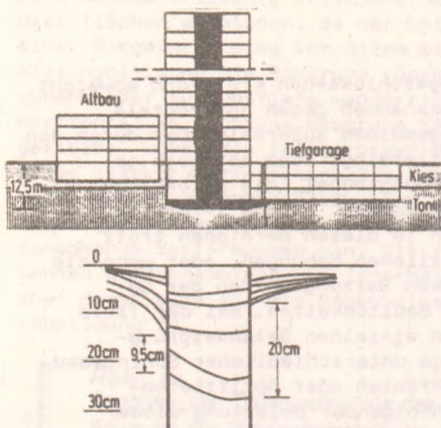


Bild 3. Setzungen infolge Hochhausbelastung bei unterschiedlicher seitlicher Belastung (D. Siroh und R. Katzenbach)

Art der Baugrubenumschließung und Aushubsgeschwindigkeit

In welchem Ausmaß und in welcher Form die Hebungs- und Setzungsverteilung außerhalb einer Baugrube eintritt, ist eine Funktion der Steifigkeit und der Abstützung der Baugrubenumschließung. Alle biegeweichen und stark nachgiebigen Verbauten (Spundwand, Bohrträgerverbau etc.) bewirken zwar geringe Hebungen an der unmittelbaren Nachbarschaft, aber um so merkbarere Setzungen im weiteren Umfeld. Insbesondere wenn Bodenanker den Verbau halten, kommt es ganz entscheidend auf deren Länge, gegenseitigen höhenmäßigen Abstand, auf das Kriechverhalten der Verpreßkörper und auf die Vorspannung der Anker an. Je kleiner die Aushubsschritte, je stärker die Vorspannung der Anker und je länger ihre freien Stahllängen über die rechnerische Mindestlänge hinaus gewählt werden, desto weniger wandert der Verbau in die Baugrube und desto geringer bleiben die ausliegenden Setzungsmulden. Die Setzungsmulden haben ihr Maximum im allgemeinen über den Enden der Verpreßkörper, das um so kleiner wird, je weniger die Anker und damit der Verbau in Richtung Aushub kriechen. Feinkörnige Böden zeigen die Verformungen deutlicher als grobkörnige. Auch steife Baugrubenverbauten zeigen deutlich horizontale Verformungen, wenn zwar volle Einspannung im Untergrund gegeben ist, aber durch große freie Auskragungen eine Drehbewegung der Wandfüße eintreten kann. Aufgelöste Pfahl- oder Schlitzwände reagieren oft trotz starker Einzelglieder wie weiche Baugrubenverbauten, da die Bodengewölbe zwischen den Traglelementen bereits vor der Sicherung mit Spritzbeton Entspannungen und Verformungen in Richtung Baugrube hinter sich haben. Baugrubenverbauten mit Stützbermen und abgetreppten Aushüben neigen besonders zu Verformungen, die sich als Setzungen außerhalb manifestieren, wenn die Sicherheitsbeiwerte der Gleichgewichtsnachweise nur knapp erfüllt werden. Besonders tiefliegende, Aussteifungs- oder Ankerhorizonte sollten bauseits rasch eingebracht und an die Baugrubenwände kraftschlüssig angeschlossen werden, da die Entspannung des Untergrundes mit zunehmender Aushubtiefe überproportional anwächst. Je schneller ausgehoben und je unnachgiebiger ausgesteift wird, desto geringer werden die Setzungsmulden außerhalb der Baugrube werden.

Wasserhaltung der Baugrube

Wenn in Anpassung an die Aushubtiefe das Gewicht des Grundwassers entfernt wird, kommt es entweder zu Setzungen oder zu Hebungen in der Baugrube und ihrer Umgebung, je nachdem ob die unterhalb des abgesenkten Grundwasserspiegels liegenden Lockergesteine auf die Entlastung in Summe stärker reagieren als die von der Auftriebsverminderung erfaßten Untergrundzonen darüber. Dieser Prozeß verläuft in Abhängigkeit zur Geschwindigkeit des Porenwasserdruckabbaues für jede Bodenschicht unterschiedlich schnell und ist innerhalb der Baugrube noch vom jeweiligen Aushubsniveau bestimmt. Außerhalb der Baugrube liegende Fläche reagieren auf eine innenliegende Wasserhaltung entweder nur durch den Entlastungsvorgang infolge Gewichtsverlust des Wassers, oder es kommt zu gleichartigen Verformungen wie innerhalb der Baugrube, wenn die Grundwasserabsenkung über die Baugrubé hinausreicht. Werden artesische Grundwasserleiter entspannt, treten die Setzungen darüber als enge tiefe Mulden auf. Kommt es zu Grundbrüchen innerhalb der Baugrube, sind Setzungstrichter außerhalb der Baugrube, mitunter direkt neben dem umströmten Verbau die Folge. Die Größe der Setzungen und Hebungen rund um eine Baugrube, wenn sie einer Wasserhaltung zuordbar sind, dürfen als empfindliches Indiz für die Wirksamkeit des gewählten Wasserhaltungskonzeptes gewertet werden.

Ausführungsqualität der Baugrube

Die Setzungen oder Hebungen der Baugrubensohle, des Baugrubenrandes und der weiteren Umgebung werden sofort von den Erfahrungswerten anderer Baustellen abweichen, wenn der Verbau säumig dem Aushub nachfolgt, die Brunnen der Wasserhaltung Erosion oder Suffosion zulassen, oder die Anker im Untergrund Kriecherscheinungen über das Normmaß erleiden. Den Baustellenmessungen kommt daher die Warnfunktion und die Sammlung der Entscheidungsgrundlagen für eine zeitgerechte Abänderung der Bauausführung zu.

WERTUNG DER EINGEREICHTEN BEITRÄGE

BOGDANOVA, SAVITSKY, SHEYNIN (USSR)

Numerical estimation of the effectiveness of measures to stabilize the ground massive when tunneling in the vicinity of buildings.

Die Autoren beschreiben eine FE-Analyse beim Tunnelbau in bebautem Gebiet mit Berücksichtigung einer Tieffundierung in der Nachbarschaft, sowie einer Bodeninjektion und der Tunnelarbeiten selbst. Es werden die bodenphysikalischen Parameter für die durchgeführte Vergleichsrechnung mit injizierten und nichtinjizierten Untergrundverhältnissen angegeben, einschließlich ihre Auswirkungen auf das zwei- und dreidimensionale Verformungsbild. Auch der Einfluß auf bestehende Gründungen wird diskutiert, wobei die zusätzlich dabei auftretenden Setzungen genannt werden.

CALINESCU, PISOI, WINTZE (R)

Subway tunnel with multiple functions on the Balcescu Boulevard, Bucharest.

Der Beitrag beinhaltet eine Zusammenfassung der grundbautechnisch interessanten Arbeiten an der U-Bahn in Bukarest im Laufe der letzten 15 Jahre.

DACHOWSKI (PL)

The influence of sinking underground structures of foundations by the Caisson Method on the surrounding ground mass and neighbouring buildings.

Die Setzungen rund um einen kreisförmigen Caisson von 40 m Innendurchmesser und ihre Ursachen werden in diesem Beitrag sehr eingehend beschrieben. Dabei werden je nach Abstand vom Caisson drei Zonen verschiedenartiger Bodendeformation genannt, und zwar:

1. die Gleitzone unmittelbar rundum mit vollständiger Veränderung der Bodenstruktur
 2. die Bodenzone mit Zerrissen außerhalb
 3. das weitere Umfeld mit Setzungen nur im Millimeterbereich.
- Insbesondere wird auf den Einfluß der Wandreibung des Caissons hingewiesen, dessen Berücksichtigung in den einschlägigen Richtlinien nicht vorgesehen ist. Abschließend werden sehr nützliche baupraktische Herstellungsempfehlungen für Caissons angegeben.

FADEEV, BAITASOV (USSR)

Additional bending of surface structures due to horizontal straining of the soil base.

Beobachtungen an Baulichkeiten in der Nähe von Kohletagbaugruben ließen horizontale Zerrungen im Untergrund erkennen, wodurch benachbarte Gebäude unter Biegebeanspruchungen gekommen waren. Die Autoren beschreiben Laborversuche, rechnerische Modelle und ausgedehnte In-situ Messungen zu diesem Phänomen.

FOTIEVA, ANTZIFEROV (USSR)

Design of Multilayer linings of the underground parallel circular-tunnels.

Verbundtubbinge aus Stahl und Beton und die Hinterfüllung zum anstehenden Boden können als Mehrfachschalenprobleme angesehen werden. Unter Benützung konkreter gemittelter Bodenkennwerte und Steifezahlen werden die rechnerischen Ergebnisse der Schnittgrößen von Mehrschalentubbingen mitgeteilt.

KERTESZ, PAAL, PRAJCZER (H)

Groundwater flow across metro-structures.

Für die U-Bahn-Linie 3 in Budapest war es in den wasserführenden sandigen Kiesen notwendig, einen Grundwasserausgleich einzurichten. Die Autoren berichten über die ausgeführten Grundwasserausgleichssysteme, die aus beidseitig des U-Bahntunnels hergestellten Schachtbauwerken bestehen mit Verbindungsleitungen unter der Tunnelbodenplatte des U-Bahn-Bauwerkes. Von den Schächten aus wird das Grundwasser mit Horizontalfilterbrunnen erfaßt. Die Vorteile dieses von FÖMTERV in Budapest entwickelten Systems werden mitgeteilt.

LIANG, SOBHANIE (USA)

Finite Element analysis of tunnel behavior in squeezing ground.

Die Autoren stellen für Tunnel im drückenden Gebirge ein FE-Programm vor, dessen Stoffgesetz aus einem modifizierten CAM-CLAY - Modell besteht. Das Stoffgesetz kann sowohl elasto- plastische Materialeigenschaften, wie Kriechvorgänge, erfassen. An einem Beispiel werden die Einflüsse aus Kriechen und Porenwasserdruck rund um einen Tunnelhohlraum dargestellt.

MALINOVA, PAVLIK, MASIN (USSR)

Stress-Strain analysis of karstic subsoil-slab (strip) footing system.

Der Beitrag bringt eine Analyse zu Platten- und Streifengründungen in durch Verkarstung gefährdeten Untergrund, fußend auf einem elastischen Mehrschichtenhalbraum. Die Methode soll sich zur Erkennung der Möglichkeiten der Wiederherstellung von Gebäuden über Verkarstungserscheinungen eignen.

PROCHAZKA, DOLEZEL (CSFR)

Global stability of tunnel heading.

Die Autoren befassen sich mit einer Anpassung der FELLENIUS-Methode des Böschungsnachweises auf eine Stabilitätsabschätzung der Tunnelortsbrust. Es wird versucht mit der Annahme eines physikalisch plausiblen Bruchkörpermodells in Form eines Rotationsparaboloides, charakterisiert durch einen Faktor k eine einfache numerische Lösung zur Definition der Sicherheit des Tunnelvortriebes zu finden und mit Meßergebnissen aus dem U-Bahn-Bau in Prag zu vergleichen.

TOL, BRASSINGA (NL)

The construction of the building for a railway tunnel in Rotterdam.

Es werden drei Fallstudien aus dem Bau eines Eisenbahntunnels in Rotterdam, Stadtzentrum, mitgeteilt. An die informative Beschreibung der Untergrundverhältnisse folgen die grundbautechnischen Auflagen für Wasserhaltungsmaßnahmen um weitreichende Grundwasserabsenkungen und damit vor allem Setzungen an den benachbarten Bebauungen vermeiden zu helfen. Alle drei Fallstudien geben einen anschaulichen Einblick in die Problematik der Baugrubenabdichtung, der Baugrubenranddeformationen und über den Erfolg setzungsmindernder Maßnahmen durch vorgespannte Baugrubenaussteifungen.

VAN IMPE, MENGE, WOLSKI (B, PL)

Stabilizing canal dikes in a mine subsidence Urban area.

Die Autoren berichten über Sickerlinien und Stabilitätsuntersuchungen an mehrfach erhöhten Deichen eines Kanals durch ein ausgeprägtes Bergsenkungsgebiet im Stadtbereich von EISDEN in Belgien. Die Begutachtung der durch die mehrfachen Erhöhungen in ihrer Sicherheit geschwächten Deiche umfaßt u.a. Aufschlußbohrungen, Porenwasserdruckbeobachtungen und bodenphysikalische Laboruntersuchungen. Die nach dem Verfahren von BISHOP durchgeführten Gleitkreisuntersuchungen erbrachten mögliche progressiv weitergehende Versagensmechanismen in den Deichen. Die Verbesserung der Stabilität erzielten die Autoren mit Bodenvernagelung und Drainagepfählen durch eine Versteilung der Sickerlinie in den Deichen. Sehr aufschlußreich werden auch die erforderlichen Korngrößenverteilungsbänder für die Kiespfähle mitgeteilt.

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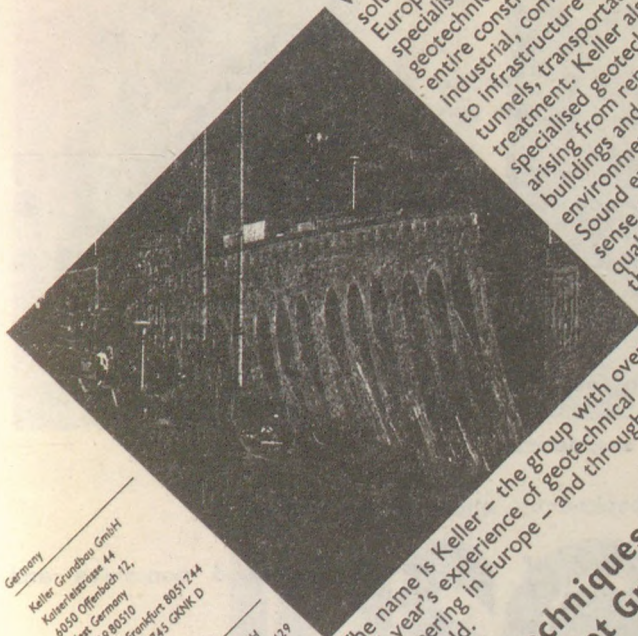
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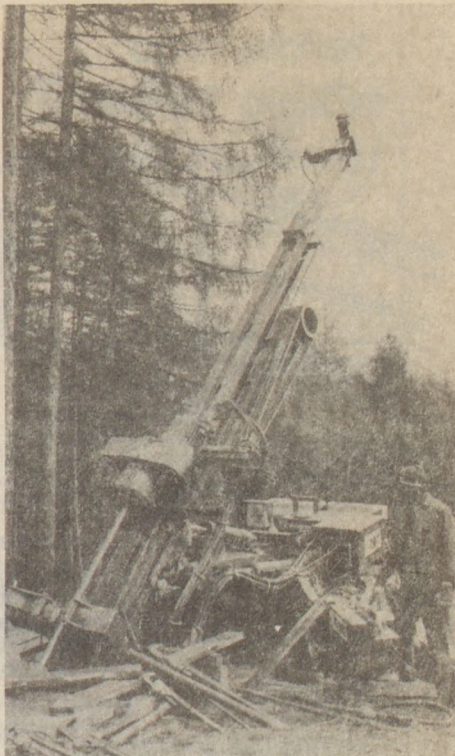
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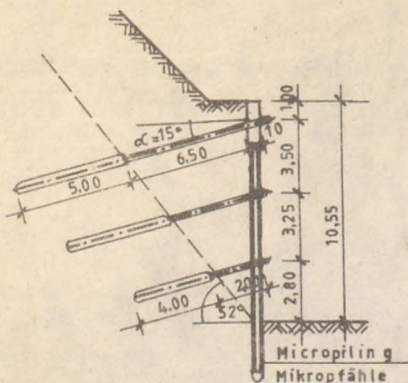
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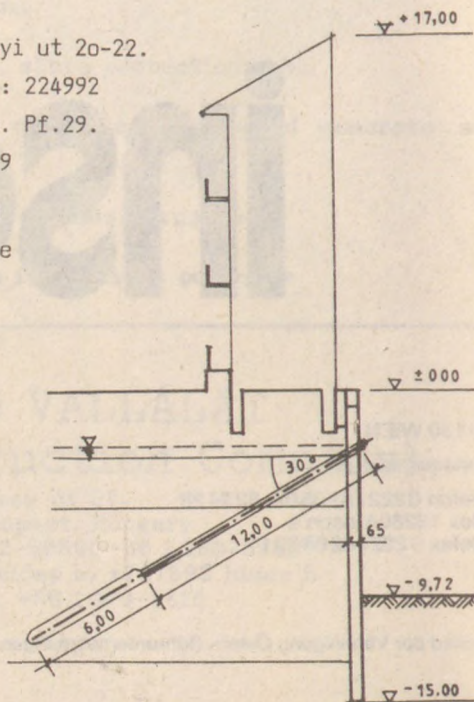
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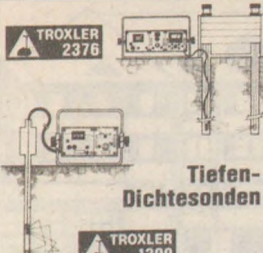
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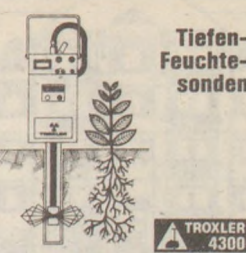
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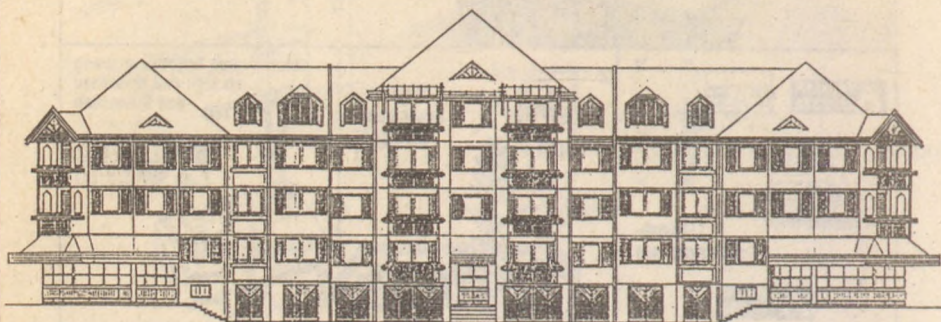
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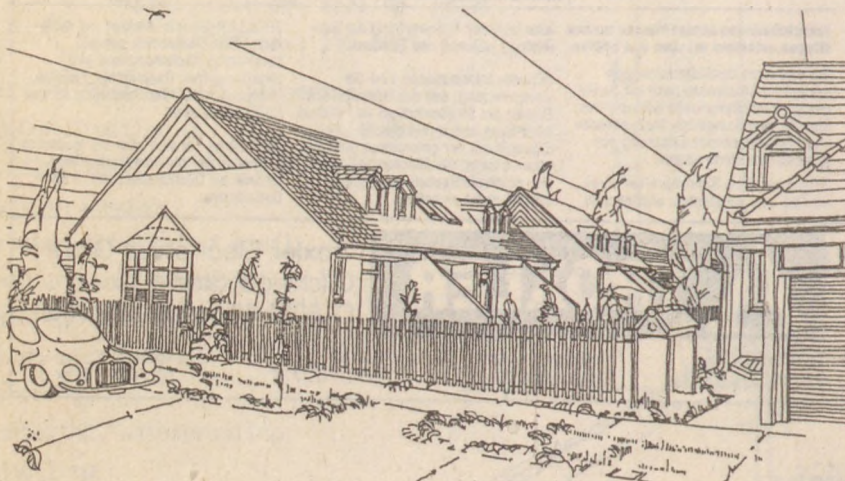
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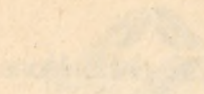
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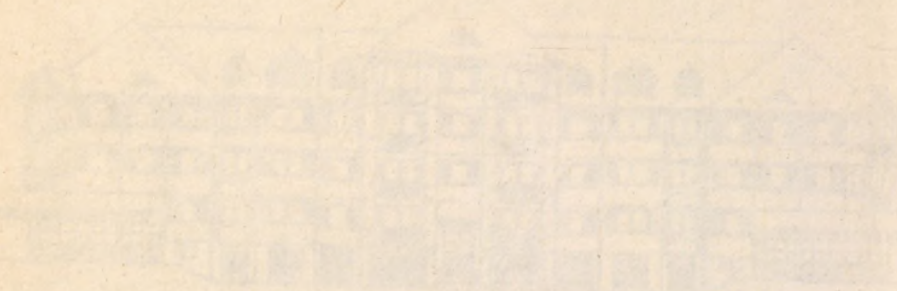


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