Soil mechanics and foundation engineering R&D for roads and bridges.

A summary of activities in the Northern and some Western European countries.

Bengt Rydell

May 1996
Close co-operation in the field of geotechnical research has existed for many years between the Swedish Road Administration (SNRA) and the Swedish Geotechnical Institute (SGI). In 1993 a seminar on road design, construction and maintenance related R&D was held with invited researchers from the Nordic countries. As the offshoot of discussions between SNRA and SGI, an international seminar on soil mechanics R&D for roads and bridges was found to be valuable. The objective of this seminar was to stimulate and encourage co-operation between European countries.

An invitation was send to a ten countries in the Northern and Western of Europe. The seminar was arranged by an Organizing Committee with participants from the SNRA and SGI. The meeting was held in November 16-18, 1993, in Sigtuna, Sweden.

This report contains papers, National Reports on ongoing R&D and other publications used at the seminar.

Linköping in May 1995

Bengt Rydell
Editor
SGI Varia 437

Seminar on Soil Mechanics on R&D for Roads and Bridges.
National Reports, Literature and Technical Papers

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1. NATIONAL REPORTS - R&D-ACTIVITIES

The participants were required to write a document describing the ongoing and planned research in each country about the chosen topics. These National Reports were then distributed to the chairmen of the plenary sessions for the preparation of a summary report. The National Reports as well as the summary reports was compiled and distributed to the participants at the beginning of the seminar.

In the following, the National Reports are published in the original form provided by the representative from the participating countries. In addition, references to research programmes and annual reports from research organizations are given.
National Report

R&D activities

Belgium
Belgium

References:

**Dynamic soil improvement methods** ¹)
Prof Dr ir W F van Impe, ir Wim Haegeman, Prof Dr M R Madhav, ir P Menge

**Analysis and settlement of dilating stone column reinforced soil** ¹)
Prof Dr ir W F van Impe, Prof Dr M R Madhav

**Load transfer through a gravel bed on stone column reinforced** ¹)
Prof Dr M R Madhav, Prof Dr ir W F van Impe

**DMT-measurements around PCS-piles in Belgium**
Research ass H Pfeiffer, Prof Dr ir W F van Impe, G Cortvrindt, M Bottiau

¹) Abstract and conclusions included. A copy of the paper can be ordered from the Swedish Geotechnical Institute, S-581 93 Linköping.
The soil mechanics' research in the Flemish part of Belgium, concentrated at Ghent University, is dealing with several topics such as:

a) SASW
b) Calibration chamber testing for vertical cyclic loading on model piles
c) Acoustic penetrometer development
d) Pile drivability analysis with excess pore water pressure-completed models
e) Waste disposal soil improvement by heavy tamping
f) Dilatancy effects in stone columns
g) Resonant frequency and vibrocompaction methods
h) DMT - as an evaluation of pile installation quality.

On almost each of those topics, papers or doctoral work have been published recently. Some of those papers are enclosed here.

With respect to R&D for roads I believe the most relevant research topic in this list is probably the SASW-research.

As a separate enclosure some results and general ideas about this topic have been given.

Sincerely yours,

Prof. Dr ir W.F. VAN IMPE.
ABSTRACT

Among the permanent soil improvement methods, an important category of techniques is dealing with the application of longitudinal and shear waves to the ground layer to be improved. Some of those methods are only meant for superficial or undepth soil layer compaction, many others although can also be classified among the deep soil improvement methods. The aim of this paper is to discuss last mentioned techniques establishing some of their particular requirements, advantages and disadvantages.

CONCLUSION

During the last decade, the use of soil improvement techniques has become, without doubt, one of the greatest challenges for the foundation engineer.

In the design of an adequate soil improvement method for a specific application, the deciding factors must be the available experience of the contractor concerned, the influence on the environment, the total energy cost and the expected improvement of the soil characteristics.

In many cases, economic reasons dictate that the more common soil improvement techniques are preferred over the more sophisticated foundation systems with which deeper, resistant layers are reached. Therefore, it is becoming increasingly important to understand clearly the technical possibilities and the geotechnical background of each improvement technique. Many of them began and were further developed starting only with experimental data.

As it appears for the moment, the most relevant steps in understanding soil improvement have been made with respect to vibratory compaction techniques. Many of the other methods increased comparably their justified share on the market of equivalent foundation engineering jobs, but usually less has been done in order to gather good data for a more profound analysis of understanding.

A new promising geotechnical engineering field related to soil improvement is the one concerning waste disposal treatment. This topic has been already included in some more elaborated research programs; one can fortunately expect therefore quickly progressing understanding to become available in the near future.

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ANALYSIS AND SETTLEMENT OF DILATING STONE COLUMN REINFORCED SOIL

W.F. VAN IMPE (1) and M.R. MADHAV (2)

ABSTRACT

Use of stone columns or granular piles in end bearing conditions for improving the bearing capacity, settlement, and resistance to liquefaction of soft clays or loose deposits has become common practice. Most of the available methods for stability analysis and for prediction of settlements of granular pile reinforced soil, are based on an elastic approach. In this paper the densified stone column material is considered to be at the limit yielding condition and hence dilating. Results obtained bring out the importance of incorporating the dilatancy effects on the prediction of settlement and the stresses on the stone column and the soil. Induced lateral stresses in the soil adjacent to the column are shown to be of the same order as the vertical stresses. The predictions of settlements based on the proposed approach appear to agree reasonably well with measurements.

CONCLUSIONS

Optimal design of stone columns requires an optimum stress concentration factor. The dense granular material dilates while yielding at peak stresses. Granular pile reinforced soil in here is analysed through a unit cell consisting of a stone column surrounded by the in situ soil. The model proposed incorporates the dilatancy of the stone columns material and the axial symmetric geometry of the problem. Results obtained show the significant beneficial effect of the dilatancy on the settlement reduction and stresses transferred to the pile. As a result, even at only 0.5 % dilatancy, the settlement of the reinforced soil is further reduced compared to a case in which the column is supposed to yield at constant volume (critical state condition). The stress ratio K of the in situ soil at the column-soil interface is close to unity indicating conditions very different from Kc condition. The predictions compare well with measured settlements so validating the approach presented here.

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LOAD TRANSFER THROUGH A GRAVEL BED ON STONE COLUMN REINFORCED SOIL

MADHIRA R. MADHAV 1 AND W.F. VAN IMPE 2

ABSTRACT

Treatment of soft or weak deposits with stone columns involves providing on top a dense gravel bed as a working platform and as a drainage layer acting as a stiff raft. It is often presumed to be rigid while no data are available to validate this statement. A simple model is proposed here for the analysis of such granular layer covering the stone column reinforced soil. The response of the system is shown to depend on the relative stiffness of the gravel bed. The load transferred to the stone column varies significantly with the relative stiffness of the gravel bed to those of the column and the soil. The design criterion proposed here ensures the gravel bed to deform more uniformly.

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CONCLUSIONS

Improvement of ground with a regular array of stone columns is commonly resorted to in case significant reduction in settlement is desired. A unit cell is analysed as typical of the treated area. The design usually implies uniform settlement of the stone column and the soft soil. A simple model for a gravel bed laid over the stone column reinforced soil is proposed and analysed for both plane strain (granular trench) and axi-symmetric (stone column) conditions. The variation of settlements with distance in a unit cell are shown to be dependent on the shear stiffness (product of shear modulus and the thickness) of the gravel bed, the relative stiffness of the stone column to that of soft soil, and the spacing of the stone columns. The load transferred to the stone column by the gravel bed also varies with the above specified parameters. For the covering gravel bed over stone columns to be considered rigid, the relative stiffness ratio, $\lambda_c$, should be less than about 0.2. For higher values of $\lambda_c$ differential settlements could be significant.

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DMT-MEASUREMENTS AROUND PCS-PILES IN BELGIUM
EVALUATION DES PIEUX PCS EN BELGIQUE BASEE SUR DES ESSAIS DMT

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SYNOPSIS: This paper evaluates the soil stress changes around a Socofonda PCS auger pile using the dilatometer test. Results of three test sites are reported. The evaluation of the execution parameters and other external influences are gathered through DMT-measurements during installation of the pile. The paper further describes a comparison of the results of soil investigation at different stages before, during and after pile installation.

INTRODUCTION

The bearing capacity, especially the shaft friction, of auger piles is strongly dependent on the execution parameters of the pile. PCS-piles (Pressurized Concrete Screw-piles) installed with a continuous auger are brought to depth causing no or a very limited soil displacement. During casting the concrete, an additional pressure is applied on the fresh concrete. For this type of pile, the execution parameters are the downward force during penetration \( F_d \), the torque \( M_0 \), rotation speed (downwards) \( n_0 \), downward velocity of the auger \( v_0 \), upward velocity \( v_u \), upward force \( N_u \), concrete pressure \( a \), the ratio diameter auger to diameter stem, the pitch (for all the discussed piles, \( p \) was 45 cm) of screwing down, and the quality of the concrete and the way of casting. This is an important factor governing the arching effect and determining the real fresh concrete pressure in equilibrium with the total horizontal soil stresses. By the use of hyperplastifiers, the \( W/C \) ratio can be limited to 0.45 and the cubic strength nowadays reaches 45N/mm² and higher.

All of these parameters are continuously measured (each 80 mm) during pile installation.

TYPE OF PILE

Generally the PCS-auger piles are using an inner stem diameter of 100 mm. During casting of the concrete an overpressure of 2 to 4 bar is applied on the fresh concrete, while the auger is regained slowly. This procedure doesn't cause vibrations. After casting the concrete, the reinforcement is brought into the pile using a vibrator. Eventual difficulties can be avoided using a greater inner stem diameter. So the reinforcement can be placed inside before casting the concrete. The outer diameter for such piles ranges between 35 and 45 cm. The high torque (100 kNm) that can be applied avoids excavating too much of soil and allows for penetration in resistant bearing layers. The degree of soil displacement can be deduced out of the overconsumption (occ) of concrete, defined as:

\[
\text{occ} = \left( \frac{V_b - V_p}{V_p} \right) \times 100 \text{ (in %)}
\]

in which:

\( V_b \) = concrete volume consumption

\( V_p \) = theoretical volume of the pile with known nominal dimensions.

This parameter (occ) gives a mean value and hides local effects such as lenses or layers where a higher excavation and/or soil displacement resulting in varying concrete consumption exists. For the discussed piles, the overconsumption was 16 % for the test site at Oudenaarde, 28 % for Dendermonde and 95 % for Dool test sites. The high overconsumption at Dool is mainly due to the fact that the upper layer (0 to -8.00) is composed of dredged material and to the presence of another very soft layer on the levels from -9,70 to -10,30.

SOIL TEST PROGRAM

For each pile, the following test soil results are gathered: (CPT) electrical cone penetration test before execution of the pile; (DMT-A) DMT-test before installation of the pile at 1,5 times pile diameter out of the center of the pile. In addition, during installation of the pile, a DMT-B test is performed with the DMT-blade installed at a fixed depth. The A-reading of the DMT curve was considered equivalent to an oedometer time-deformation curve. Using Casagrande's log fitting method: \( t_{50%} \) was determined before the start of the piles installation. Pile installation started when the decreasing ratio of A-readings became less than 5 kPa/hour. By this, consolidation and relaxation, due to the installation of the DMT blade did only have a negligible influence on the measurements lateron during pile installation. Finally a (DMT-1-C) test after installation of the pile at 1,5 times pile diameter out of the center of the pile was performed. The membrane is oriented towards the pile shaft.

For the orientation of the blade, one has two possibilities:

- radial position (membrane towards the pile). The advantage here is the direct measurement of horizontal stress variation. For large soil displacements, arching effect can occur around the blade,
- tangential position: the advantage is a smaller disturbance of the initial stress field around the pile. On the other hand as long as no clear relationship exists between the principle stress changes, the
The interpretation of such DMT readings with tangential blade orientation mainly stays difficult.

The aim of this research program was to evaluate the effect of PCS pile installation, in sandy layers, on the surrounding soil stress field.

DISCUSSION OF THE TEST SITES

Dendermonde-test site

The results of this test site, with all of the difficulties linked normally to each new research experience have been discussed in an earlier paper Peiffer et al (1991).

Dendermonde-test site

The results of the field tests are given in Fig. 1 a,b,c and 2.

Fig. 1a. DMT-results at Dendermonde

Fig. 1b. Initial CPT at test site

From the linearly increasing CPT in the sand layers where the DMT (1-B) is installed, it becomes evident the sandlayer was normally consolidated. The starting DMT A-reading is slightly higher than expected probably due to the local increased stress field around the blade. The DMT-membrane is directed towards the pile centre. The screwing-in energy resulting in

Fig. 1c. DMT-IB result at Dendermonde

Fig. 2. PCS-pile installation parameters at Dendermonde test site

very high downward penetration in the cohesive top layers, induces excess pore water pressures and "heavy liquid pressures" which compensate a normally expected soil relaxation. When the auger passes by at DMT-level, the whole of the remoulded soil column along the auger being more or less in suspension, induces for the rest of the screwing down movement a remarkable total stress increase (water pressure increase). One can easily explain an input of total stress increase of order of \( \leq 30 \) kPa. Obviously, the water overpressure fades out with time. During the casting process the total stresses, induced by the fresh concrete are detected, especially again starting at the level of the blade. The final DMT A-reading is apparently flattening out at about 160 kPa, being almost 50 kPa higher than the starting value. From this point of view this pile system would be somewhat beneficial to the soil-condition. One however must be careful since only DMT A-readings, performed some days after pile installation would indicate reliable more results. In Fig. 2 DMTI-A/I-C one sees such difference between the DMT-test before and after full pile installation indicating that there is almost no change in horizontal stress index and constrained modulus.
Two piles were examined at this site; piles n° 205 and n° 207. The results are gathered in Fig. 3, 4 and 5.

From the Fig. 4b, the (tangential) DMT-1B analysis on pile n° 205 indicates that soil arching is built up gradually during screwing down of the auger, resulting in a general overall (gradual) decrease of the A-value.
with some peaks in between, probably due to collapses of former arches while the auger continues to penetrate. The further general decrease after DMT A-reading during casting the fresh concrete, only shows the gradual out-facing influence of the soil arching with increasing fresh concrete weight.

From the readings for pile n• 207, with radial DMT-IB readings, a lot of interesting differences with the previous analysis are shown. Over the first five meters, the auger penetrates with a rather small torque, but with a high downward velocity through the loose silt/silty sand. This results this type of dry and only very slightly cohesive material in a dramatic decrease of the total stress combination felt by the DMT blade. The fresh concrete passing the level of the DMT, the concrete pressure influences tremendously the A-reading of the DMT-total stress in radial direction. Such overpressures are again fading out in time, partly compensated by the increasing fresh concrete weight above the DMT-measurement level.

CONSIDERATIONS ON PILE INSTALLATION EFFECTS

Our main interest in this kind of research is to finally understand much better the shaft capacity of screw piles. For this purpose, the pile installation effects have to be evaluated more carefully with respect to the effective soil stress changes around the shaft. Comparison between DMT-A and DMT-C readings gives an indication for the degree of such stress changes in terms of total stress. Referring to the horizontal stress index (table 1), the results of such comparison are presented for the three test sites.

Table 1. Comparison of DMT-stress index

<table>
<thead>
<tr>
<th>Site</th>
<th>Measured Concrete Overconsumption</th>
<th>Kp.after inst.</th>
<th>Soil type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Doe!</td>
<td>95%</td>
<td>1,30</td>
<td>medium dense sand</td>
</tr>
<tr>
<td>Oudenaarde</td>
<td>28%</td>
<td>1,20</td>
<td>medium dense sand (NC)</td>
</tr>
<tr>
<td>Dendermonde</td>
<td>16%</td>
<td>1,00</td>
<td>medium dense sand-slightly (OC)</td>
</tr>
</tbody>
</table>

Moreover, the increase of DMT-horizontal total stress in the top layer and the decrease near the pile tip are remarkable. This is discussed in an earlier paper Peiffer, Van Impe, Cortvrindt, Van Den Broeck (1991). Because also the pile installation parameters are available, a proposal could be to calculate the idealized specific screwing-in energy E\text{sc}. The use of this specific energy value in relation to the pile installation parameters is suggested in Van Impe (1988).

The today's analysis in this research program is also going out from continuous pressure measurements at the DMT-B blade during pile installation. Together with pile installation parameter, a more complete stress field analysis so became available.

CONCLUSIONS

In this paper the dilatometer is described as a tool that could help to evaluate the influence of pile execution parameters on the soil condition around the pile shaft. For PCS-piles some increase of horizontal stress is mainly dependent on the OCR, soil type, dilatant character of the soil, and predominantly on the installation details. It can be deduced that a too low auger penetration velocity and losses of time during installation can affect largely the final stress state around the pile.

REFERENCES

Van De Velde, K. en Van Hoye, P., 1992, Onderzoek naar het schachtdraagvermogen van boorpalen uitgaande van dilatometerproefresultaten, Gent

Van Impe, W.F., 1988, Considerations on the auger pile design, 1ste International Geotechnical Seminar: Deep Foundation on bored and Auger Piles, 7-10 June, Ghent.

National Report

R&D activities

Denmark
Seminar on Soil Mechanics and Foundation Engineering Research and Development for Roads and Bridges

Information on the research and development work at the Danish Geotechnical Institute and in short at other Danish research organisations

1. Introduction

The present status of R&D in the actual area in Denmark is very much restricted by lack of allocated funding for the building industry and in particular concerning the topic area. However, the very high activity related to the on-going construction activities at Storebælt and Øresund for major combined railroad and road traffic systems to some degree improve this situation.

The projects being of high technical complexity are placed in Geotechnical Category 3 (EC7) calling for high requirements to the extent and quality of geotechnical investigations, design and construction control. This has caused a large number of project related development projects and introduction of new methods and equipments. Furthermore, multidisciplinary integration of specialists work has provided background for progress in niches and at interfaces between, f.inst., geology and geotechnics.

Formally, much of the detailed information on soil and rock properties and soil-structure interaction is restricted by clients. Fortunately, a number of articles prepared on the basis of the investigations performed for the fixed link across Storebælt have been released as shown in Encl 1.

The Danish Geotechnical Society are publishing dgf-Bulletins containing the topics related to the present symposium. DGI is a main contributor to the different series on soil mechanics, foundation engineering, design practice, engineering geology and engineering geology. The present list of dgf-Bulletins is copied in Encl 2.

In Denmark the below-mentioned research institutes and organisations are the most important contributors to R&D for soil mechanics and foundation engineering related to road and bridges:

- DGI (GD) Danish Geotechnical Institute
- VD (SV) Danish State Road Directory - The State Road Laboratory
- DSB Danish State Railroad Directory - Bridge and Environmental Departments
- AUC Aalborg University Centre, Department for Water, Soil and Environmental Science, Foundation Laboratory
- TUD Technical University of Denmark, Department for Geology and Geotechnical Engineering
- (IVTB) Institute for Roads, Traffic and Planning of Towns
- DIA (GD) Danish Engineering Academy - Geotechnical Department

Beside these a number of governmental organizations and private companies are working especially in environmental sciences. However, much research and development are not specifically related to road and bridges, but deal with utilization or disposal of residues from coal-fired power plants. Also in this matter DGI has contributed with a number of references, Encl 3.

Furthermore, R&D in ground water and soil pollution is the topic for the Danish Environmental Research Programme funded by 50 mill DKK over the next 3 years. Their is no connection to R&D for road and bridges in this programme.
2. DGI contributions to R&D on Soil Mechanics and Foundation Engineering for Road and Bridges.

DGI contributions may be exemplified by the number of 84 internal R&D projects for 1993 with a budget of 5.3 mill DKK covering personal education, participation in national and international R&D activities, in standardization and conferences, specific development of methods and equipment and publication and documentation activities. The costs in taken from the overhead of the average yearly turnover of approx 120 mill DKK. DGI does not receive any general governmental support, but deals commercially as a self-supporting fund using possible profit for R&D-activities within our field of interests.

2.1 Geotechnical Design inclusive advanced numerical and statistical methods and field and laboratory testing.

Major DGI projects are related to international standardisation and it may of course be questioned being a development activity. However, we consider the international cooperation as strategic for our development as well as for our marketing. The activities have a total budget of 750,000 DKK in 1993.

DGI director Niels Krebs Ovesen is chairman for the drafting panel of Eurocode 7: Geotechnical Design, General Rules. A number of DGI specialists are involved in related activities e.g. standardisation of laboratory and field work, correlation of design practices, principles and requirements for safety and serviceability.

In connection to the 50 year anniversary of DGI an international symposium on "Limit state design in geotechnical engineering" was arranged 26-28 May 1993. The papers, keynote lectures, general reports and other contributions is published in dgf-Bulletin 10, Vol 1-3, pp 700.

DGI is lead partner in the partly EEC-financed SPRINT programme: "Quality Assurance in Geotechnical Testing" with the purpose of establishing a network of collective industrial research centres in 10 European countries and preparing commonly accepted recommendations on testing practice based on evaluated local input. The work is now being disseminated in the countries as an important part of the technical background for the coming European geotechnical laboratory standard. A comprehensive report on "Recommended Practice in Geotechnical Laboratory Testing" 1993-03-31 with four task group reports are available from DGI on soil identification tests, strength tests, compressibility tests and rock tests.

Most of the more specific research and development at DGI are related to specific consultative projects and are typically self-financed additional studies or publication activities. Other important activities are development of computer-guided automatic test equipment and specialized test methods for customized field and laboratory testing. Furthermore, DGI are introducing advanced calculation methods for earth pressure and soil structure interaction in drained and undrained conditions on PC's and work stations using self-developed or commercial software. The costs for these activities are approx 770,000 DKK in 1993.

The other Danish research centres contribute to local evaluation under the auspices of the Danish Geotechnical Society. Approximately 75% of R&D activities at the technical universities TUD and AUC and the Engineering Academy DIA are related to geotechnical design and testing. However, the activities are of a general nature and seldom directly related to roads and bridges.
The Foundation Laboratory at Aalborg University Centre is partner in the international research project MAST II / MC5 Monolithic Coastal Structures funded by EEC with a budget of 320,000 DKK. FL also participates in a multilateral research project together with Delft Geotechnics and Norwegian Geotechnical Institute on "Dynamite Response of Granular Material to Wave Load" funded with 850,000 DKK by STVF (Danish Technical Scientific Foundation).

Furthermore, STVF has funded with 1.3 mill DKK the FL research project "Behaviour of Structures Subjected to Dynamic ground motion". The project involves generation of shear and compression waves by shock loading of piles, vertical and horizontal movements of piles during driving, propagation of surface waves energy dissipation and interaction between vibrating structure and soil.

Actual Ph.D.-projects consider "Clay Till Properties" (Helle Trankjaer), "Response of Dynamic loaded Foundations" (Morten S. Rasmussen) and "Quality of CPT-testing" (Kirsten Luke).

The research at the Technical University of Denmark in Lyngby is generally related to static soil behaviour and mathematic modelling using statistical methods. It is generally state financed, however IGG has received private funding of more than 1.5 mill DKK for establishing automatic testing equipment for element testing including 3D-triaxial equipment in a newly developed educational and research laboratory.

Currently finalized and running Ph.D.-projects relate to "Geology and Geotechnics of Tertiary Clays" (Karen Furbo-Rasmussen), "Use of geotechnical methods in the mathematical simulation of natural fracture development in chalks" (Anette Olsen) and "Stochastic finite element methods on Soil" (Hans Christian Lybye). The funding is from TUD for Furbo-Rasmussen and Lybye and from the Danish Research Academy and The Geological Survey of Denmark for Anette Olsen. DGI is directly involved in the two first projects.

The research at Soil Mechanics Laboratory of the Danish Engineering Academy is mainly related to centrifuge testing of sand and clay and soil reinforcement by use of geotextiles. The Laboratory participate in an EEC-funded project on strength and deformation properties of sand together with English and French partners.

2.2 Foundation Engineering (deep and shallow foundations and soil reinforcement/improvement)

The relation to the before-mentioned group of R&D makes it difficult to distinguish specifically the present group. Some overlap is unavoidable.

At DGI most R&D in this group is related to consultative development projects placed in geotechnical category 3, where new methods of deep and shallow foundations subjected to complicated horizontal and vertical forces including static -, shock- and cyclic-dynamic loadings are developed. The work includes model testing, advanced computer analysis, construction and performance control.

Typical examples are related to ship impact on bridge piers and performance of anchor block placed on stone wedge with reference to the Storebælt connection. Steenfelt (1994, in press): "Shear transfer through stone wedge to clay interface". XII ISMFE, New Delhi.

DGI has also been part in a bilateral EEC-funded project with Greek lead partner on "Cyclic loading response of model anchor piles in clay". The results are also published in New Delhi by Steensen-Bach and coworkers.
A number of Storebælt-related development projects are published currently as shown in Encl 1. The costs amount to more than 10 mill DKK per year during the construction period covering advanced field and laboratory scale testing, analyses and control measurements. Consequently, these activities of project related studies contribute to the most costly R&D activities in Denmark at present within soil mechanics and foundation engineering.

At VD and DSB project related studies for the Storebælt as well as the Øresund connection have been major topics. No specific information is presently available being restricted because the projects are under construction or are considered for tendering. It will be very difficult also in future to separate project investigations and design from development and research activities. From a pure scientific viewpoint most activities can not be classified as research (being defined as "something nice to know") but very often very limited to topics of necessity for the projects ("need to know"). Both state directorates are performing major detailed control programme on existing dams and bridges. DGI has been involved in a number of internal studies related to f.inst. sheet pile walls, pile installation in limestone, frost susceptibility of limestone and fill properties, but no reports may be published at present.

Both geotechnical university laboratories (FL/AUC and IGG/TUD) have been active specialist consultants on soil properties, soil structure interaction and advanced calculation principles during the design and verification phase of the described bridge and tunnel connections. Some parallel testing and Ph.D. and M.Sc. research projects have been initiated based on this consultant work.

2.3 Environmental Engineering.

The environmental research at DGI is internally funded by more than 1.7 mill DKK. However, the research is considering basic measuring and calculation principles and not directly related to road and bridges.

DGI has developed methods for analyzing vibrations and is currently engaged in evaluations of vibrations from traffic and heavy duty construction methods as piling a.o. Dr. Hans Denver is a specialist in these matters.

DSB has also developed expertise in vibration analysis and has currently 3 partly industry funded Ph.D. research projects activated. A major future problem is connected to high velocity railroad connections in Denmark and towards Germany.

Environmental impact is considered a major topic for design and construction of the Øresund link. R&D activities are currently being activated in order to achieve a minimum disturbance to the population and the nature along the connection.

General concerns related to pollution from smoke, sulphur, CO₂ and NOₓ from power plants, industry and household have given background for an internationally recognized research and development in utilization of end-products as slag, flyash and desulphurization products. DGI has participated together with f.inst. the State Road Laboratory and alone in these activities and published a number of research reports and articles in this matter, Encl 3. A possible utilization as road base has been studied in laboratory and in field scale and evaluation techniques have been proposed for environmental impact as well as for geotechnical properties related to drainage, load carrying and deformation properties and frost susceptibility.

The Institute for Roads, Traffic and Planning of Towns - IVTB - at the Technical University of Denmark in Lyngby has also been very active in these studies.
Presently, Laboratory Report 66 "Subbase of incinerator Residue - Guidance - Standard Specifications - General Working Procedure" 1989 from the State Road Laboratory is considered for revision based on a combined research programme including DGI, SY, the Danish Water quality Institute and AFATEK A/S. The funding for the project is obtained from the Danish Environmental Agency and amount totally to 2 mill DKK for the next two years.

Danish incinerator residue can and have been used as subbase. Tender specifications for Danish conditions will be revised based on the research.

The utilization of end-products and limitation of environmental impact is of major concern for Danish research institutes and the State Environmental Agency. It will be a major subject for further research and development in future, but the funding is limited to some million DKK per year depending on the costs for establishing of deposits or utilization.

Slope stability is a major problem for motorways in Jylland through areas with outcrops of very highly plastic clays of Tertiary age. Plasticity index of up to Ip = 300% causes instability in areas with a natural slope of 5 to 10 degrees under water flow parallel to the surface and in cuttings in a hilly landscape. DGI has through many years of consultancy developed a database on Tertiary Clay Properties. Karen Furbo-Rasmussen has just finalized a Ph.D.-project on Geology and Geotechnical Properties for Tertiary clay at IGG/TUD in cooperation with DGI. A major problem is related to a possible bridge or tunnel connection between Denmark and Germany at Rødby-Femarn through an area with glacial floes and Prequaternary deposits of these Tertiary clays.

Ground water contamination is also of major concern to the environment. At DGI we have more than 20 technicians, geologists, biologists, chemists and civil engineers working with pollution from municipality waste, industry, petrol stations and traffic. This area attract many research institutes and consultants due to estimated environmental costs in the order of some 20 billion DKK for the next 10 years to fight pollution. It is however considered in Denmark not to be related very much to road and bridges, and a specific research activity is generally not attached to the previously mentioned institutes.

Possible Danish activities and Nordic Road and Transport Research may have been registered in the IRF or IRD data bases coordinated by the Danish Road Laboratory in Roskilde represented by Ms Lillian Olling.

3. Concluding remarks.

R&D activities specifically related to road and bridges in Denmark are very limited, but many more or less coordinated research and development activities is taken place in connection to other themes and directly related to the Storebælt and Øresund connections. The lack of funding is mainly counteracted by in-house and consulting activities related to projects in Geotechnical Category 3. The need for knowledge gives possibilities for further developments and the international cooperation related to EEC standardization is very important for the geotechnical society in Denmark.
Geotechnical papers prepared on the basis of the investigations performed for the fixed link across Storebælt.

A. Papers by authors/co-authors with DGI

AI: Published papers, which have been approved by A/S Storebæltssforbindelsen


A2: Papers based on general experience from the investigations, however without application of specific data (published without reference to A/S Storebæltsforbindelsen)


B. Papers written by other authors/co-authors

B1: Published papers, which have been approved by A/S Storebæltsforbindelsen


B2: Papers based on investigations performed at Storebælt, however without reference to A/S Storebæltsforbindelsen


List of dgf-Bulletins

The Bulletins may be ordered directly from the Danish Geotechnical Society, Maglebjergvej 1, DK-2800 Lyngby. Postage is not included in the quoted prices.

The Bulletins are published with the same layout, but are color-coded in six different series:

- red: English version
- blue: Soil mechanics
- green: Foundation engineering
- grey: Design practice
- yellow: Engineering geology
- brown: Environmental engineering

The red series contains contributions in English from the entire spectrum of geotechnical engineering, whereas the specialized series are mainly in Danish. The title is listed in the language of the paper. For Danish contributions, the English title is given in parentheses.

1988

   DKK 150.

   DKK 60.

1989

   Om geologisk kartegning (On geological mapping), pp. 53-71.
   DKK 120.

   DKK 120.

1990

   DKK 120.

1991

   DKK 120.
National Report

R&D activities

Finland
QUESTION 1

Geotechnical design, foundation engineering and environmental geotechnics

Research area

Pavements and foundations.

Name of research project, objectives, time schedule

TPPT-Road Structures Research Programme

The general aim of the TPPT programme is to improve the serviceability of both new and reconstructed roads so that the annual cost will be reduced and environmental impacts minimized. Road serviceability is described by the functional performance (evenness) and by the structural performance (fatigue, resistance against frost heave and geotechnical bearing capacity) viewpoints. Special emphasis is laid on the control of evenness due to uneven frost heave and settlement of subgrade.

The specific aims set for the programme are:

1. Improvement in the economy of road-keeping by 10% for new roads and 5% for reconstructed roads over the 1991 level, as measured according to the principle of annual costs.

2. Increasing road serviceability while halving improvement costs due to unpredictable damage (fatigue, frost heave, settlement and other such damage) by 1996.

3. Reduction in environmental impacts as a result of cutting back on the use of gravel materials and increasing the use of waste materials.

The research programme is planned over the period of 1993 - 2000. This Year the State-of-the-Art reviews have been made and detailed project plans are under reparation. Total cost for the project will be app. 10 million USD. Moreover in conjunction with this project private sector will finance their own product development projects.

Item for Possible oral presentation of special research area

R&D management: "Presentation of TPPT-programme"

"The role of manufacturers and contractors in the development of road geotechnics"
QUESTION 2

Use of geotechnical knowledge

Dissemination of R&D results
- forms of distribution of results

Practice at Finnish National Road Administration (FinnRA)
1. FinnRA has its own publication series.
2. Lectures, meetings and seminars are arranged for FinnRA personnel.

- practical use of new knowledge
The results are utilized in regulations, design guides etc. when their usefulness has been tested in practice.

Technology transfer
- knowledge transfer from university to industry
Continuos education courses on soil behaviour, models and calculation methods have been arranged in HUT (professors Korhonen, Wood, Leroueil)

Technology transfer center FinnT² has just started in co-operation with the Federal Highway Administration and FinnRA
More accurate and more reliable design methods for evenness, fatigue, frost heave and settlement of roads.

More precise control over the effects of traffic and the climate on the behaviour and deterioration of road structures taking into account the variability of the factors.

Variations in the quality of road structures will be controlled with the aid of new quality criteria related to the functional properties of the roads.

New production methods for road construction.

The application of new measurement techniques, providing more accurate and more reliable input data for pavement design and quality control.

New equipment and methods that can be used to estimate the service life of pavement structures in controlled conditions (e.g. a test facility).

A comprehensive system based on life-cycle cost analysis and knowledge of various fields, making it possible to exercise better control over pavement technology than at present.

Contact information

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Road, Traffic and Geotechnical Laboratory
P.O. Box 116, 02151 Espoo, Finland
Tel: Int. +358 0 4561 Fax: Int. +358 0 463251
TPPT for meeting future highway challenges

The Road Structures Research Programme (TPPT) is an eight-year (1993-2000) FIM 45 million research programme funded by the Finnish Road Administration (FinnRA). The Technical Research Centre of Finland (VTT) has responsibility for carrying out the research programme under the guidance of FinnRA. In addition, universities of technology, consultants, contractors and industry will all participate in the research work and in the implementation of its results.

TPPT focuses on improving the serviceability of roads and the economy of road-keeping and reducing environmental impacts. A need exists for new pavement technology in Finland for following reasons:

- Current pavements are massive structures, which have resulted in the large-scale use of environmentally valuable landscape features (local gravel and rock areas).
- High quality materials are used in modern pavement structures, also in unbound layers. In the future good quality unbound materials will no longer be available as economically as before.
- The rapid developments that have taken place in measurement techniques have only been partially reflected in site investigation methods.
- As far as road subgrades are concerned, the potential benefits offered by new stabilization techniques have not been sufficiently exploited.
- The present approach to the structural design of roads is largely based on traditional structures with thick unbound layers of good-quality material. Bound structures incorporating new technology (e.g. composite structures) have not been widely investigated and applied in practice so far.
- Uncontrolled variations occurring in road structures (e.g. layer thickness) due to current production techniques and quality control methods are too often the cause of premature damage and decreased serviceability.

Objectives

The general aim of the TPPT programme is to improve the serviceability of both new and reconstructed roads so that annual costs will be reduced and environmental impacts minimized. Road serviceability is described by the functional performance (evenness) and by the structural performance (fatigue, resistance against frost heave and geotechnical bearing capacity) viewpoints. Special emphasis is laid on the control of evenness due to uneven frost heave and settlement of subgrade.

The specific aims set for the programme are:

1. Improvement in the economy of road-keeping by 10% for new roads and 5% for reconstructed roads over the 1991 level, as measured according to the principle of annual costs.
2. Increasing road serviceability while halving improvement costs due to unpredictable damage (fatigue, frost heave, settlement and other such damage) by 1996.
3. Reduction in environmental impacts as a result of cutting back on the use of gravel materials and increasing the use of waste materials.
Research approach

The aims of the programme will be achieved by developing new pavement structures, pavement design guidelines, production techniques, quality control systems and environmental protection guidelines. This will allow the performance of the pavement to be controlled in different loading and climatic conditions in the expected way, without producing unreasonable impacts on the environment.

In this comprehensive research programme, the road is examined as a single entity from the foundation to the surfacing. The entire structural system can be made to perform with sufficient and desired accuracy, when the individual and combined effects on the different factors to the entity as a whole are known. The framework of the research approach is illustrated in Figure 1.

Pavement performance will be determined under different loading conditions in controlled experiments provided by laboratory tests, full-scale pavement tests and test structures. The actual performance of road structures will be checked by means of field tests. The latest developments in advanced measuring techniques will be utilized in the research work.

The research programme includes the following research areas:

1. Input information
   - traffic, climate, materials, foundation
2. Pavement and subgrade structures
   - new road structure types and their performance
3. Materials technology
   - unbound and bound materials, special materials, material models related to the structures and climatic impacts
4. Pavement design
   - fatigue, frost, settlement and their interaction, reliability
5. Production technology
   - construction methods, equipment, QA/QC, safety
6. Measurement techniques
   - non-destructive and destructive methods
7. Research techniques
   - experimental design, theory, laboratory tests, test structures (laboratory, field), test roads
8. Target criteria and pavement performance
   - performance and distress-modelling
   - condition, economy and environmental criteria
   - life-cycle cost analysis

Figure 1. Framework of the 'TRIP' research programme
Timetable and finance

The planned timetable for the programme is presented in Figure 2. State-of-the-art reviews and the preparation of detailed project plans began on 13.9.1993 and the programme is scheduled to end in 2000. The programme is funded by the Finnish National Road Administration (FinRat), the estimated total cost for the period 1993 - 2000 being in the region of FIM 45 million. This amount will be divided up so that the costs of the state-of-the-art reviews initiated in 1993 will amount to FIM 2.5 million, and FIM 6 million per annum will be allocated for other tasks. This does not include the costs involved in the construction of test roads.

The other organizations participating in the programme (e.g. municipalities, the National Board of Aviation) will make their own financial contributions as and when necessary. In addition, the private sector has its own research and development work related to the IPPT research area.

Expected results and benefits

The research programme is expected to produce new:

1. Pavement and foundation structures
2. Pavement and geotechnical design guidelines
3. Quality control systems
4. Working methods
5. Environmental protection guidelines

The results will encompass the following matters:

- Longer lasting and more economic pavement and foundation structures
- Improved control over road building materials, including poor quality materials, soft soils, geosynthetic products and industrial by-products.
- Control of the functional properties of materials in different conditions, as well as numerical models for predicting the behaviour of materials.

<table>
<thead>
<tr>
<th>Year</th>
<th>1993...</th>
<th>...1994-95...</th>
<th>...1996-97...</th>
<th>...1998-99...</th>
<th>2000</th>
</tr>
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<tbody>
<tr>
<td>Pavement structures and subgrade</td>
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<td>Design technique</td>
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<tr>
<td>Production</td>
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<tr>
<td>Budget (FIM million)</td>
<td>2.5</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>6</td>
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</tbody>
</table>

Figure 2. The timetable and budget for the research programme.
QUESTION 1:

Geotechnical design

ONGOING R & D-PROJECTS
- research areas
  Soil reinforcement
  Soil models and calculation methods, including frost action
  Bearing capacity of large piles
  Soil stabilization, incl. peat

- name of research projects, objectives, time schedules
  Geotechnical design of piles for bridges without bearings. TUT, Diss. M.Koskinen
  Non-dynamic analysis methods for bearing capacity of large piles. TUT Lic., J.Heinonen
  Development of testing methodology for dynamic soil properties. UO, O.Ravaska.
  Geophysical methods in road design. VTT, O.Okko
  Deformation of coarse granular material for road basement. TUT, Diss., P. Kolisoja
  Deformation of road embankment on soft subsoil. TUT, Lic., T. Länsivaara
  Properties of lightweight aggregate. HUT, E.Slunga 11/93
  Properties of stabilized peat. UO, K.Kujala
  Long term bearing capacity of timber piles in embankment piling. VTT, M.Juvankoski 12/93
  Frost susceptibility of stabilized clay. HTU, E.Slunga, 3/94
  Numerical modelling of frost heave phenomena. UO, K.Kujala
  VTT, H.Kivikoski, 1994
  Modulus calculation from falling weight tests. HTU, E.Slunga 12/93
  New geotextile specification .VTT, H.Rathmayer, 11/1993

- preliminary research results

- joint projects or EC-projects
  Thaw weakening and springtime bearing capacity. HUT, E.Slunga 12/93, UO, K.Kujala &
  VTT, S.Saarela (ISSMFE- TC8 "Frost in geotechnical engineering")

- item for a possible oral presentation of a certain research area
  Problems encountered with quality control of deep soil stabilisation.

PLANNED R & D-ACTIVITIES
- problems in infrastructure related to soil, rock and ground water
- planned R & D-projects in a 1 - 3 years period
  The TPPT programme, initiated by the Public Road Administration, is at its starting point
- co-operation with other organisations
  National cooperation with VTT, TUT (Tampere TU), UO (Oulu Uni.), HUT (Helsinki TU),
  the geotechnical departments of major cities, private consultants, the National Board of

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Telex 122972 vrtha sf
Waters and the Environment, the Public Road Administration and Industry in the Construction Materials Sector.

**Foundation engineering**

**ONGOING R & D-PROJECTS**
- **research areas**
  - Steel piles,
  - Bearing capacity of piles
- **name of research projects, objectives, time schedules**
  - Foundation of scaffolding. TUT, dipl.wk., H.Kulmala
- **preliminary research results**: NO
- **joint projects or EC-projects**: NO
- **item for a possible oral presentation of a certain research area**: NO

**PLANNED R & D-ACTIVITIES**
- **problems in infrastructure related to soil, rock and ground water**
- **planned R & D-projects in a 1 - 3 years period**: Corrosion of steel piles (possibly)
- **co-operation with other organisations**

**REMARKS**: The whole sector is deeply suffering from the ongoing recession.

**Environmental geotechnics.**

**ONGOING R & D-PROJECTS**
- **research areas**
  - Utilization of byproducts in road constructions
  - Structure of noise barriers
  - Groundwater protection
- **name of research projects, objectives, time schedules**
  - Utilization of bottom slag for stabilization. UO, K.Kujala
  - Structural design of noise barriers. HKI/Geo, U.Anttikoski
  - Traffic vibrations (from trains) and the environment, VTT, J.Törnqvist
- **preliminary research results**: On groundwater protection at road slopes.
- **joint projects or EC-projects**: NO
  - Numerical modelling of groundwater contamination through salt from highway de-icing. VTT, A.Niemi, 12/94 & National Board of Waters and Environment.
- **item for a possible oral presentation of a certain research area**
  - Groundwater protection at road slopes.

**PLANNED R & D-ACTIVITIES**
- **problems in infrastructure related to soil, rock and ground water**
- **planned R & D-projects in a 1 - 3 years period**: The TPPT programme, initiated by the Public Road Administration, is at its starting point.
- **co-operation with other organisations**
QUESTION 2:

Use of geotechnical knowledge

Dissemination of R & D-results
- forms of distribution of results
  Practice at VTT:
  1. The Technical Research Centre has its own publication series
  2. Contract research for the Public Road Administration is published in the series of the Public Road Administration
  3. Conference proceedings, international journals, etc. are utilized
  4. Lectures, meetings, exhibitions, etc. are organized if spraying of information is searched.
  Practice at Universities:
  Diploma works are published in small edition for limited distribution.
  Major research works are published like at VTT:
  Public Road Administration and Geotechnical Office of the City of Helsinki:
  Own publication series.
  Finnish Society of Civil Engineers:
  Publication of handbooks and codes of practice.
  Finnish Geotechnical Society:
  Publication of handbooks and codes of practice.

- practical use of new knowledge
  The results of research projects are utilized by the Public Road Administration in their regulations, design guides etc.

Technology transfer
- knowledge transfer from university to industry and vice versa
  Research reports initiated by the industry (contract research) are not necessarily available for public use. Even information on ongoing research projects is kept secret.
- experiences from theory to practice and vice versa.

Activities in the EC R & D-programmes
- existing projects: no
- planned activities: no purely geotechnical project proposal issued
- transnational dissemination of knowledge, scientific exchange
  No activities due to lack of funds available
National Report

R&D activities

France
Introduction

Geotechnical research for roads and bridges in France has traditionally been mainly performed by the Laboratoire central des Ponts et Chaussées and its network of 17 regional laboratories. During the last ten years, two additional research frameworks have been established:

- the so-called "National projects" (Projets nationaux) were established by the Division of Economic and International Affairs of the French Ministry of Infrastructures and Transportation, in co-operation with the Federation of Public Works contractors. These National Projects are aimed at solving a given practical problem, under close co-operation of private construction firms and research centres (including LCPC and CEBTP in most cases). They are typically organised for a 4 to 5 years period. National projects related to geotechnical engineering are:
  * Clouterre (1985-1990) for soil slope nailing,
  * Tunnel 85-90 (1985-1990) for shield tunnelling,
  * Itelos (1987-1992) for site monitoring,
  * Materloc (1987-1992) for increasing the use of subnormal road base materials and aggregates.
  * Clouterre II (1992-1995) for soil nailing,
  * Forever (1993-1998) for root piles (injected micro-piles),
  * Microtunnels (under study) for micro-tunnelling,
  * Eupalinos (under study) for tunnelling in heterogeneous ground conditions;

- and the "GRECO Géomatériaux", which was established by the French CNRS (Centre National de la Recherche Scientifique); GRECO is standing for "Groupe de REcherche en COopération". It can be described as a sort of research association between public laboratories, mainly from Universities and Public research organisations. The GRECO Géomatériaux has been established for a 4 years period in 1985 and was renewed for a second 4-year period in 1990. It has developed research co-operations with some Universities of EC-countries and a European network "ALERT" is now under preparation with European funding. Research works under the supervision of GRECO Géomatériaux were mainly devoted to the following subjects:
  - Static and dynamic laboratory testing of cohesive and granular soils
  - Numerical modelling
  - Discontinuities, shear bands and fissures
  - Behaviour of unsaturated soils.
Before coming back to the Ponts et Chaussées research activities, it might be of interest to provide some general information about French research organisations concerned by geotechnical research. Public research in France has been funded during the last fifteen years:

- by the "Budget for Civil Research" (Budget civil de la recherche), which is used to fund research in agencies depending on the Ministry of Research and on other Ministries (for example, to LCPC and its network of regional laboratories through the Ministry of Infrastructures and Transportation) as well as research projects;
- and by the own budget of "technical Ministries", such as the Ministry of Environment and the Ministry of Infrastructures and Transportation (Road Division) in the case of road and bridges research.

The funding and co-ordination system relative to geotechnical research, including road and bridge R & D, can be described as follows:

<table>
<thead>
<tr>
<th>Ministry of Public Works (Infrastructures and transportation)</th>
<th>Ministry of Universities and Research (at some periods it has been split into two separate ministries)</th>
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<tbody>
<tr>
<td>- CORGEC (Advisory Committee)</td>
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<tr>
<td>- LCPC and Regional Laboratories</td>
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<td>- Research programmes</td>
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<td>- URA</td>
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<td>- UMR</td>
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</table>

CORGEC : Conseil d'orientation pour la recherche en génie civil  
DRAST : Direction de la Recherche et des Affaires Scientifiques et Techniques  
CNRS : Centre National de la Recherche Scientifique  
GRECO : Groupement de recherche en coopération  
UR : Unité de recherche (propre au CNRS)  
URA : Unité de recherche associée  
UMR : Unité mixte de recherche (commune avec une autre organisation)

**Detailed research programmes**

*a. Laboratoires des Ponts et Chaussées*

*a1. Internal research programmes*

Research programmes of the LPC network are currently divided into pluriannual programmes and annual programmes. Pluriannual programmes, which are called "Research themes" have an annual funding of about 2 to 4 thousands French francs, including labour costs. There are 7 such "Thèmes de recherche" devoted to:

- GEO 24 : Reinforcement of rock masses by means of passive anchors  
  - Mechanical properties and behaviour of discontinuous rock masses
- Laboratory testing of rocks and reinforcing elements
- Failure mechanisms of discontinuous rock masses
- Rock slope stability

- GEO 25: Deformation analysis of unstable slopes
  - Site monitoring
  - Rainfalls and slope movements
  - Rheology of slip surfaces
  - Numerical modelling of slope deformations
  - Slope nailing

- GEO 29: Durability of geotextiles and behaviour of reinforced structures
  - Site investigations
  - Laboratory studies
  - Full-scale and centrifuge models
  - Design methods

- GEO 30: Widening of road embankments
  - Geotechnical investigations of existing embankments
  - Stability analysis of embankments and reinforcing structures
  - Widening of road embankments on soft soils

- GEO 31: Site effects and behaviour of structures under seismic loading
  - Seismic loads and site effects
  - Effects of seismic loading on bridges
  - Behaviour of buildings under seismic loading

- GEO 32: Design of road structures and behaviour of unbound materials
  - Full scale observations on the behaviour of unbound gravel
  - Modelling of the elastic behaviour of soils and unbound materials
  - Modelling of the anelastic behaviour of soils and unbound materials
  - Modelling of the behaviour of soft road base layers and pavements.

Besides, 4 technical committees are in charge of programming research on an annual basis (Research programmes with a bold title are those with funding in excess of 1000 to 2000 French francs):

- CT 21 on "Quarries and aggregates"
  * Methods of testing aggregates
  * European standardisation, including comparative tests at the European level
  - Laboratory and field behaviour of aggregates
  - Quality control of aggregates
  - Production of aggregates

- CT 22 on "Road geotechnical engineering"
  - Recommendations and codes of practice
  - Quality of earth works
  - Sounding and control of earth works
  - Execution techniques for earth works
- **CT 23 on "Geology and site investigation methods"**
  - Geotechnical regional summaries
  - Teledetection
  - Prospection methods (including the radar)
  - Hydrogeology
  - Underground quarries
  - Rock breaking or blasting
  - Soil and rock identification
  - Site investigations for tunnels

- **CT 24 on "Soil and rock mechanics and foundation engineering"**
  - Finite element analysis (CESAR-LCPC)
  - Penetrometer and pressuremeter testing
  - Testing and monitoring equipment
  - Knowledge-based systems
  - Computer-aided design
  - Centrifuge modelling
  - Limit state analysis and design
  - Embankments on soft soils
  - Soil improvement
  - Geomembranes (material properties and uses)
    - Texsol
    - Pneusol (soil reinforced with old tyres)
    - Lightweight fills
    - Foundations on rock
    - Probabilistic analyses
    - Modelling of expansive clays
    - Retaining structures
    - Tunnels
  - Foundations (groups of root-piles, new design methods)
  - Laboratory testing applied to foundation design
  - Soil sampling
  - Soil rheology (sands, marls, clays)
  - Unsaturated soils

a2. Co-operative research (European and other international research projects)

**BRITE**
- Automation of aggregate plants

**SCIENCES - Human Resources and Mobility**
- Centrifuge modelling

**SPRINT**
- Quality control for geotechnical testing

**EUREKA**
- PREMEC : Use of the mechanical pre-cutting method for tunnelling in hard and soft water bearing rocks
France-Portugal
- Behaviour of geotextile-reinforced structures

France-Germany
- Behaviour of a geotextile-reinforced retaining wall
- Tunnelling

France-Greece
- Foundation design and slope stability

b. National Projects

CLOUTERRE
- Full scale and laboratory experiments on soil nailing
- Site and soil investigation methods
- Design methods for nailed structures
- Durability

CLOUTERRE II
- Follow-up of the practical uses of Clouterre Recommendations
- Pull-out tests on nails
- Monitoring of the displacements of nailed walls
- Study of the behaviour of the facing
- Freeze-thaw behaviour of nailed structures
- Behaviour of nailed structures under static and dynamic loading

FOREVER
- Behaviour of a single micro-pile
- Behaviour of groups of micro-piles
- Behaviour of networks of micro-piles
- Dissemination of results and recommendations

Answers to the questionnaire

Most questions related to Plenary sessions 1-3 have been answered in the preceding section. Detailed information about the research programmes of the Laboratoires des Ponts et Chaussées can be found in different documents published annually by LCPC (in particular the Annual Activity Report, the annual list of publications and the annual description of the research programmes, including the name of the persons in charge of co-ordinating each research, the participating technical units and a brief description of the research to be performed within a year period. All these documents are in French). If you are interested, I could give a more detailed presentation of some of the main research activities.

Future research is usually related to existing activities, which are extended for some particular reason. The main sectors of activity which will be particularly developed during the next three-years period are:

- tunnel design,
- use of radar for geotechnical investigations,
- stability of abandoned underground quarries.
Other activities which will be extended, though on a still undefined basis, are: environmental geotechnics and foundations on rock masses. As far as possible, new activities are organised on a co-operative basis, either with French research organisations or enterprises or with foreign research centres.

Many persons participating in the LCPC research programmes are only part-time researchers. They are mainly involved in geotechnical site investigations or in the design of geotechnical structures. Such an interrelation between research and practice is considered to be particularly fruitful. It is one of the major keys to the dissemination of R&D results in our country.

Use of geotechnical knowledge

The main forms of dissemination of R&D results from the LPC network are:
- the publication of research reports, guides and recommendations
- the participation in the teaching of geotechnical engineering, both at elementary engineer level and at higher levels (doctoral studies),
- the preparation of French norms,
- the publication of journal papers and conference papers (mostly in French),
- the organisation of technical seminars,
- the participation in continuing education courses (mainly at the Ecole Nationale des Ponts et Chaussées, Paris)
- the participation in common R&D programmes with private firms.

The number of people working on geotechnical problems in the LPC network is about 600 (engineers, technicians, workers). Out of them, the number of full-time researchers is of the order of 80 to 100.

Jean-Pierre MAGNAN
October 24, 1993
France

References (not included):

Monographies d'études et de recherches
Réseau des Laboratoires des Ponts et Chaussées
Répertoire des publications 1992
Repertoire Central des Ponts et Chaussées
Programme des études et recherches du réseau des laboratoires des Ponts et Chaussées 1992
Rapport général d'activité 1992
Laboratoire Central des Ponts et Chaussées
National Report

R&D activities

Italy
In spite of the fact that our country is suffering from a severe economical recession and a deep political crisis, we still hope that some major projects may start in the next future. The following short notes might be of some help in the preparation of the seminar under questions.

**MOST IMPORTANT ON-GOING PROJECTS:**


2. One-span, 3300 m long, suspended bridge over the Messina Straits(*). Start of construction: unknown. Political will to build the bridge is uncertain.

3. Widening and improvement of the highway between Torino and Savona, length ≈ 160 km: probable start of construction autumn 1994.

4. "High speed" railway system Torino-Venice and Milano-Naples total length ≈ 900 km. This project includes many important bridges and very deep (200+500 m) and long (6+14 km) tunnels in hard clays and soft rocks. Probable start of construction: end of 1993 for the section from Naples to Rome (≈ 220 km). Construction of the line from Milano to Venice is more uncertain than the one of Milano-Naples and Torino-Milano sections.

(*) projects in which the writer is involved.
IMPORTANT TECHNOLOGICAL INNOVATIONS:

Italian specialized foundation construction firms (Rodio, Trevi, etc) are strongly engaged in new technologies related to underground works in soils and soft rocks. Of major interest are the applications of the R.P.U.M. (Reinforced Protective Umbrella Method) pretunnel and precutting techniques. All these techniques are aimed at forming either temporary and/or final linings before excavating a tunnel or an underground space.

Almost all these new procedures have been applied in the last few years in many important projects with good results, even if their design criteria are still very crude and empirical. This problem will represent in future an attractive area of the geotechnical research.

I look forward to hearing from you and to seeing you in Sweden next November.
Italy

References:

Validity of in situ tests related to real behaviour
M Jamiolkowski, D C F Lo Presti

Stiffness of Toyoura sand at small an intermediate strain
M Jamiolkowski, R Lancellotta, D C F Lo Presti, O Pallara

Flat dilatometer tests in Toyoura sand
R Bellotti, C Fretti, M Jamiolkowski, F Tanizawa

Stiffness of carbonatic Quiou sand
M Jamiolkowski, D C F Lo Presti

GEO-COAST '91 - Design parameters from theory to practice (theme lecture) 1)
M Jamiolkowski, S Leroueil, D C F Lo Presti

Predictive soil mechanics 1)
R Crova, M Jamiolkowski, R Lancellotta, D C F Lo Presti

1) Abstract and conclusions included. A copy of the paper can be ordered from the Swedish Geotechnical Institute, S-581 93 Linköping.
It happens quite frequently that discussions arise about the relative merits of in situ versus laboratory tests. The writers believe that this debate is generally of little practical value, because laboratory and in-situ soil testing are complementary rather than competing methodologies.

Another important point is that progress that is achieved within the two areas of soil testing renders the comparison in-situ and laboratory testing mutable with time.

At present, considering the remarkable progress made in the eighties in the area of laboratory testing and undisturbed sampling, especially in cohesive soils, the most relevant applications of in situ tests are the following:

a. soil profiling and characterization;

b. evaluation of in situ horizontal stress;

c. assessment of hydraulic conductivity;

d. evaluation of the stiffness of cohesionless soils;

e. calculation of settlements and bearing capacity of foundations directly from in situ test results, especially for cohesionless soils.

This list of priority applications appears clearly to be finalized at:

- testing of soil volumes which are appreciably larger than those usually involved in conventional laboratory tests, e.g., a) and c);
- assessment of stress-strain properties of soils in which undisturbed sampling is still very difficult, e.g., d) and e);
- assessment of the spatial variability of soil deposits, e.g., a);
- employment of non-destructive techniques, such as geophysical methods, in order to evaluate the soil state variables, e.g., b), c) and d), with special reference to the assessment of soil stiffness at very small strain or the attempts to evaluate the void ratio of cohesionless deposits in situ;
- empirical design of foundations, correlating the in-situ test results directly to the behavior of the relevant prototype, e.g., c).

Due to the constraint in space the writers will only briefly deal with the evaluation of in situ horizontal stress and stiffness of sands via in situ tests.

The assessment of the design stiffness represented by an "operational" value of the secant Young's modulus $E_s$, appropriate for settlement calculations using the formula of the theory of elasticity can be attempted following the procedure outlined below:

\[
E_s = \frac{SD}{s} \cdot I_s \cdot \left( \frac{E}{D} \right) \cdot (1-v^2)
\]

Panel discussion - Plenary Session A - Soil Properties
The modulus number \( K[E_s] \) has been computed as follows:

\[
K[E_s] = \frac{E_s}{\frac{\sigma_{n1} + \Delta \sigma_{n}}{P_a}}^{0.5}
\]

being:

- \( \sigma_{n1} \) = vertical consolidation stress applied to the CC specimen
- \( \Delta \sigma_{n} \) = increase of vertical stress at the depth D/2 under the centerline of the plate
- \( P_a \) = reference stress = 98.1 kPa

while that related to the initial stiffness was obtained as follows:

\[
K[E_i] = \frac{E_i}{\frac{\sigma_{n1}}{P_a}}^{0.44}
\]

where:

- \( \sigma_{n1} \) = mean consolidation stress acting on the CC specimen prior to the start of the PLT.

Fig. 1. Plate loading tests in Calibration Chamber.

While the writers are fully aware that for the coherence reasons also \( E_s \) should be normalized with respect to the value of the current \( E_s \), the current value of \( \sigma_{n1} \) has been chose for sake of practicality. (Simplicity?). Figures 2 to 4 show the ratio of the secant modulus number \( K[E_s] \) to the initial modulus number \( K[E_i] \) as function of s/D. It appears that at least up to s/D = 10% a double logarithmic plot leads to a straight line which describes adequately the decay of the average operational stiffness \( E_s \) with increasing s/D. These figures suggest that once \( \sigma_{n1} \) (or \( E_s \)) in situ is assessed via seismic tests, a value of \( E_s \) at the desired stress level for the relevant value of s/D can be computed.

This very pronounced degradation of stiffness as evidenced in Figs. 2 through 4 holds for pluvially deposited Ticino sand. As suggested by Ishihara (1993), the degradation of soil stiffness is natural granular deposits might result even more pronounced than that observed in freshly deposited cohesionless soils.

Finally, the numerical analysis of PLT's performed by Boccio (1993), reported by Ghionna et al. (1993), indicate that within the range of s/D ≤ 10% the experimental results are not influenced by the limited dimensions of the CC specimen (height = 1500 mm, diameter = 1200 mm). Within this range of the s/D the experimental results can be fitted by the following formulae, valid for \( K[E_i]/K[E] \leq 1 \):

- NC. CC specimens: \( K[E_i] = 0.045 \) \( K[E] \) (s/D 100)\(^{-0.5} \)
- OC. CC specimens: \( K[E_i] = 0.061 \) \( K[E] \) (s/D 100)\(^{-0.5} \)

As to the determination of the in situ horizontal stress in cohesionless soils two basic approaches are presently available, Robertson (1986):

a. The direct methods, exemplified by the Self-Boring Pressuremeter tests, (SBPT) for which it is assumed that the insertion of the probe does not cause any appreciable disturbance in the surrounding soil so that the lift-off pressure \( \rho_o \) of the pressuremeter membrane corresponds to the initial in situ total horizontal stress \( \sigma_{n1} \) (Fahey and Randolph (1984), Bruzzi et al (1986)). However, the experience gained in late eighties (Fahey and Randolph (1984), Jamiolkowski et al (1985), Bruzzi et al (1986), Bellotti et al (1993)) have clearly shown that even small amounts of disturbance and mechanical compliance of the sensors measuring cavity strain can render the assessed values of \( \sigma_{n1} \) not fully reliable. This is especially true in case of the cohesionless soils.

b. The indirect methods are based on the empirical correlations between the large strain parameters measured using different penetration devices and \( \sigma_{n1} \). Among them, the Marchetti's Flat Dilatometer test (DMT), Marchetti (1980) and the Lateral Stress Cone Penetration test (LSCPT), (Huntsman (1985), Jefferies et al (1987), Sisson (1990), Campanella et al (1990)) allow to measure directly the total horizontal stress \( \sigma_{n1} \) acting on the device after penetration. If the penetration pore pressure \( u \) is also measured during the penetration the horizontal effective stress can be assessed.

The evaluation of the initial in situ effective horizontal stress \( \sigma_{in} \) is then referred to the amplification factor:

\[
AF = \frac{\sigma_{in}}{\sigma_{ho}}
\]

In case of the DMT the AF is represented by the ratio of Marchetti's (1980) horizontal stress index \( K_{ho} \) to the reference value of the in situ coefficient of the earth pressure at rest \( K_{ho} \).

The AF is obtained via calibration of said devices in the calibration chambers, e.g. Sisson (1990) or on sites where the reference values of \( \sigma_{n1} \) or \( K_{ho} \) have been assessed based on geological information or on SBPT's, e.g. Jamiolkowski et al. (1985), Bruzzi et al. (1986) and Bellotti et al. (1989). Accumulated experimental evidences show that the AF increases with increasing the density of the cohesionless deposit or more precisely it increases with decreasing its state parameter \( v \) (Robertson (1986), Been and Jefferies (1985), Been et al. (1986)).

On overall, the reliability of the methods to assess \( \sigma_{n1} \) or \( K_{ho} \) from penetration tests results still appears to be very uncertain. This is mainly due to the subjectivity of the reference values against which the results of LSCPT and DMT have been calibrated. In addition, the AF values are substantially higher than one especially in dense sand and largely depend on the volume change characteristics during shearing of a given sand at the given state.
Table 1. Summary of deep plate loading tests performed in Calibration Chamber tests in dry Ticino sand.

<table>
<thead>
<tr>
<th>TEST N.</th>
<th>OCR</th>
<th>a' (X)</th>
<th>τ'ult</th>
<th>K(E)</th>
<th>a'ut</th>
<th>q'ut</th>
<th>σ'ult</th>
<th>D (µ)</th>
<th>OCR (µ)</th>
<th>q'ut</th>
<th>D (µ)</th>
<th>OCR (µ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>307</td>
<td>1</td>
<td>93</td>
<td>1</td>
<td>314</td>
<td>122</td>
<td>165</td>
<td>102</td>
<td>9.57</td>
<td>47.8</td>
<td>3.95</td>
<td></td>
<td></td>
</tr>
<tr>
<td>308</td>
<td>1</td>
<td>93</td>
<td>1</td>
<td>216</td>
<td>86</td>
<td>143</td>
<td>98</td>
<td>10.61</td>
<td>42.8</td>
<td>3.45</td>
<td></td>
<td></td>
</tr>
<tr>
<td>317</td>
<td>1</td>
<td>54</td>
<td>1</td>
<td>62</td>
<td>24</td>
<td>63</td>
<td>36</td>
<td>12.28</td>
<td>10.8</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>318</td>
<td>1</td>
<td>58</td>
<td>1</td>
<td>216</td>
<td>94</td>
<td>115</td>
<td>59</td>
<td>18.36</td>
<td>21.5</td>
<td>1.78</td>
<td></td>
<td></td>
</tr>
<tr>
<td>319</td>
<td>1</td>
<td>57</td>
<td>6.3</td>
<td>65</td>
<td>55</td>
<td>79</td>
<td>95</td>
<td>29.14</td>
<td>15.0</td>
<td>1.42</td>
<td></td>
<td></td>
</tr>
<tr>
<td>320</td>
<td>1</td>
<td>58</td>
<td>6.3</td>
<td>63</td>
<td>51</td>
<td>78</td>
<td>93</td>
<td>31.65</td>
<td>15.8</td>
<td>1.80</td>
<td></td>
<td></td>
</tr>
<tr>
<td>321</td>
<td>1</td>
<td>58</td>
<td>6.3</td>
<td>121</td>
<td>78</td>
<td>104</td>
<td>101</td>
<td>34.74</td>
<td>29.0</td>
<td>2.71</td>
<td></td>
<td></td>
</tr>
<tr>
<td>322</td>
<td>1</td>
<td>92</td>
<td>6.3</td>
<td>67</td>
<td>66</td>
<td>104</td>
<td>106</td>
<td>57.45</td>
<td>36.9</td>
<td>3.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>323</td>
<td>1</td>
<td>91</td>
<td>6.3</td>
<td>28</td>
<td>84</td>
<td>51.2</td>
<td>50.14</td>
<td>19.4</td>
<td>1.90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>324</td>
<td>1</td>
<td>58</td>
<td>6.3</td>
<td>26</td>
<td>51</td>
<td>50</td>
<td>56</td>
<td>46.74</td>
<td>29.4</td>
<td>1.89</td>
<td></td>
<td></td>
</tr>
<tr>
<td>325</td>
<td>1</td>
<td>58</td>
<td>6.3</td>
<td>412</td>
<td>178</td>
<td>154</td>
<td>101</td>
<td>34.74</td>
<td>29.0</td>
<td>2.71</td>
<td></td>
<td></td>
</tr>
<tr>
<td>326</td>
<td>1</td>
<td>91</td>
<td>6.3</td>
<td>65</td>
<td>51</td>
<td>78</td>
<td>93</td>
<td>31.65</td>
<td>15.8</td>
<td>1.80</td>
<td></td>
<td></td>
</tr>
<tr>
<td>327</td>
<td>1</td>
<td>91</td>
<td>6.3</td>
<td>26</td>
<td>51</td>
<td>50</td>
<td>56</td>
<td>46.74</td>
<td>29.4</td>
<td>1.89</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Based on the above it clearly appears that at present the state of the art of evaluation of $a'$ or $K_p$ in cohesionless soils from in situ tests, is far from being satisfactory. Fig. 5 shows an example of the evaluation of $K_p$ in the geotechnically well investigated, slightly overconsolidated, Po river sand based on the results of the DMT's and SBPT's, for further details see Jamiolkowski et al. (1985), Jamiolkowski and Robertson (1988) and Bellezza (1992). The same figure reports also the $K_p$ inferred from three LSCPT's at the same location normalized with respect to the effective overburden stress $a'_o$. The measured values of $a'_o$ lead to the AF = 1.48 ± 0.25 if the reference $a'_o$ is inferred from the SBPT's whose results are taken as the best estimate of the existing in situ horizontal stress. The above results clearly indicate the many existing uncertainties when attempting to evaluate $a'_o$ and $K_p$ in sands.

Fig. 2. Modulus number of medium dense NC dry Ticino Sand from deep plate loading tests performed in Calibration Chamber.

Fig. 3. Modulus number of very dense NC dry Ticino Sand from deep plate loading tests performed in Calibraton Chamber.

Fig. 4. Modulus number of medium dense NC Ticino Sand from deep plate loading tests performed in Calibration Chamber.

Fig. 5. Coefficient of earth pressure at resp of Po River sand from SBPT and DMT tests.

The possibility to improve our ability in assessing $a'_o$ and $K_p$ in cohesionless deposits is, at least in principle, offered by geophysical tests as firstly suggested by Stakoe (1985).

In fact, if one is able to generate and to measure the velocity of the vertically ($V_{vel}$) and the horizontally ($V_{h}$) polarized shear waves, the
assessment of \( \sigma'_{\text{hi}} \) and \( K_0 \) can be attempted taking into account the following relationships [Roesler (1979), Stokoe et al. (1985, 1991)] referred to the effective in situ stresses:

\[
V_{\text{shi}}^2 = C_{\text{shi}} (\sigma'_{\text{hi}})^{n_h} \cdot (\sigma'_{\text{oi}})^{n_o}; \quad V_{\text{sh}}^2 = C_{\text{si}} (\sigma'_{\text{oi}})^{n_v} \cdot (\sigma'_{\text{oi}})^{n_h}
\]

being:

\( C_{\text{sh}} \) and \( C_{\text{si}} \) = material constants
\( n_h \) and \( n_v \) = material exponents
\( \sigma'_{\text{oi}} \) = effective overburden stress

assuming typically for sands [Stakoe et al. (1985), Lo Presti and O'Neill (1991)]:

\[
\frac{C_{\text{sh}}}{C_{\text{si}}} = 1.1; \quad n_h = n_v = 0.125
\]

it is possible to derive the following relationship:

\[
K_0 = 0.47 \left( \frac{V_{\text{sh}}}{V_{\text{hi}}} \right)^{10}
\]

The main problem arising when using in practice this approach is linked to the high exponent to which the ratio of shear wave velocities is raised that requires the utmost accuracy when measuring \( V_{\text{sh}} \) and \( V_{\text{hi}} \) in order to obtain reliable values of \( K_0 \). A similar approach can be inferred from the recent work by Hryciw and Thomason (1993).

An example of application of this approach at a site in the central part of Italy where both \( V_{\text{shi}} \) and \( V_{\text{hi}} \) have been measured using cross-hole technique is reported in Fig. 6 and 7. Unfortunately, the complex soil profile and lack of reference values of \( \sigma'_{\text{oi}} \) and \( K_0 \) makes the assessment of the reliability of the approach quite difficult. A severe scatter of the \( K_0 \) observed in Fig. 7 is probably associated to the inaccuracy of the measured shear wave velocities.

A partial validation of the method can be attempted for cohesive layers of the examined soil profile at the points where the results of oedometer tests, during which the \( \sigma'_{\text{oi}} \) have been measured, run on high quality undisturbed samples are available.

Table 2 reports the measured values of \( V_{\text{shi}} \) and \( V_{\text{hi}} \), together with the values of OCR results from oedometer tests. The comparison reported in Table 2 seems to be quite promising and suggests further validation of said approach.

Table 2. \( K_0 \) values of cohesive layers inferred from shear wave velocities and from oedometer tests.

<table>
<thead>
<tr>
<th>DEPTH meters below G.L.</th>
<th>OEDOMETER TESTS</th>
<th>SEISMIC TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma'_{\text{hi}} ) (kPa)</td>
<td>OCR</td>
<td>( K_0 )</td>
</tr>
<tr>
<td>15.85</td>
<td>215</td>
<td>2.09</td>
</tr>
<tr>
<td>34.85</td>
<td>330</td>
<td>2.33</td>
</tr>
<tr>
<td>48.10</td>
<td>440</td>
<td>1.93</td>
</tr>
<tr>
<td>54.82</td>
<td>520</td>
<td>1.91</td>
</tr>
</tbody>
</table>

\[
(*) \quad K_0 = K_{\text{sh}}^{(OCR)} = 0.58 \cdot (OCR)^{0.42} \quad \text{with} \quad 0.42 < c < 0.52
\]

RELATIONSHIP FROM OEDOMETER TESTS ALLOWING TO MEASURE \( \sigma'_{\text{oi}} \).

REFERENCES


STIFFNESS OF TOYOURA SAND AT SMALL AND INTERMEDIATE STRAIN

JAMIOLKOWSKI, M., LANCELLOTTA, R., LO PRESTI, D.C.F. and PALLARA, O.
Technical University of Torino, Italy

SYNOPSIS: This paper presents a limited series of laboratory tests performed on Toyoura sand using different apparatuses allowing to measure the strain down to $10^{-6}$. The paper is aimed at investigating the stiffness of the test sand at small and intermediate strain, analyzing the shear moduli $G$ as yielded by the different types of tests and comparing their results against those obtained on the same soils by a number of Japanese researchers.

INTRODUCTION

The stiffness of soils at small ($\epsilon < 10^{-5}$) and intermediate ($10^{-5} < \epsilon \leq 10^{-3}$) strain raises a number of relevant problems in geotechnical and earthquake engineering. It is accepted that within the range of strain $\epsilon < 10^{-5}$ the soil exhibits an apparently linear elastic response and after exceeding a certain threshold value the stress-strain relationship becomes highly non-linear (see as an example Hardin (1978), Jardine et al. (1984), Jardine (1985), Burland (1989), Tatsuoka and Shibuya (1991)). The evaluation of the stiffness in the above mentioned range of strains has been recently enhanced thanks to:

- The improvements of the conventional triaxial (TX) apparatuses obtained by reducing the equipment compliance [Ladd and Dutko (1985), Tatsuoka (1988)] and by implementing a new set of sensors for the assessment of the local axial strain [Symes and Burland (1982), Goto et al. (1991)], making the value of $\epsilon_a$ measurements reliable down to the strain of the order of $10^{-6}$.

- The development of a new generation of torsional shear apparatuses housing long hollow cylindrical specimens (THCS) in which the distribution of the shear strain $\gamma$ is quite uniform. Thus the values of $\gamma$ measured at the top of the specimens are reliable hence scarcely affected by the bedding errors.

This paper presents the results of a series of monotonic THCS, of resonant column (RC) and TX tests performed on specimens of dry, fine Toyoura sand (TOS) having different porosities and stress-strain histories. The test program has been devised with the aim to investigate the soil moduli as obtained from different laboratory tests, to clarify the influence of the mechanical overconsolidation on the shear stiffness $G$ of TOS, and to compare the writers' results with those obtained, on the same test sand, by other researchers (e.g. Tatsuoka et al. (1986), Tatsuoka and Shibuya (1991)) who have remarked that the difference of the mechanical properties of Toyoura sand in dry and saturated state is negligible.

This soil is a predominantly quartz fine sand, having a mean size of 0.16 mm, the uniformity coefficient of 1.3 and the specific gravity of 2.59 kN/m³. The maximum ($e_{max}$) and minimum ($e_{min}$) void ratio are respectively equal to 0.977 and 0.605.

EQUIPMENT

The tests were performed using TX and THCS apparatuses which are available in the geotechnical laboratory at the writer's University. A TX apparatus, housing solid cylindrical specimens 70 mm in diameter and 140 mm in height and controlled by a PC through a 16 bit A/D converter, was used.

As shown in Fig. 1 four independent values of $\epsilon_a$ have been measured during the early stage of the shearing phase of such tests with the aim to separate the effect of different bedding errors and of the equipment compliance [Jardine et al. (1984)]. The following measurements were involved:

a. local $\epsilon_a$ using high resolution submergible LVDTs;
b. internal to the cell $\epsilon_a$ between the pedestal and the cap using high resolution Proximeters;
c. external to the cell $\epsilon_a$ using the same Proximeter as above;
d. external to the cell $\epsilon_a$ using a conventional low resolution LVDT.

The radial deformations ($\epsilon_r$) were all measured locally by means of said Proximeters. Furthermore, the volumetric strain ($\epsilon_v$) has been assessed using high precision automatic system designed by Lo Presti (1987). A fixed-free RC apparatus has been adapted to enable the execution of the monotonic torsional shear tests, see for details Lo Presti et al. (1993). This apparatus, whose schematic layout is shown in Fig.2, allows the measurements of $\epsilon_a$, $\epsilon_r$ as well as the angular displacement of the specimen's head using high resolution Proximeters. It houses a hollow cylindrical specimen, 142 mm in height and whose outer and inner diameters are 71 and 50 mm respectively. A steel strand passing through the internal cavity of the hollow cylinder allows to transmit the

TEST SAND

Toyoura sand is a well known Japanese test sand whose stress-strain characteristics have been thoroughly explored by numerous researchers [e.g. Tatsuoka et al. (1986), Tatsuoka and Shibuya (1991)].
axial load to the top cap enabling the anisotropic consolidation. The maximum achievable in this apparatus is less than $1 \cdot 10^{-3}$.

The maximum $-y$ achievable in this apparatus is less than $1 \cdot 10^{-3}$.

Air cylinder-------

\(\text{(a)}\) \(\varepsilon_y\), external proximeter
\(\text{(b)}\) \(\varepsilon_y\), internal proximeter
\(\text{(c)}\) \(\varepsilon_y\), local LVDTs

Load cell

Figure 1 - Layout of triaxial cell.

\(\varepsilon_y\) proximeter

Coils

Magnets

Hollow cylinder specimen

Ball bearing

Air cylinder

Concrete anchor block

Figure 2 - Layout of torsional shear-resonant column apparatus.

TEST PROGRAM

Four anisotropically consolidated monotonic torsional shear (THCS) tests and three triaxial compression tests at radial stress $a_r = \text{const.}$ (TX) have been carried out on normally (NC) and overconsolidated (OC) specimens of dry pluvially deposited TOS. Two TX tests have been performed on isotropically consolidated specimens (CID) and one on $K_0$-consolidated specimen (CK$_0$. D). The THCS and TX tests were run; stress controlled ($\sigma = 0.3$ to 0.5 kPa/min) and strain controlled ($\varepsilon_y = 1 \cdot 10^{-4}$/min, respectively). RC tests have been performed on specimens that have been subject to THCS, after the monotonic torsional shear stage. Thereafter, the same specimens have been subject again to the monotonic torsional shear with the aim to investigate the influence of cyclic prestraining ($\varepsilon_y$ prestraining (3.3 to 7.3) $10^{-4}$) originated by the former RCT's on G. Due to length constraint this aspect of the research will not be discussed in this paper except for the inclusion of the values of the initial shear modulus $G_0$ in the Fig.4, which for all practical purposes resulted very similar to those measured before cyclic prestraining.

The initial conditions of all tested specimens prior to shear stage are illustrated in Table 1.

Figure 3 - Comparison of axial strain measured by different sensors during early stage of triaxial test.

Figure 4 - Initial shear modulus of Toyoura sand from different laboratory tests.

TEST RESULTS AND RELATED COMMENTS

The analysis of the obtained experimental data is limited to a few key issues and is presented in a concise format. Within this frame the following can be anticipated:

Axial strain measured in TX tests
The discrepancies between the local $\varepsilon_y$ (LVDT) and the external $\varepsilon_y$ (Proximeter) resulted to be negligible at small strain ($<5 \cdot 10^{-5}$) but
increased with increasing the strain and becoming relevant for the assessment of the stiffness in the range of strain from $10^{-3}$ to $10^{-2}$; see Fig.3. This is in good agreement with the findings by Goto et al. (1991) and Tatsuoka and Shibuya (1991). From Fig.3 it can also be inferred that the external $e_c$ measured using poor resolution LVDT does not allow a reliable assessment of the stiffness, at least at a strain lower than $2 \cdot 10^{-4}$, despite the quite limited compliance and bedding errors exhibited by the TX apparatus used.

**Initial shear stiffness $G_0$**

The values of $G_0$ obtained from different laboratory tests are plotted in Fig.4 as function of the mean consolidation stress $p'$. Such values have been measured in the range of shear strains $\gamma$ less than the elastic threshold ($\gamma_f$) strain that, in case of TOS for all practical purposes depends only on $e$, $p'$ and on the sand fabric and is only marginally influenced by the type of loading (e.g., monotonic vs cyclic) and by the mechanical overconsolidation. What above stated is in agreement with what has been found for several predominantly silica granular soils, including the TOS, by Tatsuoka and Shibuya (1991) and Iwasaki et al. (1978) from RCT’s and by Teachavorasinsuk (1989) from THCS’s also displayed confirming the validity of the conclusions that have been drawn on the $G_0$ and testifying a good reproducibility of the laboratory tests examined.

**Table 1. Initial Conditions of Specimens**

<table>
<thead>
<tr>
<th>Type of Test</th>
<th>Test</th>
<th>$e_c$</th>
<th>OCR</th>
<th>$\varepsilon'_s$</th>
<th>$\varepsilon'_r$</th>
<th>$\varepsilon'<em>s</em>{\max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>THCS(*)</td>
<td>1</td>
<td>0.871</td>
<td>-</td>
<td>1.04</td>
<td>0.52</td>
<td>1.04</td>
</tr>
<tr>
<td>THCS(*)</td>
<td>2</td>
<td>0.751</td>
<td>1</td>
<td>1.20</td>
<td>0.47</td>
<td>1.20</td>
</tr>
<tr>
<td>THCS(*)</td>
<td>3</td>
<td>0.906</td>
<td>2.75</td>
<td>1.06</td>
<td>0.77</td>
<td>2.91</td>
</tr>
<tr>
<td>THCS(*)</td>
<td>4</td>
<td>0.758</td>
<td>2.75</td>
<td>1.08</td>
<td>0.71</td>
<td>2.97</td>
</tr>
<tr>
<td>TX-CID</td>
<td>1</td>
<td>0.783</td>
<td>3.04</td>
<td>1.43</td>
<td>1.40</td>
<td>1.43</td>
</tr>
<tr>
<td>TX-CID</td>
<td>2</td>
<td>0.770</td>
<td>1</td>
<td>1.28</td>
<td>1.08</td>
<td>1.28</td>
</tr>
<tr>
<td>TX-CIK</td>
<td>3</td>
<td>0.690</td>
<td>1</td>
<td>1.89</td>
<td>0.86</td>
<td>1.89</td>
</tr>
</tbody>
</table>

(*) specimens subject to the monotonic torsional shear followed by the RCT’s and again by a monotonic torsional shear.

**Shear stiffness $G$**

At strains higher than the elastic threshold the stress-strain response of TOS becomes highly non-linear and the magnitude of the tangent shear modulus $G$ is influenced by the type of loading and by the stress history, see Fig.6.

In Fig.7 the decay of the secant shear stiffness $G_s$ normalized with respect to $G_0 = f$ as function of the shear stress ratio $f$ is shown, being $f(TX)=q/q_{\max}$ and $f(RC)=f(THCS)=r/r_{\max}$, where $q_{\max}$ and $r_{\max}$ correspond to the deviator and to the shear stress at failure, respectively. It can be noticed that the hyperbola can very hardly reproduce the observed soil non-linearity for monotonic loading conditions. On the contrary, this kind of fit of experimental stress-strain curves seems to work quite well in case of cyclic loading e.g. the RCT’s.

**Figures 5 - Poisson's coefficient of Toyoura sand.**

**Figures 6 - Shear stiffness of Toyoura sand vs. shear strain.**


**REFERENCES**


**ACKNOWLEDGMENT**

The authors wish to acknowledge the valuable help by Mr. M. Raino and Mr. R. Maniscalco in setting up triaxial apparatus and running the tests.

**REFERENCES**


FLAT DILATOMETER TESTS IN TOYOURA SAND

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SYNOPSIS: The paper presents the results of 18 flat dilatometer tests performed in a large Calibration Chamber on dry Toyoura sand. These results have been evaluated to assess the capability of the device for determining the coefficient of earth pressure at rest and the stiffness of the test sand.

INTRODUCTION

The results of 18 flat dilatometer tests performed in a large Calibration Chamber (CC) on dry pluvially deposited Toyoura sand (TOS) are presented and discussed with the aim to get a deeper insight of the capability of such devices for assessing the design parameters of sand deposits in situ.

Two types of dilatometers have been employed in this research, namely the standard Marchetti’s Flat Dilatometer (DM), see Marchetti (1980) and the Research Dilatometer (RD) developed by ISMES of Bergamo and described by Fretti (1990) and Fretti et al. (1992). The two devices are identical as to shape and size but the latter has been equipped with some instrumentation which has made possible the measurement of the deflection at the centre of the membrane, the inflation pressure at the membrane and the penetration force immediately above the blade. This last measurement is also carried out during DM. A similar research device has been developed in middle eighties at the University of British Columbia in Vancouver and is illustrated in the works by Campanella et al. (1985) and Campanella and Robertson (1991).

The RD allows to obtain a continuous relationship between the inflation pressure \( p \) and the deflection at the centre of the membrane \( \delta \) leading to a \( p \) vs \( \delta \) being similar in shape to the pressuremeter expansion curve [Campanella et al. (1985), Fretti (1990)]. However, due to length constraint, this paper focuses on the conventional interpretation of the tests as outlined by Marchetti (1980), and does not deal with the discussion of the entire dilatometer expansion curves and with the unload-reload loops that can be performed during the RDT’s, see Campanella and Robertson (1991) and Fretti et al. (1992).

Tests Program

18 dilatometer tests have been performed in CC specimens whose relative density (DR) ranges between 39 and 95%, see Table 1. Out of them nine have been carried out using RD while the remaining 9 by means of the conventional DM. With the exception of the RD test n°326, all CC specimens have been penetrated under constant vertical \( a^v \) and horizontal \( a^h \) effective stresses, B-1 boundary condition. Specimen 326 has been penetrated under the condition of \( a^v \) = const. and zero lateral strain, B-3 boundary condition.

In order to investigate the response of the dilatometers to the changes in \( e'_v \), six multistage CC tests (MST) have been carried out (the result of RD - MST n°327 is not reported in Table 1). During such tests, the CC specimens were firstly penetrated to the depth of 0.6 m under a given set of \( e'_v \) and \( e'_h \), thereafter, with the blade kept stopped, the \( a^h \) was increased, generally almost doubled, maintaining the \( a^v \) unchanged and the penetration was completed at the depth of 1.2 m.

Reduction of Tests Data

For all tests the following dilatometer indexes have been computed (Marchetti (1980) and Campanella and Robertson (1991)):

- Material Index: \( I_D = (p_1 - p_0) / (p_0 - u_0) \)
- Horizontal Stress Index: \( K_D = (p_0 - u_0) / e'_D \)
- Dilatometer Modulus: \( E_D = C (p_1 - p_0) \)
- Dilatometer Wedge Resistance: \( q_D = (p_1 - \Lambda_1 \cdot \gamma_1) / h_1 \)

\( p_0 \) = corrected lift of pressure
\( \rho_1 \) = corrected pressure for the expansion of the dilatometer membrane of 1.1 mm and 1.0 mm in case of DMT and RDT respectively

\( \rho_0 \) = hydrostatic pore pressure

\( F_D \) = thrust force measured just above the blade

\( t_s \) = soil-blade interface friction assumed to be equal to the local shaft friction \( f_s \) measured during the Cone Penetration Tests (CPT)

\( A_f \) = lateral area of the dilatometer blade = 0.00128 m\(^2\)

\( A_b \) = tip area of the dilatometer blade = 0.0313 m\(^2\)

\( C \) = constant equal to 34.6 and 38.2 for DM and RD respectively. KD/Ko

\( K_0 = \) ratio of \( \rho_1^p/\rho_1^H \) prior the RD and DM penetration from the K11 test results in a satisfactory manner. This situation is mostly linked to the fact that the penetration of any device, in the considered case of the dilatometer blade, produces the following conditions:

- a pronounced increase of the horizontal effective stress \( \rho_1^H \) above its hydrostatic pore pressure
- a large straining of the sand surrounding the dilatometer blade. This situation is reflected in the following equation linking the dilatometer amplification factor \( K_D/K_0 \) to the state parameter \( \psi \) [Been and Jefferys (1985)], which fits the CC tests results obtained in TOS:

\[ K_D/K_0 = 1.05 \exp (3.07 \psi) \]

\( K_0 \) = ratio of \( \rho_1^p/\rho_1^H \) prior the RD and DM penetration from the Table 1.

The above equation shows that the ratio of \( K_D/K_0 \) is a complex function of the mean effective stress and of the void ratio of the ground and not only of the \( \rho_1^H \). The amplification factor increases exponentially as the \( \psi \) increases.

Table 1. Results of Dilatometer Test in Dry Toyoura Sand

<table>
<thead>
<tr>
<th>Test</th>
<th>BC</th>
<th>z</th>
<th>e</th>
<th>DR</th>
<th>( \rho_1^e )</th>
<th>( \rho_1^c )</th>
<th>( \rho_0 )</th>
<th>( \rho_1 )</th>
<th>GCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>324(*)</td>
<td>B-1</td>
<td>75</td>
<td>0.660</td>
<td>85.1</td>
<td>113</td>
<td>116</td>
<td>904</td>
<td>3256</td>
<td>7.2</td>
</tr>
<tr>
<td>325(*)</td>
<td>B-1</td>
<td>70</td>
<td>0.456</td>
<td>86.2</td>
<td>113</td>
<td>50</td>
<td>434</td>
<td>3964</td>
<td>1.0</td>
</tr>
<tr>
<td>326(*)</td>
<td>B-3</td>
<td>75</td>
<td>0.624</td>
<td>94.5</td>
<td>112</td>
<td>64</td>
<td>593</td>
<td>2770</td>
<td>1.0</td>
</tr>
<tr>
<td>327(*)</td>
<td>B-1</td>
<td>70</td>
<td>0.756</td>
<td>59.3</td>
<td>111</td>
<td>51</td>
<td>430</td>
<td>1604</td>
<td>1.0</td>
</tr>
<tr>
<td>328(*)</td>
<td>B-1</td>
<td>75</td>
<td>0.807</td>
<td>45.7</td>
<td>111</td>
<td>55</td>
<td>239</td>
<td>1143</td>
<td>1.0</td>
</tr>
<tr>
<td>329(*)</td>
<td>B-1</td>
<td>75</td>
<td>0.806</td>
<td>46.0</td>
<td>112</td>
<td>107</td>
<td>340</td>
<td>1474</td>
<td>1.0</td>
</tr>
<tr>
<td>416 B-1</td>
<td>75</td>
<td>0.772</td>
<td>52.2</td>
<td>112</td>
<td>66</td>
<td>452</td>
<td>1579</td>
<td>1.9</td>
<td></td>
</tr>
<tr>
<td>417(*) B-1</td>
<td>55</td>
<td>0.625</td>
<td>40.8</td>
<td>109</td>
<td>56</td>
<td>257</td>
<td>943</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>418(*) B-1</td>
<td>75</td>
<td>0.821</td>
<td>41.8</td>
<td>115</td>
<td>112</td>
<td>389</td>
<td>1376</td>
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</tr>
<tr>
<td>419 B-1</td>
<td>75</td>
<td>0.755</td>
<td>59.7</td>
<td>112</td>
<td>74</td>
<td>177</td>
<td>1971</td>
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<td></td>
</tr>
<tr>
<td>420 B-1</td>
<td>75</td>
<td>0.760</td>
<td>58.3</td>
<td>111</td>
<td>50</td>
<td>282</td>
<td>1648</td>
<td>1.0</td>
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</tr>
<tr>
<td>421 B-1</td>
<td>75</td>
<td>0.754</td>
<td>56.9</td>
<td>111</td>
<td>58</td>
<td>457</td>
<td>1589</td>
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</tr>
<tr>
<td>422(*) B-1</td>
<td>75</td>
<td>0.753</td>
<td>60.2</td>
<td>115</td>
<td>77</td>
<td>420</td>
<td>2989</td>
<td>2.7</td>
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<tr>
<td>423(*) B-1</td>
<td>70</td>
<td>0.746</td>
<td>62.5</td>
<td>113</td>
<td>126</td>
<td>470</td>
<td>2515</td>
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<tr>
<td>424(*) B-1</td>
<td>75</td>
<td>0.633</td>
<td>92.5</td>
<td>112</td>
<td>113</td>
<td>784</td>
<td>2309</td>
<td>7.0</td>
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<tr>
<td>425(*) B-1</td>
<td>70</td>
<td>0.636</td>
<td>91.1</td>
<td>113</td>
<td>56</td>
<td>445</td>
<td>2490</td>
<td>1.0</td>
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</tr>
<tr>
<td>426(*) B-1</td>
<td>70</td>
<td>0.633</td>
<td>92.5</td>
<td>117</td>
<td>109</td>
<td>615</td>
<td>3261</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>427(*) B-1</td>
<td>48</td>
<td>0.633</td>
<td>92.5</td>
<td>102</td>
<td>158</td>
<td>1015</td>
<td>4587</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>428(*) B-1</td>
<td>70</td>
<td>0.527</td>
<td>94.1</td>
<td>318</td>
<td>307</td>
<td>1499</td>
<td>5938</td>
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</tr>
<tr>
<td>429(*) B-1</td>
<td>72</td>
<td>0.604</td>
<td>47.3</td>
<td>312</td>
<td>558</td>
<td>556</td>
<td>2134</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>430(*) B-1</td>
<td>72</td>
<td>0.797</td>
<td>48.3</td>
<td>320</td>
<td>312</td>
<td>786</td>
<td>2789</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

(*) Research Dilatometer; (**) Multistage tests

\( e_{\rho_1}^H \) = vertical and horizontal effective stress, respectively, prior dilatometer penetration

Engineering Correlations

This paper is intended as a preliminary presentation of the acquired RDT’s and DMT’s data. Therefore, only minor comments concerning the engineering correlations are presented.

At the present stage of the research the following can be anticipated:

1. Coefficient of earth pressure at rest \( K_0 \)

The estimation of \( K_0 \) in sands from penetration tests results is, in its broad outlines, rather a complicated problem and far from being solved in a satisfactory manner. This situation is mostly linked to the fact that the penetration of any device, in the considered case of the dilatometer blade, produces the following conditions:

- a pronounced increase of the horizontal effective stress \( \rho_1^H \) above its hydrostatic pore pressure
- a large straining of the sand surrounding the dilatometer blade. This situation is reflected in the following equation linking the dilatometer amplification factor \( K_D/K_0 \) to the state parameter \( \psi \) [Been and Jefferys (1985)], which fits the CC tests results obtained in TOS:

\[ K_D/K_0 = 1.05 \exp (3.07 \psi) \]

\( K_0 \) = ratio of \( \rho_1^p/\rho_1^H \) prior the RD and DM penetration from the Table 1.

The above equation shows that the ratio of \( K_D/K_0 \) is a complex function of the mean effective stress and of the void ratio of the ground and not only of the \( \rho_1^H \). The amplification factor increases exponentially as the \( \psi \) increases.

Table 2. Results of Multistage Dilatometer Tests

<table>
<thead>
<tr>
<th>Test</th>
<th>1st STAGE</th>
<th>2nd STAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>N*</td>
<td>( e_1^C )</td>
<td>( e_1^L )</td>
</tr>
<tr>
<td>417 DM</td>
<td>109</td>
<td>56</td>
</tr>
<tr>
<td>418 DM</td>
<td>100</td>
<td>54</td>
</tr>
<tr>
<td>424 RD</td>
<td>112</td>
<td>58</td>
</tr>
<tr>
<td>426 RD</td>
<td>313</td>
<td>157</td>
</tr>
<tr>
<td>427(*) RD</td>
<td>110</td>
<td>57</td>
</tr>
<tr>
<td>428 RD</td>
<td>311</td>
<td>157</td>
</tr>
</tbody>
</table>

\( e_1^V, e_1^H \) = vertical and horizontal stress after penetration at the elevation of the dilatometer test.

(\( \rho_0 \) not reported in Table 1)

Such procedure, valid for freshly deposited silica sands, when applied to RDT and DMT results, obtained in TOS, leads to the data reported in Fig.1. On average, there is an acceptable agreement between the measured \( K_0 \) and that inferred from the dilatometer tests results for NC specimens. For OC specimen, the correlation by Baldi et al. (1986) greatly underestimates \( K_0 \).
K (COMPVTED) = 0.376 + 0.095 Ko · 0.0017 q0/a;

(Baldi et al., 1986)

Fig. 1 - Computed versus measured coefficient of earth pressure at rest.

2. Constrained modulus M

Fig. 2 shows M measured during the one-dimensional compression of the CC specimens compared against the values computed from the dilatometer tests. As already observed for other silica sands tested in the CC's, the procedure suggested by Marchetti (1980) leads to a reliable prediction of M for NC consolidated sand but amply underpredicts that of the mechanically overconsolidated specimens.

Fig. 2 - Computed** versus measured constrained modulus.

NC specimens
OC specimens

(*) Computed from RDTs and DMTs following Marchetti's (1980) procedure.

Fig. 3 - Initial shear modulus as function of Eo and Dk.

G/Eo = 2.96 - 0.02 Dr

Fig. 4 - Ratio of shear to dilatometer moduli as function of strain level.

3. Shear modulus G

As already suggested by Jamiolkowski et al. (1988) and Jamiolkowski and Robertson (1988) the Eo can be reliably correlated to the initial shear modulus G0 measured at shear strain less than to the elastic threshold strain y0 (= 10^-3%), see Jamiolkowski et al. (1991), Tatsuoka and Shibuya (1991) and Shibuya and Tatsuoka (1992). An attempt of such correlation is shown in Fig. 3. The observed decrease of G0/Eo with increasing Dr is typical for all kinds of correlations between deformation moduli and parameters obtained from penetration tests, for further details see Jamiolkowski and Robertson (1988) and Baldi et al. (1989).

In Fig. 4 a tentative correlation between the shear modulus G and Eo at shear strain y > y0 is presented for two values of Dr. The values of G used to work out this correlation have been obtained from the
monotonic torsional shear tests performed by Teachavorasinskun (1989) on NC and OC specimens of TOS. The correlation itself holds for NC sand. The correction factor $\beta$ allowing the estimation of $G = f' (E_D)$ in overconsolidated TOS having OCR $\leq 4$ is also given. The two curves $G/E_D = f(\gamma)$ reported in Fig. 4 for two values of $\gamma$ refer to the moduli at $p' = 100$ kPa.

4. Dilatometer wedge resistance $q_D$

Fig. 5 compares the values of $q_D$ measured during RDT's and DMT's just above the blade against the values of $q_c$ evaluated using equation $q_c = f(a, h, f)$ given above. On average the $q_D$ results 10 to 15% higher than $q_c$, confirming the result presented by Campanella and Robertson (1991).

![CPT cone resistance versus RD and DM wedge resistance.

FINAL REMARKS

A preliminary examination of the results of a limited number of dilatometer tests performed in CC on dry Toyoura sand allows the following comments:
- The RD and DM tests lead to very similar results in the test sand for the rank of relative density ranging between 40 and 95%.
- The evaluation of $K_0$ based on the results of dilatometer tests is still far from being solved mainly because of the complex nature of the amplification factor $K_0/K_0$ involved.
- As already observed the RD's and DMT's allow to predict in a reliable manner the initial shear modulus $G_0$ of sands knowing $E_D$ and the value of $D_B$.
- The procedure suggested by Marchetti (1980) allows to evaluate correctly the constrained modulus of NC sands but underestimates that of OC sands.

The above exposed comments apply only to freshly deposited sands. Much care should be paid if applied to natural sand deposits of the same geological age.

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(for papers co-authored by more than three persons, only the first author is evidenced)


Marchetti, S. (1980). In Situ Tests by Flat Dilatometer. JGED, ASCE, GT3.


STIFFNESS OF CARBONATIC QUIOU SAND
RIGIDITÉ DU SABLE CARBONATIQUE DE QUIOU
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(1) ISMES, Bergamo, Italy
(2) Politecnico di Torino, Italy.

SYNOPSIS
The stiffness of the carbonatic un cemented Quiou sand was measured in laboratory on dry and saturated reconstituted samples by means of Resonant Column, Triaxial and Bender Element tests. Particular attention was devoted to the stiffness at very small (< 10^-3 %) and at intermediate strain level (from 10^-3 % to 0.5 %).

The laboratory apparatus and the testing procedures are briefly described; typical examples of the obtained results are presented. Assuming isotropic elasticity, the experimental data were analysed in terms of maximum shear modulus as a function of the state parameters and the results of the different tests were compared.

Finally, the effects of the overconsolidation on the stiffness are examined, together with some considerations on modelling the stress-strain response by the hyperbolic model.

1. Introduction.

Preliminary results of a series of laboratory tests performed on specimens of dry and saturated carbonatic Quiou sand (QS), are presented. The testing program includes Resonant Column test (RC) on dry samples, drained Triaxial Compression test (TX) on saturated samples, and measurement of shear wave velocity by means of piezoelectrical transducers, called Bender Elements (BE), on both dry and saturated soil samples.

Both RC and TX specimens have been subjected to isotropic consolidation in the range of stress varying between 100 to 600 kPa. While the TX and RC tests allow to explore a wide range of strains, the BE, incorporated on the triaxial cell, enables to measure the shear wave velocity and therefore the shear modulus, only at very small strains (i.e. about 10^-4 %).

The above mentioned laboratory tests are part of a more extensive research effort involving the validation of different in situ devices in crushable and slightly silty sand using the calibration chambers and using numerical modeling (Almeida et al. 1991, Ulloa et al. 1991, Bellotti et al. 1982, 1988, 1991).

Because of length constraint, the proceeding presentation is limited to the exam of the measurements of the shear stiffness at very small (< 10^-3 %) and intermediate strain level (from 10^-3 % to 0.5 %).

2. Laboratory Apparatuses.

The triaxial testing system used for the tests, consists in a stress path triaxial cell with internal tie rods completely controlled by a personal computer. The cell has an internal load cell to measure the axial load without any piston friction. For some of the tests finalised to the determination of stiffness at very small strain, a load cell of 0.5 kN with an accuracy of 0.5 % of the measuring range, was adopted. This means an accuracy of 0.005 kPa in measuring axial stress.

The loading piston can be connected to the sample cap before assembling the pressure cell, therefore some of the binding errors can be avoided or reduced (such as the load and sample alignment). Nevertheless, in order to avoid vertical compliance of the triaxial apparatus, which involves the deformation of the load cell, the vertical and radial deformations of the sample are measured locally by means of the proximity transducers; this allows more reliable Poisson's ratio values, at least at small strains.

The used system of six proximities hanged on two bars, is schematised in figure 1: four out of them are fixed vertically to measure axial strains while two are fixed horizontally to measure the radial strains at the middle height of the sample. The proximity transducers used have a maximum measuring range of 2.5 mm; the resolution (< 0.3 microns) and the accuracy (= 0.3 microns) were obtained from a calibration by a laser interferometer. The vertical targets, supported by pins, are inserted into the specimen and scaled to the membrane, while the horizontal ones are stuck on the membrane.

Figure 1 - Local measurements of axial and radial displacements by proximity transducer.
For the data logging acquisition, an analogical to digital converter of 16 bit was employed capable of reading 40 channels per second. In order to maximise the amplification of the proximities and the loading cell output signals, an analogue recorder was adopted in parallel, only for the first part of the shearing stage.

The triaxial cap and pedestal were instrumented with piezoelectrical transducers called Bender Elements (BE). When a driving voltage is applied to one of the elements, it bends to one side generating a shear wave that propagates along the sample; the other element, which acts as a receiver, converts the deflection due to the particle motion into an electrical signal and thus detects the arrival of the shear wave at the other end of the sample. The test result consists of the measure of the travel time from which the shear wave velocity \( V_s \) is calculated and, assuming isotropic elasticity, the shear modulus is \( G = \rho V_s^2 \) (for details see Brignoli and Gotti 1991).

A fixed-free Resonant Column apparatus was used for solid cylindrical dry specimens, consolidated isotropically. Since the apparatus is well known, details are neglected. Volume and height variations were measured during consolidation steps and dynamic shearing phases.

3. Test sand and testing procedures.

The physical properties of QS are summarised in figure 2. It is a sub angular well graded, coarse to medium sand, containing about 2% of fines, as shown in figure 2, where are also indicated the minimum (obtained by pluviation) and the maximum (ASTM D4254-83) unit weight and the correspondent void ratios.

A high degree of crushability, even at small compressing or shearing stresses, is its peculiarity, which causes a continuous relevant change of void ratio, grain size distribution and grain morphology during the tests; the grain size distribution after TX testing is also shown in figure 2.

The specimens of 70 mm in diameter and 140 mm in height, were reconstituted by pluvin deposition using a travelling sand spreader (Passalacqua 1991). Only for some RC tests smaller samples were tested (i.e. 50 mm in diameter and 110 mm in height).

The tests, whose results are being presented, were isotropically consolidated within the range of stress varying from 100 to 600 kPa; after the consolidation, 12 hour's rest was observed to allow a significant stabilisation of strains. The BE tests were performed at the end of the rest period.

The shearing stage of the TX tests was carried out in drained conditions, under a strain controlled frame, at a constant rate of axial strain (about 10^-4 % per second). The radial pressure was kept constant or adjusted in order to keep constant the mean effective stress \( p' \).

For dry samples (RC and some BE tests) the current void ratio was evaluated from the volume changes measured by a special device capable to measure the air volume changes.

4. Test results.

A first part of a typical stress-strain relationship obtained from a TX test (CID304P), is reported in figure 3, where the deviatoric stress is plotted against the deviatoric shear strain. The experimental data are fitted by small segments, whose slope is taken as tangent modulus. The shear modulus so evaluated is also shown in figure 3 versus the correspondent shear strain.

A typical trend of the Poisson's ratio versus the axial strain, is shown in figure 4. No substantial differences can be noted between the strains locally measured and those derived by mean of the external measures of the axial strain and the volume changes. It is important to note that the \( v \) values start at
0.27 at very small strain, then decreases to 0.1 at \( \varepsilon = 0.1 \% \) successively increases for large strains. Such behaviour, usually noted in the tests so far performed, can be ascribed to the crushability and the consequent rearrangement of the particles of the QS which increases as the strain increases up to a certain value (e.g. 0.5 %). Furthermore, values of \( v = 0.25 - 0.27 \) have been found by measuring shear wave velocity and compression wave velocity using BE and compression piezoclectrical transducers both on the same dry specimens.

Some of the results obtained from RC tests on low density reconstituted samples, are reported in figure 5 as shear modulus normalised by its maximum value, versus shear strain. The samples were consolidated at three different isotropic pressures (i.e. \( p' = 100, 250 \) and 400 kPa). The three samples consolidated at \( p' = 100 \) kPa, show the same trend and, although there is a small difference between the \( G_0 \) values, they reflect the repeatability of the tests. Finally it can be seen that, as the pressure increases, \( G_0 \) increases and the normalised curve is shifted on the right, such that the strain level dependency of \( G \) decreases.

![Figure 4 - Poisson's ratio versus axial strain from a TX test.](image)

The mechanical response of geotechnical material, like QS, is generally elasto-plastic; nevertheless, it has been remarked by many researches that at very small strains (\( \varepsilon < 10^{-5} \)) the value of the shear modulus is practically independent of the loading conditions (i.e. static or dynamic, monotonic or cyclic) and the material shows a negligible hysteresis, therefore, from an engineering point of view, the stiffness can be assumed elastic within this range of strain (Tatsuoka and Shibuya 1991). Furthermore, the magnitude of the elastic threshold strain depends on the type of soil, on its geological and stress history, on the strain rate at which the sample is tested and on the cyclic prestraining (e.g. Dobry and Vucetic 1987, Ampadu 1991).

As initial shear modulus, \( G_0 \) is inteded the value of maximum stiffness resulting from the RC tests in the range within which \( G_0 \) remain constant, while from TX tests the \( G_0 \) is assumed to be one third of the slope of the initial segment of the \( q - \varepsilon_a \) relationship.

It was established that \( G_0 \) mainly depends on state parameters such as void ratio \( e \) and mean effective stress \( p' \) (Haddad and Doversieh 1972), therefore it can be expressed as:

\[
G_0 = f(e)p'(1 + e)
\]

In order to define a formulation of this functions, two assumptions are made supported by the experimental observations:

1. the shear wave velocity is linearly dependent on the void ratio, so it can be assumed:

\[
f(e) = (b - c)e/(1 + e)
\]

2. the dependency of \( G_0 \) on \( p' \) can be expressed by a power function, so:

\[
H(p') = p'^n
\]
In conclusion the shear modulus can be written as:

\[ G_0 = C \cdot \left[ \frac{(b - e)^2}{(l + e)} \right] (p' \cdot 0.38) \cdot p' \cdot 0.62 \]

where:
- \( C \) = non dimensional material constant function of the soil type (multiplying \( C \) by \( F(e) \) one obtains the non dimensional modulus number \( K_g \));
- \( p' \) = reference pressure = 1 MPa.

The equation that better fits the RC tests results, becomes:

\[ G_0 = 81.4 \cdot \left[ \frac{(4.16 - c)^2}{(1 + e)} \right] (p' \cdot 0.38) \cdot p' \cdot 0.62 \]

In figure 6 it is reported a comparison between the above formula and all experimental data (RC, BE and TX), in terms of \( G_0 \) at \( p' = 1 \text{ MPa} \) versus \( e \). It can be noted that all the RC tests results are located on the fitting curve within a range of \( \pm 5\% \), while for BE and TX tests results are more scattered for low density samples \((e < 0.95)\). The \( G_0 \) values from BE are within the same range for high density samples \((e > 0.9)\) but show generally higher values than RC and TX at medium and low density \((e > 0.9)\).

The reasons for such a discrepancy are not easy to explain considering the theoretical frame assumed (elasticity). In first approximation, however, one can postulate that the observed difference between \( G_0 \) (BE) and \( G_0 \) (RC) measured might be linked to the following factors:
- difficulties in the determination of the first shear wave arrival on the receiver transducer which seems to be greater on saturated samples;
- uncertainties in the calculated \( V_s \), due to the relatively short, few centimetres, travel path of the specimen;
- inherent difficulties in comparing different laboratory tests (the testing frequencies are different for the two tests), which is maximised because of the high sensitivity of the QS, for example to the time effects, i.e. aging during consolidation and strain rate during shearing.

Figure 6 - Influence of \( e \) on \( G_0 \).

Effects of the overconsolidation.

The effects of the overconsolidation (OCR) on the stiffness of QS are examined on a limited number of TX tests and on 13 RC tests. In figure 7 the results of two TX tests are compared in terms of normalised secant shear modulus \((G/G_0)\) against axial strain. Note that the elastic threshold strain becomes larger and the strain level dependency of \( G \) decreases. As the strain increases, the OCR heavily affects the secant shear modulus.

Nevertheless, contrarily to what is observed for silica sand \([e.g.\ Tatsuoka\ and\ Shibuya\ (1991)]\), the mechanical overconsolidation seems to influence the initial shear modulus, see figure 8, where the RC tests results are reported. Since the pressure levels and the void ratio were different between NC and OC samples, the comparison was made by normalising the \( G_0 \) on the \( G_0 \) calculated for NC samples. The best fitting of such results gives:

\[ G_0 (OC) = G_0 (NC) (OCR)^{0.31} \]

Figure 7 - Effect of OCR on \( G_0 \) from TX tests.

Figure 8 - Effects of OCR on \( G_0 \) from RC tests.
The reasons for such behaviour is not completely clear and deserve further laboratory tests. However it can be postulated that the increasing of stress produces a crushing of the particles and their rearrangement, with consequent increase of the number of inter-particle contacts. Upon unloading, the recoverable deformation is so low that the number of inter-particle contacts remains almost unchanged and appreciably higher than those of a virgin sample at the same $p'$. The effect of crushing apparently is not adequately reflected in the actual void ratio function.

The influence of soil non-linearity and OCR on modelling stress-strain response of soils is evidenced in figure 9, which shows the ratio of $q/a_{max}$ (TX) and $t/t_{max}(RC)$ on the horizontal axis versus the ratio $E/E_0$ (TX) and $G/G_0$ (RC) on the vertical axis.

Both $E$ and $G$ correspond to the secant deformation moduli, the diagonal line shown in figure 9 reflects the hyperbolic stress-strain relationship. It is clear that the NC exhibits a non-linearity much higher than the one that can be obtained by the hyperbolic stress-strain model. As the OCR increases, the stress strain curves moves closer to the hyperbola which better fits the data from RC tests; for similar results see Tatsuoka and Shibuya (1991) and Jamiolkowski et al. (1991).

![Figure 9](image)

**Figure 9 - Normalised secant shear and Young moduli versus shear stress level.**


1. Local measurements of axial and radial deformations in a TX test are suggested to obtain more reliable data, to increase the precision in measuring stiffness at very small strains, so the data can be compared to other tests (RC and BE).

2. For NC samples, the maximum shear modulus $G_0$ depends principally on the state parameters: void ratio and mean effective stress. For practical purposes by fitting the experimental data, a similar expression can be found:

$$G_0 = F(e) H(p') = 61.4 (4.16 - e)^2 (1 - e) p'_e^{0.38} p'^{0.62}$$

The increase of the consolidation pressure has the effect of decreasing the strain level dependency of the shear modulus.

3. Similar magnitudes of the initial stiffness at comparable void ratio have been found from RC and TX tests, while $G_0$ values from BE are generally 20 % greater at low density. More tests are required in this direction.

4. When QS is strained beyond the elastic threshold, its stress-strain response appears highly non linear.

5. The effects of the OCR on QS can be summarised as follows:

- increasing the elastic threshold
- decreasing the strain level dependency of $G$
- increasing the maximum value of the shear modulus.

6. Assuming $E_0$ or $G_0$ to be the initial stiffness of QS the hyperbolic model does not fit its stress-strain curve. Nevertheless, results from RC tests and generally for samples with high OCR, seem to be closer to the hyperbolic relationship.

7. Acknowledgements.

All the tests presented in this paper were performed at ISMES: The authors would like to thank Mr. O. Hanleury for the execution of the RC tests and for being involved in the interpretation analysis of the results; Mr R. Capoferrì for the execution of the triaxial tests and for the support given.

8. References.


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S. LEROUEIL - Professor, Laval University, Quebec, Canada  
D.C.F. LO PRESTI - Research Assistant, Technical University of Turin, Italy

Theme Lecture: Design Parameters from Theory to Practice

SYNOPSIS

An overview of the qualitative stress-strain and strength behaviour of lightly and moderately over consolidated natural clays is presented. The relevance of the soil structure and of the geological time on mechanical behaviour of clays is pointed out. Finally, a number of specific issues related to the evaluation of the initial state variables, stiffness and strength are discussed making mainly reference to the outcome of the most recent laboratory test results. More practice oriented problems related to the selection and use of the design parameters in stability, settlements and consolidation analyses are covered by the companion General Report presented to this session.

1. INTRODUCTION

This Theme Lecture (TL) summarises the recent outlook on the fundamental behaviour of natural soft clays. The examination of more practice oriented problems, like assessment of design parameters, their link with the method of analysis and their relation with the examined boundary value problem, will be treated in the Companion General Report (GR) by Leroueil and Jamiolkowski.

Because of the extremely broad topic of this TL, and considering the many valuable and exhaustive State of the Art Reports on Soil Behaviour that have been published in the last six years [Wroth and Houlsby (1985), Jamiolkowski et al. (1985), Tavenas and Leroueil (1987), Yudhbir and Wood (1989), Atkinson and Sallfors (1991), Burghignoli, Sagaseta et al. (1991)], the presentation is mainly restricted to a few specific areas which have recently experienced some significant developments that have brought a better understanding of the mechanical behaviour of natural soil deposits. Therefore, the writers believe that the following issues deserve to be examined:

- A qualitative description of the mechanical behaviour of natural soils pointing out the continuous nature of the deformation process from elastic behaviour, through yielding to peak-resistance and eventually at large strain towards critical state.

- A brief discussion on the initial state variables with reference to the estimate of the preconsolidation pressure ($\sigma'_p$) and of the coefficient of earth pressure at rest ($K_0$).

- The analysis of the parameters that characterize the soil stiffness in both undrained and drained conditions, making reference to the results of laboratory tests. Particular attention will be devoted to the main factors controlling the drained stiffness of soils.
- A discussion on parameters that characterise the shear strength in both undrained and drained conditions referring also to laboratory tests.

In examining the above, special attention will be devoted to the following aspects:

- the recent experience gained in laboratory with strain measured locally directly on the samples and, generally speaking, to the improved accuracy in the measurement of strains during laboratory tests:

- the influence of geological time on the behaviour of natural soils in relation with the development of their structure through processes like: aging, early diagenesis, precipitation of cementing agents, etc.
GEOTECHNICAL CHARACTERIZATION OF GRAVELLY SOILS AT MESSINA SITE

SELECTED TOPICS

by

Crova, R.¹, Jamiołkowski, M.², Lancellotta, R.² and Lo Presti, D.C.F.²

SUMMARY

The paper summarizes the experience gained in the geotechnical characterisation of sand and gravel deposits, in connection with the design of the one span suspended bridge over the Messina Strait. The discussion is focused on the development of a dynamic penetration test using a spoon sampler larger than the one employed in Standard Penetration Tests (SPT) with the aim of investigating the possible influence of gravel particles on the measurements with the SPT. The second part of the paper is devoted to a discussion on the influence of the geological age of granular deposits on the penetration resistance and stiffness.

1. INTRODUCTION

Due to the extreme difficulties and high costs connected with undisturbed sampling and with the difficulty of performing many important in situ tests, the characterisation of gravelly soils for design purposes still represents an uneasy task of the experiments soil engineering. Therefore, the writers will illustrate some of the problems encountered in the geotechnical characterisation of gravelly deposits along the proposed crossing of the Messina Strait with a one span suspended bridge.

The Messina Strait is located in one of the regions with highest seismicity where both atmospheric and marine conditions are extremely hostile. See as an example Table 1 which shows the earthquake and wind conditions that have been considered in designing the bridge.

The subsoil conditions of both sides of the Messina Strait consist of gravelly deposits of holocene and pleistocene age underlain by soft rocks of pliocene and miocene age. Especially on the Sicilian shore, sand and gravel deposits extend to a depth beyond 180 m below the existing G.L.

This paper deals with the geotechnical characterisation of such deposits on the Sicilian side, see cross-section in Fig.1.

The geotechnical investigation for the preliminary design consisted in the operations summarized in Table 2 in addition to the evaluation of the index properties and some laboratory tests performed on samples reconstituted with the assumed in situ density.

One discussion that follows will focus on the following:

- Development and validation of Large Penetration Test (T PT),
- Evaluation of the influence of the geological age of the deposit on the SPT resistance and on the small strain shear modulus ($G_0$) inferred from shear wave velocity ($V_s$) measured using cross-hole techniques.

(1) Studio Geotecnico Italiano, Milano

(2) Technical University of Torino
Crova R., Jamiolkowski H., Lancellotta R. and Lo Presti D. C. F.

Figure 1 - Geological cross section on Sicilian Shore.

Table 1 - Design values for earthquake and wind.

<table>
<thead>
<tr>
<th>Type of test</th>
<th>CALABRIA</th>
<th>SICILY</th>
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</thead>
<tbody>
<tr>
<td>Boring (1)</td>
<td>Foundation</td>
<td>Anchor</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>SPT (2)</td>
<td>7</td>
<td>6</td>
</tr>
<tr>
<td>LFT (2)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>PLT (3)</td>
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<td>NONE</td>
</tr>
<tr>
<td>Pumping test</td>
<td>1</td>
<td>NONE</td>
</tr>
<tr>
<td>CPT</td>
<td>1</td>
<td>NONE</td>
</tr>
<tr>
<td>CH (4)</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>SASW</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 2 - Soil investigation program.
National Report

R&D activities

The Netherlands
The following R&D-activities in the area of geotechnics and geo-environment are undertaken in the Netherlands.

The information is given by prof.dr.ir. F.B.J. Barends (Delft Geotechnics), ing. R.J. Termaat (Ministry of Transport, Public Works and Water Management) and myself (CUR).

NATIONAL KNOWLEDGE-INFRASTRUCTURAL PROJECTS

The Dutch government launched a national stimulation fund to enforce the knowledge infrastructure. The purpose is to enhance collective research and to interest industry by financial commitments (30% to 50%). The project consists of 8 cluster projects, three of which have a strong connection to geotechnics and geo-environment technology.

1. Research program for underground construction (start 1995; managed by the Centre for Civil Engineering and Codes; CUR - Gouda)
   - Boring in very soft soils
     To gain experience and knowledge on shield-tunnelling in typical Dutch soft soils, and to gain insight in the risks of boring.
   - Reconnaissance, prediction and monitoring
     Development of reliable reconnaissance, prediction and monitoring instruments for tunnel construction in soft soils in order to reduce risks.
   - Economic tunnel construction
     Optimization and renewing of tunnel construction techniques for soft soils in order to reduce costs.
   - Construction, management and maintenance
     Achieving the optimum results for the user, the manager and the surroundings.

2. Land and water impuls (start 1994; managed by the Centre for Civil Engineering and Codes; CUR - Gouda)
   - Estuarias and coasts
     Development of a generic decision support system.
   - Rivers
     Development of modular decision support system for the control and maintenance of riversystems.
   - Durable planning and maintenance of industry, harbour and city areas
     Development of partial planning systems for industry plants and roads.
   - Large scale line infrastructure
     Preparation of the European high speed train.
   - Information technology architecture
     Development of insight in system architectures for decision support systems.

3. NOBIS (National Research Biological In Situ Cleaning) (start 1995; managed by the Centre for Civil Engineering and Codes; CUR - Gouda)
   - Characterization of micro- and macrosystems
   - Heterogenity and processes
   - Development of concepts, strategies and processes to control biological cleaning
   - Development of techniques to apply the methods efficiently and economically
4. **MUNDIEGO** (Monitoring for durable planning and exploitation of the build-up environment)
   - Proposal for near future

5. **BONA** (Building with the future)
   - Proposal for near future

II **RESEARCH PROGRAM MINISTRY OF TRANSPORT, PUBLIC WORKS AND WATER MANAGEMENT**

The Directorate-General Rijkswaterstaat of the Ministry of Transport, Public Works and Water Management has four core areas of responsibility: Flood protection, Vehicle use, Water quantity and quality and Road safety.

The Road and Hydraulic Engineering department of Rijkswaterstaat is responsible for the R&D activities in the area of geotechnics and geo-environment needed for the core responsibilities of Rijkswaterstaat.

Nearly 80% of the research work is contracted out to research institutes (mainly Delft Geotechnics), universities and consulting firms. The R&D related to flood protection is guided by the Technical Advisory Committee on Water Defences.

The turnover of R&D in the area of geotechnics and geo-environment is about 4.5 million DFI.

Overview of the main research projects:

- **Actual stress and strength conditions:**
  Monitoring condition existing dike constructions and design extension existing road and dike constructions. In situ measurements of stress, stiffness and strength.

- **Soil improvement techniques:**
  Extension existing construction, reduction deformations, construction time and stability problems, innovation soil improvement techniques, design guideline and decision support system related to the effectivity of the improvement, both technical and cost (total of construction and maintenance).

- **Construction support system:**
  Monitoring systems for stability and deformations, management support system; update predictions etc.

- **Long term behaviour:**
  Modelling and validation creep behaviour; prediction long term behaviour related to maintenance problems. Analysis long term stability of dike constructions.

- **Tunnelling in very soft soil:**
  To gain knowledge and insight in risk of boring tunnels in very soft soil.
Probability in geotechnical engineering:
Improvement of probabilistic design methods, to gain the knowledge of the variability of the subsoil, the reliability of soil parameters versus construction and design method.

Peat and organic soils:
Modelling and validation of the behaviour of embankments on peat and organic soils, the preparation of a design guideline for embankments on peat and organic soils.

Groundwater flow in river dikes:
Design method for stability of slopes loaded by overtopping, determination design parameters sand-carrying wells, time effects on the stability introduced by unsteady groundwater flow, watertight screens (flow cut-offs).

3. RESEARCH AT CUR

As an independent, non-profit making organization the CUR occupies itself with the development, acquisition and transfer of knowledge and experience in the broad field of civil engineering; it is characteristic that the programming and implementation of these activities are directed by the parties who require knowledge.

1. Information technology
Computer science and information technology are an outstanding tool for supporting an efficient, competitive and high quality building process.
As an example research is carried out for the standardisation of exchange of data concerning the reinforcement elements in concrete structures.
In close cooperation with the government and branch organisations CUR contributes to the policy development for information technology.

2. Material technology
Material technology is related to the chemical, physical and mechanical properties, including environment aspects. The field covers a very broad range of subjects. For example, research is conducted into (long-term) behaviour of concrete under a range of conditions to sound-absorbing concrete as a road surfacing material, fibre reinforced concrete, concrete repair work, concrete protection, durability of masonry, (geo) synthetics, recycling of demolition waste and industrial residual materials.

3. Concrete structures
CUR has paid close attention to the development of knowledge and an understanding of concrete and concrete constructions since its foundation; conceptual models have been prepared to describe the behaviour of concrete structures. Emphasis lies upon the integration of material technology, construction mechanics and information technology.
Other important fields of research into concrete are for example special concrete mixtures, high-performance concrete, fire resistance of and fatigue. A comprehensive package with quality guidelines for building with concrete has been issued. These are important to all disciplines which are involved in the (concrete) building process.

4. Concrete-steel structures
In close co-operation with organizations of the steel industry research is being done into steel and concrete composite structures. Knowledge is made suitable for practical use via the compilation of calculation guidelines and design manuals for practice.

5. Hydraulics and geotechnics
In an increasing number of projects of knowledge in soil, road and hydraulic engineering is presented. English language manuals are prepared to utilize this knowledge worldwide. Hydraulics, hydrology, interaction between water and sediment and soil mechanics are important fields of interest.

The development of a method for the management and maintenance of dikes is the basis for a practical guideline for water/dike authorities. Implementation, management and maintenance of Nature-friendly river banks is an important field for attention.

Geosynthetics are more often used in environmental and civil engineering. Research is being carried out for improving design rules and test methods.

There are a number of current developments in geo and environmental engineering. Horizontal and vertical barriers of granular material in the subsoil are a subject for study.

6. Dredging research
By fundamental research into the physical processes in dredging, such as sand-water mixing processes, sludge dredging, rock dredging, water extraction and dumping of sand for underwater dams. Important topics in these research programmes are the environmental effects of dredging.

7. Above and below ground level
More use of underground space is becoming a necessity. The findings of the CUR committee on building deeper into the ground were published in a report. Among others this report was responsible for the increase in attention in this type of building.
The character of the "higher" high building requires a specific approach. Subjects for research are not only purely technical but also include aspects pertaining to the hindrance for adjacent areas, working conditions, indoor climate, safety and suchlike. Experts from various disciplines are working together to implement a specific and integral approach.

CUR also follows this integral approach in the field of the underground transport infra-structure. This involves research into technical, economical, procedural, judicial and environmental aspects in a joint context.

8. Masonry
Masonry has a long tradition in the Netherlands; the building material is widely used for a long time. There has however been little fundamental research into masonry constructions. Work is being done on complementing the lack of knowledge in this field in order to better utilize the properties of masonry. The programme for research into the behaviour of masonry constructions covers numerical simulations and specific material and construction trials and the durability of brickwork.

9. Environment
The building, management, exploitation and demolition of the built-up environment exert effects on the general environment, and not only negatively. Building has been making an appreciable contribution to solving environmental questions for years, and therefore to the reduction of environmental pollution. The results of research into useful uses of building and demolition waste and industrial residual materials have accelerated the acceptance of these materials.

Contributions are being made to the support of government legislation. For example: The Building Decree, The Building Materials Decree, radiation risks, environmental measures for building, re-use of materials, environmental design of manure basins and.

10. Nature and building
Society requires knowledge about building with respect to Nature. Nature building is a discipline which is on the rise.

The accent here lies on civil engineering; projects in which, in addition to the civil engineering contribution, Nature itself does part of the work. The set-up of criteria for the ecological design of river banks, land reclamation, creation of tidal areas, dynamic coastal management and "living" river management, require new knowledge and experience.
IV RESEARCH AT DELFT GEOTECHNICS

Delft Geotechnics is a non-profit foundation. It receives yearly a so-called mission subsidy, which is devoted to the following research and development subjects.

1. **Geocentrifuge**
   Validation of numerical methods; pile group effects; penetration of piles and probes in layered soils;

2. **Isolation and in-situ cleaning of contaminated areas**
   Development of monitoring systems; development of isolation techniques; development of the "reactor barrel" concept;

3. **Numerical models for underground construction**
   Private financing participation in national research program underground construction; development of 3D numerical models; development of large strain models;

4. **Geodynamics**
   Development of methods to measure and interpret vibrations; improvement of pile driving predictions;

5. **Geotechnical, geo-environmental and non-destructive measuring techniques**;
   Implementation of geo-environmental probes; development of 3D georadar and 3D geo-electric system; cofinancing Research Program Underground Construction;

6. **GIS**
   Development of GIS-system for geo-environmental application;

7. **Participation in European projects**
   Participation in MAST, Climat & Environment, Brite Euram, Sprint;

8. **Fundamental research**
   Ph-D studies; try out of promising new concepts

9. **Automatization and quality control**
   Participation in (inter)national communication network activities.

Gouda, 20 June 1995
FJ/JdR 190695 5132
Dear Mr. Bengt Rydell,

Thank you very much for your kind invitation for the R&D Seminar next November in Sweden. As you know Mr. Jonker is ill at this moment and probably he is not recovered in time for the seminar. So we suggest that Prof. F.B.J. Barends will fill up his place. Prof. Barends is head of the research branch of Delft Geotechnics and professor at Delft University.

For the preparation of this letter I have made a little survey among the Dutch organisations active in the field of geotechnical R&D:
- Delft University,
- Delft Geotechnics,
- Ministry of Transport and Public Works,
- Public Works Rotterdam,
- Technical Advisory Committee on Water Defences,
- CUR (an organisation for joint research and guidelines in the field of civil engineering).

I have restricted mine information to two categories of R&D projects. The first categories are projects which have already some form of international cooperation or this cooperation will start in the near future. The second category are projects with a positive attitude for international cooperation, which means that the projects have already a financial framework in the Netherlands and the project engineers are able to participate in an international cooperation. During the seminar we can give an overview of all the Dutch geotechnical research activities.

Projects with international cooperation:

More dimensional creep behaviour of soft soils.

For the Dutch geotechnical engineers creep is related to maintenance and high excess pore pressures for an unexpected long time period. The latter has our special attention because it is an important phenomena for the quantification of the safety conditions of our existing levees. Both problems, maintenance and the excess pore pressures, asking for an understanding of the creep mechanism and a design tool for the prediction of the effectivity of remedial measures or construction alternatives.

Three main items can be recognized in the research project on creep:
1. constitutive modelling, including numerical implementation,
2. element tests, such as biaxial and triaxial tests, and
3. validation.

As first step the Adachi-Oka model is implemented in a finite element code, because it is a simple extension of the well know Cam-Cay model. Despite its simplicity a robust implementation takes time. Parallel to this we are searching for an alternative formulation, which must be a balance between a correct description of the creep behaviour and the suitability for numerical implementation. Probably a part of this research will be done together with the University of Stuttgart and the University of Manchester (Prof. Molenaar).

The literature presents only oedometer and undrained triaxial tests on creep. The latter are restricted to relaxation and strain rate tests. The literature data are not sufficient for the formulation of an practical acceptable constitutive model, because in reality the stress and strain paths are different. So additional biaxial tests are planned at the
Technical University of Delft. Also the research programme proposed by Prof. Molenkamp has our attention. Furthermore there is an interchange of information with the University of Karlsruhe. They are involved in a creep research concerning the behaviour of historic buildings. Validation is the third item of the research. In the past the validation was restricted to measurements at construction sites and information from literature. The experience with measurements at construction sites was negative, because they hadn't the first priority during construction despite the positive intentions and agreements. Data at crucial points and time were missing, damage to measuring equipment and changes in the construction scheme were not reported in time. At this moment we use the data from the Vasa test field in Finland for validation. A next step in our validation program will be a full scale test. We are like to do such test in an international cooperation. For example in the SEBC test side in Bothkennar UK.

Geomechanics of peat (incl. classification).

As result of the International Peat Workshop in Delft last June, we are preparing a research and development program on the geomechanics of peat. This program will be made in cooperation with Warsaw University (prof. Wolski). We hope that this program can be incorporated in the framework of the Technical Committee TC 15 "Peat" of the ISSMFE.

This project contains the following items:

1. Analysis of factors determining the safe performance of embankments (for roads, dykes or others) on peaty soils during construction and maintenance.
2. Elaboration of field and laboratory testing procedures for the determination of geotechnical parameters of peaty soils.
3. Elaboration of classification chart of organic soils based on field investigations.
5. Elaboration of calculation methods for safety evaluation of embankments on peaty soils (engineering approach and developed methods)
6. Development of a constitutive model for finite element applications (this point will be done in cooperation with Cambridge University, prof. Molenkamp)

Guideline: "Determination soil parameters and actual conditions in soft soil"

This project can be divided in the following phases:

1. State of the art and selection of test methods,
2. Improvement of the equipment,
3. Field tests and the set up of a database,
4. Producing guideline,

The first phase of this project is ready. For the set up of the database we will cooperate with Building Research Establishment (UK). BRE has already a database containing data from several test sites in Europe (United Kingdom, Norway, Italy and Sweden).

Projects with an attitude for international cooperation:

Tunnelling in soft soil.

In the Netherlands we meet an increase interest in underground locations of the infrastructure, which asks for the assessment and development of tunnelling technology:

- obstacle detection methods to scan the pathway of a tunnel,
- productivity of excavation,
- the effect of tunnelling on existing structures at the surface,
- vibrations,
- safety and risk analyses,
- etc.
Embankment material.

- reduction of the use of natural materials (sand, gravel etc.)
- application of waste material.

Probabilistic design.

Required sample density, related to soil variability and design (construction type and required safety).

Design guidelines and standards.

- soil improvement techniques,
- calculation methods, especially finite element programmes. Important item is quality guarantee (certification?),
- determination design parameters.

Furthermore I will underline the importance of the discussion items technology transfer and quality assurance.

I am aware that the information in this letter is limited, but I think that it is sufficient for a first step in the strengthening of our international R&D network. We must realise however that there are already a great number of international committees, active in different frameworks, such as ESMFE, ISSMFE and the European Community. Next november we must discuss the possibilities of an effectively use of these existing frameworks.

A last point that I will arise for the discussion, is how we can motivated the European Industry in participation in and founding of R&D in the field of geotechnics.

I am looking forward to meet you next november.

With kindest regards

Ruud Termaat
Head Geotechnical Department.
National Report

R&D activities

Norway
Norway

References (not included):

Forskning og utvikling i Statens Vegvesen. Plan 1993
Statens Vegvesen, Vegdirektoratet. Forskning og Utvikling nr 1
Stål- og betongelementer i løsmassetunneler. Støttekonstruksjoner i armert jord
Statens Vegvesen, Vegdirektoratet. Publikation nr 69
Dear Mr. Rydell,

SEMINAR ON SOIL MECHANICS AND FOUNDATION ENGINEERING R&D FOR ROADS AND BRIDGES

Referring to your invitation to participate in the seminar mentioned above, I may offer the following information regarding R&D activities within the Public Roads Administration.

The Norwegian Public Roads Administration annually publishes plans for R&D activities in each specific year. R&D plans for 1993 are listed in the enclosed copy of R&D report no 1 in the R&D series. In later years funds for R&D activities have been of the order of 1% of the total budget in the Public Road Sector with an increase in 1993.

For the Norwegian Road Research Laboratory, R&D plans for 1993 are also given in the enclosed tables, pages 1 to 10.

Related to your questionnaire the following supplementary information may be given.

FIELD AND LABORATORY INVESTIGATIONS

Total Sounding

In cooperation with the Norwegian Geotechnical Institute a drilling method has been developed for site investigations ensuring penetration of the drillrod for all conditions. The method is based on the rotary-pressure sounding method but with a rock drillbit and a special flushing valve. Ordinary rotary-pressure sounding procedures are followed until further penetration is prevented by hard soil layers, boulders or rock. Drilling may then be continued by flushing, increasing the speed of rotation and/or using ordinary percussion drilling. When the obstacles are penetrated, ordinary rotary-pressure sounding may be resumed. At present total sounding drilling procedures are being standardised based on the field performance of alternative procedures.
CAD - Geoplot

With automatic recording systems available for most drilling methods, an interactive CAD-sytem has been developed for presenting geotechnical data on drawings. The system was first developed based on the CAD system Technovision. A new version based on AutoCad is now implemented allowing borehole data to be merged with topographic data as well as road alignment data to present geotechnical data on road plans. The system can handle both digitised data, analog data and data entered manually from the keyboard in any combination and is fully integrated with existing road planning programmes. If required rockhead contour maps may be generated automatically. Digitised data from GEOPLOT may also be used as direct input to stability analysis programmes, but the latter two features are not available yet.

DEEP AND SHALLOW FOUNDATIONS

Tension Leg Anchoring

In connection with the new concept of submerged floating bridge tubes, one particular project for the crossing of Høgsfjorden in the County of Rogaland has been analysed. The object here is to cross a 1350 m wide fjord with a submerged floating road tunnel. Four different design concepts have been developed, one employing the tension leg system known from drillrig platforms in the oil industry. Various methods for installing tension leg systems providing the necessary anchoring forces are considered. The maximum water depth at the site in question is 150 m and the soil consists of sand and silty materials with stones and possible large boulders. The soil layer has a thickness of up to 200 m below the seabed.

Load Distribution in Pile Groups

Full scale monitoring of forces on a bridge abutment has shown that stresses due to moment loads in the piles may be more critical than stresses due to axial loads. Possible further changes with time in the load/stress history of the piles are monitored.
SOIL REINFORCEMENT/IMPROVEMENT

Reinforced Earth Structures

Various applications of reinforced earth systems are tested and monitored regarding stress/strain behaviour of the reinforcement material, earth pressure and overall deformations in the structure. Possible changes in these parameters are monitored on a long term basis. Also applications of reinforced earth systems in steep walled sound barriers and similar structures are tested.

Thin Walled Superspan Culverts

Large span corrugated steel culverts and thin walled concrete culverts have been built for road projects and also used for avalanche protection purposes. In this connection a simplified design method has been proposed for designing such thin walled structures. Several structures are monitored on a long term basis regarding stress/strain in the structure, earth pressure on and overall deformation of the structure in order to detect possible changes with time. So far conditions have remained relatively stable after construction was completed.

Superlight Filling Materials

In order to overcome bearing capacity problems on soft soils, blocks of Expanded Polystyrene (EPS) have been used in Norway for more than 20 years as a superlight filling material in road construction projects. Various new applications have been implemented over this period of time, including using 5 m high EPS fills as foundations for the abutments of a temporary flyover bridge. The bridge has now been in service for four years and is performing well. In connection with the use of EPS in road projects, a long term monitoring programme has also been implemented. This includes measuring possible changes in the water content and compressive strength of the EPS blocks as well as long term deformations. As expected, no ill effects have been detected so far.

VIBRATIONS

Vibrations from Road and Rail Traffic

With large areas in the most densely populated regions in Norway covered with soft marine clay deposits, both road and rail traffic generate vibrations that may be disturbing to roadside residents. The present international standards on recommended vibrations levels
in houses indicate levels that are easily exceeded under the soil conditions prevailing in large parts of Norway. In order to shed light on this particular situation, the Norwegian Building Research Institute and The Norwegian Geotechnical Institute have since 1991 been pursuing a common project financed by the Norwegian State Railways, the Public Roads Administration and the municipal tramline company in the City of Oslo.

Findings so far have been summarised in four reports, including proposals for recommended vibration levels. A distinction is here made between a common recommended permissible level where resulting vibrations in nearby buildings are perceptible but not disturbing when large vehicles or heavy trains pass by. In cases where this level is exceeded and where necessary costs to reduce the level is prohibitive, a maximum level which must not be exceeded, is recommended. Vibrations may then easily be perceived when large vehicles and heavy trains pass by, but will not be experienced as disturbing to most people.

Later steps in the project is to establish a common database where existing and future vibration data are stored. The structure of such a database is already specified and it is hoped that other countries will contribute to the database as well. Also methods for evaluating possible vibration levels at different distances from the source under various soil conditions will be considered on an empirical basis. With such a tool the cost of implementing various permissible vibration levels may be analysed. Establishing a Norwegian standard on recommended permissible vibration levels in houses will be part of the project.

USE OF GEOTECHNICAL KNOWLEDGE

Within the Public Roads Administration Road Laboratory activities are organised with a central Road Research Laboratory and local laboratories in each County Roads Office. R&D results are disseminated within the organisation through reports, internal courses, site visits etc. On the international level information is channeled through Publications from the Road Research Laboratory and through the periodical Nordic Road & Transport Research. Papers are also presented at international conferences (ICSMFE, ECSMFE, PIARC Congresses and Technical Committees, OECD RTR etc.) International conferences are also organised in Norway, e.g. Strait Crossings, Bearing Capacity of Roads and airfields, Low Cost Road Tunnels etc.

Also new knowledge is implemented in the handbook system the Public Roads Administration has established for Standard Specifications and Codes of Practice. In the Soil Mechanics field the following handbooks are available:
Standard Specifications

- Handbook 014 Laboratoriedersøkelser (Laboratory tests)
- Handbook 015 Feltundersøkelser (Site investigations)
- Handbook 018 Vegbygging (Road Construction)
- Handbook 021 Tunnelbygging (Tunnel Construction)
- Handbook 100 Bruprosjektering 03 Støttemurer
  (Bridge Design 03 Retaining walls)
- Handbook 100 Bruprosjektering 04 Landkar
  (Bridge Design 04 Abutments)

Codes of Practice

- Handbook 016 Geoteknikk i vegbygging (Soil Mechanics in Road Construction)
- Handbook 084 Støyekermer (Noise Barriers)
- Handbook 154 Geoteknisk opptegning (Geotechnical Drawings)
- Handbook 165 Sikring av vegskråninger (Protecting Road Slopes)
- Handbook 166 Vegrekkerverk (Guardrails)

To be published soon:

- Handbook Oppbygging av vegfyllinger (Construction of Road Embankments)
- Handbook Grunnforsterkning (Subsoil Improvements)

I hope the information given above may shed some light on our R&D activities. If requested an oral presentation may be given at the seminar in November for one or more of the projects mentioned.

Norwegian Road Research Laboratory
Yours sincerely

[Signature]
Tor Erik Fjeldlund

2 enclosures

Copy without enclosures: Kjell Karlsrud, NGI
R&D AT NGI RELATED TO ROADS AND BRIDGES

NGI carries out a wide range of R&D projects. The following gives a brief summary of those that are directly or indirectly related to road and bridge foundation engineering. Although we were not certain whether or not tunneling was to be included, we have done so in this list.

ONGOING PROJECTS

1. Geotechnical Design
   A. Comparison between reliability of various methods for prediction of axial pile capacity in clay soils. (Part of a larger project on risk assessment that has been going on for several years).
   B. Further developments of the suction caisson foundation concepts. Field and model studies are carried out in sands and clays and new theoretical models developed. Concepts may be applicable to bridge foundations in relatively deep water.
   C. Development of new generation FEM program for continuum soil-structure interaction. Version 1, primarily applicable to deep supported excavations will be completed this year.
   D. Preparation of piezocone interpretation manual.
   E. Optimized rock blasting technology. Carefully controlled fragmentation tests are carried out using different borehole patterns and explosives.

2. Foundation Engineering
   A. Long-term monitoring on reinforced earth test fill. The test fill was built 6 years ago and has been monitored since then. Pull-out tests on reinforcement are now being planned.
   B. Verification testing of deep cement/lime mixed columns. Sampling and laboratory testing will be carried out in a test area with different cement/lime mixes.
   C. Microtunneling methods and their applicability. This project includes a literature survey of available techniques and assessment of their
applicability in different soil conditions. Furthermore, one monitors the performance of various projects in Norway.

D. Tunneling in soils. The main scope of this project is to find alternatives to use of closed face tunneling machines, which become very costly for relatively short road- and railway tunnels. Alternative concepts considered include various forms and combinations of ground improvement (ground freezing, jet-grouting), micro-tunneling and open or partly closed jacking shields. Methods for assessing stability and designing tunnel linings are also being reviewed.

E. Updating of NGI's Q-classification system for rocks and design of tunnel support. The updating concentrates on including stress level and new support methods.

F. State of art documentation of Norwegian sprayed concrete technology in relation to rock tunneling.

3. Environmental Geotechnics

A. Contamination in saturated and unsaturated zones - Transport mechanisms and geochemical processes. This is a combinations of assessment of numerical models and column testing in the laboratory.

B. Electrokinetic remediation of contaminated soils. Preliminary laboratory tests are carried out on clay samples artificially contaminated with lead, and some in-situ samples taken from a selected site.

C. In-situ bio-remediation of ground contaminated with oil spills. A full scale field test is ongoing in a sand/gravel aquifer that has been contaminated by oil spill.

PLANNED PROJECTS

Many of the ongoing projects listed above will continue in 1994. Only one new project is so far planned:

* Direct use of in-situ tests for design of piles and shallow foundations. This is an EEC project with several participants. The main scope is to make direct correlations between actual case records (test piles, footings etc) and various in-situ tests, including PCPT, Pressuremeter and Dilatometer.
National Report

R&D activities

Sweden
SUMMARY OF RESEARCH AND DEVELOPMENT PROJECTS
ROADS AND BRIDGES

Rainer Massarsch and Håkan Stille
Department of Soil and Rock Mechanics
Royal Institute of Technology, Stockholm, Sweden

GENERAL
Research and development projects at the Royal Institute of Technology, related to infrastructure projects can be divided into the following areas:

- Geotechnical problems
- Foundation engineering applications
- Soil dynamics problems
- Subsurface construction

In general, research activities are oriented towards the solution of practical engineering problems and are therefore carried out in close cooperation with the construction industry in Sweden and abroad.

The research philosophy at the department is based on the "active design concept". Geotechnical design is considered to be part of the entire construction process, i.e. the planning phase, involvement in the construction process and monitoring of the performance during the operating phase.

The initial geotechnical design is flexible and accommodates future changes with regard to the construction process. This in turn requires active participation of the geotechnical engineer during project execution. Field monitoring techniques are thus an important part of active design. Site supervision of the various stages of the construction process are necessary to check the validity of the initial design assumptions.

Research facilities consist of routine geotechnical and geodynamic laboratory equipment, field investigation equipment, field monitoring and testing instruments, and analytical tools such as different computer codes (finite element, finite difference and boundary element programmes).

In the following, a summary of research and development projects, relevant for roads and bridges will be presented:

1. GEOTECHNICAL DESIGN

- Interpretation of cone penetration test (CPT) in granular soils

The objective of the project is to develop correlations between CPT results (tip resistance and sleeve friction) and relative density of granular soils. The measured cone resistance is normalised with respect to a reference
stress, similar to the Standard Penetration Test. A simple correction method for overburden pressure has been developed which is based on extensive calibration chamber tests.

A method for the assessment of the deformation characteristics of granular soils (tangent modulus) is being developed which makes it possible to estimate settlements of compacted soil deposits. The project is carried out in Sweden but research co-operation exists with German foundation contractors.

The basic concept has been published and can be presented orally.

- Development of an acoustic cone penetrometer for determination of soil type

A method for determination of soil type by acoustic measurements has been developed at the department and was tested successfully in the laboratory and in the field. The acoustic cone penetration test makes it possible to identify thin layers of silt and sand in clay soils.

The research project is presently continued as a doctoral thesis at the Geotechnical Department of Gent State University, Belgium. A formal research co-operation is planned between the Royal Institute of Technology and Gent State University.

2. FOUNDATION ENGINEERING

- Vibratory pile driving

The objective of the project is to investigate the behaviour of vibrated piles and to compare the driving resistance of vibrated and impact-driven piles. Different types of piles (open- and closed steel tubes, concrete piles etc.) are installed in different soil deposits. The significance of various parameters such as vibrator frequency, vibration amplitude, moment of eccentricity, static vs. dynamic mass for the driving resistance are investigated.

A theoretical model for soil-pile interaction is being developed, based on cone penetration tests (point resistance and sleeve friction) which can be used to assess the required vibrator characteristics for pile installation.

Preliminary results from vibratory driving tests in Germany and Sweden are available and can be presented.

The project is carried out in cooperation with German foundation contractors and manufacturers of vibrator equipment.

- Soil compaction using vibratory probes (MRC compaction)

A method for densification of granular soils by vibratory probes, using the resonance effect of soil deposits, was developed at the department in the past. The method is being used successfully in different parts of the world. A vibrator with variable-frequency control and a specially designed compaction probe with low impedance (dynamic stiffness) are used to achieve optimal transfer of vibration energy to the soil. The compaction process is monitored and controlled by a computerised process control system which assists the machine operator in the execution of the compaction process. The compaction effect can be determined based on measurement of a variety of parameters, such as resonance frequency, vibration velocity of the ground, penetration speed of the compaction probe etc.
Present research activities are aimed at further improving the various elements of the MRC system, such as vibrator performance, automatisation of the compaction process (electronic control of the compaction process), interpretation of various parameters which are recorded during MRC compaction etc.

- **Lime and lime/cement columns**
  Lime and lime/cement columns are frequently used in the Nordic countries for the stabilisation of cohesive soils. The objective of the present project is to determine the strength and deformation characteristics of stabilised soil columns by dynamic testing methods. Specially developed vibration sensors are installed in the stabilised columns and the travel time of a shear waves is measured between different depth intervals. Correlations are being established between travel time, column stiffness and column strength.

  An important aspect of the project is to develop design methods for the interaction of groups of columns with the surrounding soil. The non-linear stress-strain behaviour of soil-column interaction shall be taken into account. The objective is to propose practically applicable design methods for column-stabilised foundations.

The project is yet in an early stage and only preliminary results can be presented.

3. ENVIRONMENTAL GEOTECHNICS

- **Vibrations caused by man-made activities** (soil-structure interaction)

  The objective of the project is to investigate vibrations and their effects on structures, caused by construction activities, such as soil compaction, pile driving and blasting. The importance of geodynamic parameters such as soil modulus (wave velocity), material damping, ground water level (soil saturation), soil layering effects etc. are studied.

  The importance of various dynamic parameters of the vibration source, such as vibration velocity amplitude and frequency is studied to assess the susceptibility of different structure types to damage from vibrations. One important parameter which normally is not considered for damage assessment is the relationship between wave length and building geometry.

  Results of extensive investigations have been reported and could be presented orally.

- **Vibration propagation in soils caused by traffic**

  The dynamic characteristics of different vibration sources from traffic are studied. The path of vibrations from the dynamic source (source-structure interaction), propagation in the ground (wave propagation) and their effects on structures (soil-structure interaction) are studied.

  Methods for vibration isolation of structures are studied such as lime column walls, gas cushion screens etc.

4. USE OF GEOTECHNICAL KNOWLEDGE

  The results of major R&D projects are documented primarily by doctoral or licentiate theses. Results of specific
scientific projects are published in the international geotechnical literature, such as conferences, scientific journals and report series.

Researchers participate in international conferences and symposia. Scientific co-operation exists with several foreign universities, however not in a formalised way.
SOIL MECHANICS AND FOUNDATION ENGINEERING R&D FOR ROADS AND BRIDGES

R&D-PROJECTS SWEDISH NATIONAL RAIL ADMINISTRATION (BANVERKET)

1. INTERACTION TRACK-BRIDGE-FOUNDATION-SOIL

The purpose of the project is to determine the distribution of forces from train traffic into track, ballast, bridge, foundation structures and soil and the interaction among the different elements in the interactive model track-bridge-soil.

Results from measurements on railway bridges and theoretical studies have been analysed. The latest development in field studies reported by the European Rail Research Institute has been studied. A measuring programme has been performed and measurements are planned to be carried out on three existing bridges and one new constructed bridge. Measurements are planned to be carried out on the track, ballast, abutments and foundation structures.

Today the measuring programme has started with measurements on existing bridges. Development of measuring instruments for earth pressure and pile forces and studies of theoretical calculation models (numerical methods) which will consider interaction among track, ballast layer, bridge structure and the surrounding soil is going to be carried out.

Alexander Smekal, Banverket Head Office, is responsible for the project’s coordination.
According to plans the project will be finished 1997.

2. STABILITY OF RAILWAY EMBANKMENT UNDER DYNAMIC TRAIN LOAD.

The project is aimed to analyse the distribution of loads in the railway embankment and subgrade at the time the train is passing and to study stability calculation models which shall consider the dynamic train forces.

Study of available literature has been done and measurements from different projects and existing calculation models have been analysed.

Measurements of deformations, earth pressure, pore pressure and vibrations, with different train loads, on a railway embankment resting on soft clay are carried out at present time.
The shape of wheel flats and dampers and the influence of speed and weight and the duration of the train passing are studied as well.
The theoretical models analysing the dynamic response of the embankment due to the train passing including dynamic laboratory tests are going to be set up the next year. The results from the preliminary measurements will be compared with the theoretical calculations. The results shall serve as a base for the next measuring serials, verifying obtained results.

Peter Zackrisson, Banverket Head Office, is responsible for the project's coordination. According to plans the project is going to be carried out up to 1996.

3. DESIGN OF CATENARY SUPPORT FOOTING

The aim of this project is to set up a few different methods for design of catenary support footings concerning ultimate limit state and serviceability limit state design. The effect of the shape and rawness of the footing and geotechnical conditions is going to be studied as well.

Studies of literature dealing with catenary support footings subjected to the horizontal load have been carried out. Interpretation of different design models including comparison with previously done loading tests is carried out at present time. Analysis of load assumptions and deformation limits and planning of in-situ loading test are also carried out.

As the last step the calculation models shall be verified and corrected with consideration of in-situ load tests.

Responsible for the project: Alexander Smekal. The project is carried out in cooperation with Swedish Geotechnical Institute.

4. VIBRATION PROGNOSTICATION

Vibration disturbance from train traffic on neighbouring buildings has turned out to be a problem when railways are constructed, especially in soft clays. The aim of this project is to find suitable models for prognostication of vibration distribution from the train traffic.

Results from vibration measurements, that have been carried out at Swedish National Rail Administration during the last 10 years, are analysed at present time. The purpose is, for different train types, to find the relations among the vibration level, soil condition, water content, soil depth, subsoil topography and the relations among vibration level, soil condition, structural design, foundation methods and damping effect due to different vibration absorption measures.
The result of the project is going to be basis for further development and a mathematical model considering the train-load and speed, the shape of the railway embankment and the soil and ground water conditions.

Project is going to be carried out up to 1997.

5. EUROCODE 7

The aim of this project is to study the consequence of transition to the EC 7 Code. The consequences regarding the proposed partial factors of safety for spread and pile foundations and stability of embankments compared with Swedish practice today are going to be studied especially. The project is performed in cooperation among Swedish public authorities.

Project is going to be carried out up to 1997.

6. FUTURE PROJECTS

Research projects for the future will deal with interaction track-ballast-soil, especially concerning increased axle loads, faster train traffic and higher requirements for comfort standards. Research projects concerning for example design models for soil reinforcement, lime/cement columns, statistical methods for foundation structural design, development and verification of models dealing with settlement calculation for cohesive and non-cohesive soils, in cooperation with other Swedish research institutes, is also of great importance.

Item for oral presentation:

- Interaction track-bridge-foundation-soil

- Stability of railway embankment under dynamic train load
Seminar on Soil Mechanics and Foundation Engineering Research and Development for Roads and Bridges

Information of the Research and Development Work at the Swedish Geotechnical Institute

CONTENT

Geotechnical Design

Ongoing Projects

G1. Settlement calculations for road embankments on fine-grained soils
G2. Modelling of variations in the groundwater situation around excavations and road cuts
G3. Optimum use of road construction material
G4. Calculation of settlements for shallow foundations
G5. Friction piles in silt and sand. Increase in bearing capacity with time
G6. Statistically calculated partial coefficients adapted to Nybyggnadsreglerna (the Swedish Building Code)
G7. Calculation of settlement for embankment on soft soil. Advanced numerical methods
G8. Field investigations in fine- and medium-grained soils
G9. Geology and geotechnics in preliminary road design

Planned Activities

G101. Development of design methods for shallow foundations on slopes
G102. Bearing strength and stability of subgrades/terraces
G103. Settlements in fine-grained soil and measures taken to eliminate these
G104. Introduction of partial safety methods in geotechnical design for roads and railways
G105. Characteristics for soil parameters concerning strength and settlement
G106. Properties of silt and mixed-grained soils
G107. Development of laboratory methods for determination of deformation and strength properties of firm and stabilised soils
G108. Behaviour of natural and/or improved slopes. Advanced numerical methods
G109. Lime-cement columns properties. A basis for probability based design methods
Foundation Engineering and Soil Improvement

Ongoing Projects

F1. Follow-up system for settlement in road embankment on fine-grained soil
F2. Settlement follow-up of Main road No. 50 in Sweden
F3. Settlement follow-up of European highway E6, section between the villages St. Höga and Ödsmål in Sweden
F4. Foamed concrete in ground, road and railroad construction
F5. Soil improvement with the Lime-column method. Improvement of design methods for settlement for embankment
F6. Prediction and performance of reinforced soil as retaining structures
F7. Follow-up of settlements of bridges
F8. The use of cement and cement-lime in deep stabilisation of soft soils
F9. Lime-cement columns, textbook on design, performance and inspection

Planned Activities

F101. Consequences of foundation works for buildings
F102. Development of regulations for material testing and design of soil reinforcement
F103. Ground water in excavations
F104. Application inventory of new, international methods for soil improvement
F105. Settlement follow-up of lime/cement column improvement for railway embankments

Environmental Geotechnics

Ongoing Projects

E1. Water protection in areas along roads
E2. Environmental impact on soil in areas along roads
E3. Geotechnical aspects of environmental risk assessment studies. Preliminary study
E4. Treatment of highway stormwater
E5. Durability of geosynthetics
E6. Groundwater modelling as a tool for risk assessment and remediation of contaminated groundwater
E7. Utilisation of coal ash for compensated foundations on European highway E4

Planned Activities

E101. Increased utilisation of waste products in infrastructure construction
E102. Testing and usage of field instruments for environmental investigations

Use of Geotechnical Knowledge
Geotechnical Design

Ongoing Projects

G1. Settlement calculations for road embankments on fine-grained soils

Settlement calculation including creep effects is now a standard procedure in road design in Sweden. The objective of the present project is to create a user-friendly calculation programme for personal computers which is to be made generally available.

The programme has been written with consideration to requests and demands that have arisen since this type of calculation came into use. The programme is Windows-based in order to make it as simple and familiar for the user as possible. Comprehensive help and monitoring functions are included in order to avoid user errors. Presentation of the results can be selected to meet the requirements of both road designers and researchers.

Together with the programme, a manual has been developed for "settlement calculations for road embankments on fine-grained soils", describing the investigations and tests required, parameter selection, the calculation process and the limitations on the methods used in the present programme.

Results of calculations with the programme are compared to results from long-term follow-ups of settlements in roads and test embankments.

Time schedule and documentation

The project is being carried out in co-operation between SGI and the Swedish National Road Administration. It started in 1992 and will be finished in 1994. The results will be presented in terms of the finished programme and the manual.

G2. Modelling of variations in the groundwater situation around excavations and road cuts

Variations in the groundwater situation during and after excavation works influence a number of factors such as slope stability, erosion, drainage conditions and bearing capacity of the bottom of the excavation.

The aim of the present project is to study the suitability of different numerical models for simulating the effects of changes in groundwater conditions. These models will then be used for a parameter study of soil characteristics, dimensions of the excavation and time aspects.

A further aim is to investigate various instruments used to measure pore water pressure in both saturated and unsaturated zones.
Documentation

Preliminary results from the project have been presented in a technical paper. Complete results will be published later in a final report.

G3. Optimum use of road construction material

Short supply and restrictions on the use of natural glaciofluvial gravel material, combined with increased quality requirements on road materials, are factors that make gravel resource planning an important task today and more so in the future. Materials of varying quality and volumes occur in different places. A important factor is that moraine material can be used as road material in many cases instead of the glaciofluvial material. The use of a methodology that includes mapping, classification and recommendations for optimum use of glaciofluvial, moraine and bedrock as road materials could save large amounts of money in road construction and maintenance.

The objective of the project is to develop a methodology for resource planning of natural materials for road construction. The project is divided into three phases.

In the first phase, different methods for mapping natural gravel resources were described and a tentative method of mapping and classification was suggested.

In the second phase, a description of requirements on road materials, especially moraine, was made. A crushing test was included.

In the third phase, a test is being planned to demonstrate the use of the suggested methodology.

The project is being carried out at the request of the Geotechnical Department of the Swedish National Road Administration.

G4. Calculation of settlements for shallow foundations

In Sweden, shallow foundations have long been designed by using a simplified bearing capacity formula and a set of permissible bearing factors based on results from weight sounding tests.

To avoid excessive settlements, the maximum permissible bearing pressures have normally been limited.

When introducing limit state design in Sweden, improvements have to be made to the design procedures, including both a new bearing capacity formula for ultimate limit state and new settlement calculation methods for the serviceability state.

The aim of this project has been to find relevant settlement calculation methods for shallow
foundations on cohesionless soils and overconsolidated clays, evaluate reliability by comparing calculated and measured settlements and finally propose guidelines for settlement calculations.

**Time schedule and documentation**

The project started in 1992 and the results will be presented this autumn. Three different methods have been chosen and guidelines have been presented to the Swedish National Road Administration.

**G5. Friction piles in silt and sand. Increase in bearing capacity with time**

The increase in bearing capacity of friction piles with time after driving has been studied. The research project includes a literature survey and a detailed analysis of a large number of piling projects in Sweden. The project concerns mainly precast concrete piles driven in silt and sand.

In many cases, a substantial increase in bearing capacity with time is documented. This increase in bearing capacity is sometimes in the order of 30-150 per cent over the time period 1 to 90 days after driving. The magnitude of the increase depends on the soil type.

The project also includes the results of a small number of tests using stress-wave measurements at dynamic penetration testing. The purpose of these measurements is to find a soil investigation method for predicting the increase in bearing capacity of piles.

**Time schedule**

The project started in 1985 and was completed in 1993.

**Documentation**

The project will be published as Report 91 by the Swedish Commission on Pile Research.

**G6. Statistically calculated partial coefficients adapted to *Nybyggnadsreglerna* (the Swedish Building Code)**

The aim of the project is to develop better rules and methods for determining partial coefficients with an improved connection to the probability of exceeding ultimate limit state and serviceability limit state. The project is concerned with two areas where experience has revealed a high degree of uncertainty, both in the calculation models and the geotechnical parameters.

Uncertainty in the geotechnical parameters. In the statistical soil model, the translation of measured data must give the correct statistical information in order to make the correct
choice of the partial coefficient. In the project, a simple model will be developed which can be used in "daily work".

Uncertainty in the model. The uncertainty in the calculation models is best determined by expert opinion. This will be done by conducting an international inquiry.

**Time Schedule**

The project has been divided into two parts. The results from the first part are now available and can be presented. The second part is still at an early stage and no results are yet available. The project will be completed in December 1994.

**G7. Calculation of settlement for embankment on soft soil. Advanced numerical methods**

SGI Report No 29 describes a method of calculating settlements for embankments on soft soil (clay). In the model, the time process of settlements is calculated as a one-dimensional consolidation process which allows the properties to be updated while the settlement process is going on. In nature, the assumption of one-dimensional consolidation is normally not fulfilled. Normally, it is necessary to be able to model the process as a two- or three-dimensional process.

The objective of this project is to test the ability of advanced numerical methods to assess the problem of settlements in embankments on soft soil. SGI has worked jointly with LCPC, Paris, France on the possibility of using CESAR-LCPC program. The program is a general program for continuum mechanics and contains a model for clay - the so-called Melanie model. The aim is to test the model in the long-term test fields described in Report 29. The importance of two-dimensional behaviour as compared to the one-dimensional model can thereby be evaluated.

**Time schedule**

The project is being carried out with financial support from SGI and BFR. The project started on 1 July 1992 and will end on 30 June 1995.

**G8. Field investigations in fine- and medium-grained soils**

Research and development of field investigation methods, equipments and techniques is a continuous project at SGI.

Recently, a large investigation on the use of CPT tests and dilatometer tests in soft and fine-grained soils has been completed. The results are now being implemented in manuals for purposes such as investigations for road construction and slope stability. New Swedish "recommended standards" for the CPT test and the field vane test were produced by the Swedish Geotechnical Society in 1992 and a new "recommended standard" for the dilatometer test is being prepared.
Present research in this field mainly concerns the evaluation of proposed techniques for simplifying and rationalising the CPT test and gathering of results from settlement observations in full-scale cases where the "new" methods have been employed in the field investigations.

Research in the near future will include evaluation of field investigation methods in silts, layered soils and mixed-grained soils.

Documentation

The results of this research have mainly been presented in SGI publications in the form of a Report, Information sheet or Varia.

G9. Geology and geotechnics in preliminary road design

Road construction and maintenance costs are highly dependent on geological and geotechnical conditions. The possibilities of influencing these costs are high in the early phases of road planning when unsuitable ground can be avoided. In this project, a model in the form of flowcharts is proposed.

on how to consider geology and geotechnics in preliminary road design.

The model stresses the close, regular cooperation between the road designer and the geotechnical engineer. The model is developed for both route location plan (when one corridor is to be selected from two or more corridors) and preliminary road design (when the location of the road within the selected corridor is to be decided). Two types of geotechnical map are suggested. A "Geo signal" map for the route location plan uses the colours red (worst), yellow and green (best) to indicate the geotechnical suitability of the terrain. A "Geo advice map" for the preliminary road design shows different geotechnical classes and their geotechnical implications as to both cuts and embankments. Advice is also given on the suitability of the soil material in road construction.

The project is being carried out at the request of the Geotechnical Department of the Swedish National Road Administration.
Planned Activities

G101. Development of design methods for shallow foundations on slopes

Foundations for bridge abutments are frequently built at an elevation higher than the piers for the same bridge. If the foundation is situated close to the slope, both the bearing capacity in ultimate limit state and the settlements in serviceability limit state will be influenced by the neighbouring slope.

The aim of this project is to improve existing design methods for shallow foundations in ultimate limit state on slopes of cohesionless material and to develop design methods for serviceability limit state.

Time schedule and documentation

The project started last year with a literature survey and comparative calculations using different design methods for ultimate bearing capacities. These results have been summarised in a preliminary report not yet published. Some additional calculations using FEM-analysis were performed in order to study the deformations below the foundation.

It is planned to continue the project with model tests at the SGI in 1994 and large scale tests at the SGI test field in 1995.

G102. Bearing strength and stability of subgrades/terraces

The new technical description, BYA 92, issued by the Swedish National Road Administration, lays down requirements on the ultimate bearing resistance of the terrace layer in road construction. To determine the bearing resistance, one of the permitted methods is the static load bearing test.

The aim of the project is to determine whether there are any other methods for controlling the ultimate bearing resistance of the terrace layer which are faster, simpler and more reproducible than the static load bearing test. One of the methods to be studied is a light type of "falling weight equipment".

In cases where the requirements cannot be fulfilled, methods of improving the terrace layer are required. The methods to be studied are:

- mixing different stabilising agents with soils
- excavation and backfill with gravel
- deep drainage of the terrace

Time schedule:
Project finish: About two years after start.

Documentation:
Report in the "Vågledning" series (published jointly by the Swedish National Road Administration and the SGI).

G103. Settlements in fine-grained soil and measures taken to eliminate these

Since 1985, the theory published in SGI Report No. 29 by Rolf Larsson, "Consolidation of Soft Soils", has been used in a number of projects at the Swedish Geotechnical Institute. As the Institute is currently working jointly with the Swedish National Road Administration on a new computer program which takes Rolf Larsson's theory into account, we will need to follow up results from constructed embankments on soft soils also in the future. The results of measured settlements will be compared with the calculated values.

In the "new" theory, the consolidation settlements will increase compared with the results from the Terzaghi theory, and this indicates that more areas will need soil improvement. During the design period, it is therefore of great importance to use different measures for soil improvement techniques to reduce or eliminate these settlements and to fulfil the requirements pointed out in the new technical description, BYA 92, from the Swedish National Road Administration. It is also important for construction to be as optimal as possible. The techniques for eliminating or reducing the difference settlement in the transition between two different improvement methods or between areas where an improvement method has been used and areas without improvement must be developed.

Time schedule:
Project start: January 1994
Project finish: June 1996

Documentation:
Papers or articles to the Nordic Geotechnical Conference and a report in the "Vågledning" (published jointly by the Swedish National Road Administration and the SGI).

G104. Introduction of partial safety methods in geotechnical design for roads and railways

In Sweden, partial safety factors have until now only been used in geotechnical design for buildings. Using total safety factors, the risk of failure has traditionally been higher for embankments and slopes than for spread foundations. This indicates that the consequences of introducing partial safety factors together with limit state design for embankments and slopes must be examined for different constructions. The consequence analysis will provide better knowledge of the present risk level for normal road and railway constructions, as well as a basis for taking action in the future. This examination will also generate input to the "National Application Document" (NAD) for roads and railways.
Time schedule:
Project start: November 1993
Project finish: June 1996

Documentation:
Project report to the clients and a draft for a Swedish road and railway NAD.

G105. Characteristics for soil parameters concerning strength and settlement

When making a geotechnical design using partial safety factors, the risk of failure, for instance, is very strongly influenced by the coefficient of variation for the shear strength. This project, which is a joint project with other fields of activity at the Institute, will determine soil properties in the field and the laboratory. The results will then be used for analysis of the mean value, standard deviation and coefficient of variation for the properties tested, for different methods and for different types of soil.

The results of this project will provide better support in selecting the partial factor $\gamma_m$ depending on the soil and the method used for the investigation.

Time schedule:
Project finish: June 1996.

Documentation:
Paper/article in Swedish and foreign magazine and also to the European Geotechnical Conference. Final project report.
F2. Settlement follow-up of Main road No. 50 in Sweden.
Final project report.
The project has been described in an article to the European conference in Florence 1991.

G106. Properties of silts and mixed-grained soils

The aim of this project is to increase knowledge of the properties of silts and mixed-grained soils and to find suitable methods of field and laboratory investigations for these soils.

In the project, a review will be made of existing knowledge of properties of the particular soils and suitable investigation and design methods. Field tests, sampling and laboratory tests and large-scale load tests in the field will be performed at several test sites. The various investigation, interpretation and design methods will be evaluated with consideration to the results of the load tests and the results of follow-ups of full-scale field cases that are available.
Time schedule

The project is planned to start in late 1993 and end in 1996.

G107. Development of laboratory methods for determination of deformation and strength properties of firm and stabilised soils

To predict the effect of stabilisation of a soil, it is important to perform laboratory tests on prepared samples. These types of tests are always carried out in connection with the design of deep stabilisation with lime-cement columns. The method of mixing in laboratory will be standardised.

Samples of lime-cement columns cast in situ can be taken to the laboratory for analysis. For this purpose, plans have been made to develop triaxial equipment for samples with a diameter of 150 mm. This equipment can also be used to investigate properties of samples from residues and other kinds of stabilised material.

G108. Behaviour of natural and/or improved slopes. Advanced numerical methods

Evaluation of the status of a natural slope or a slope that ought to be improved is normally performed with the help of conventional calculation of the safety factor. These calculations involve many assumptions in the models and it is difficult to provide good input for the evaluation of the behaviour of the slope.

The objective of the project is to test the ability of advanced numerical methods for the design of slopes, particularly the ability to determine the influence of different types of improvement of slopes.

The importance of taking into account the influence of modelling the slope in the conventional way or in a more advanced format will be shown.

Another objective of the project is to obtain better knowledge about slope behaviour.

The project has been planned and will start in 1994.

G109. Lime-cement column properties. A basis for probability based design methods

The purpose of the project is to extract statistical data for determining the strength of lime-cement columns and to estimate uncertainties in a model for settlement calculations. The results will be used as a basis for determining partial safety factors for future design methods based on a statistical approach.
An inventory of existing knowledge and suitable methods for strength tests, statistical analysis and follow-up of constructions founded on deep stabilised soil will be carried out. Strength and compression properties of columns will be determined, mainly through laboratory tests and a statistical analysis of the results will be made. In addition, a follow-up of column reinforcement for a building or for a mat foundation will be carried out at the SGI test field. Traditional settlement calculations and more advanced calculations for determining interaction between soil, column and construction for different loads will be carried out, as well as an estimation of different strength parameters importance in the design model.

**Time schedule**


**Documentation**

Research report.
Foundation Engineering and Soil Improvement

Ongoing Projects

F1. Follow-up system for settlement in road embankment on fine grained soil

The need for follow-up of settlements carried out in a systematic way has increased with the introduction of Nybyggnadsreglerna (the Swedish building code). It should be possible to use the follow-up in contexts such as improving the calculation models.

The aim of the project is to propose a selection method and a follow-up system for settlements in road embankments on fine-grained soil. The proposal should describe the method for selecting the appropriate object, measuring methods and routines, field instrumentation, design of the presentation and the relevant documentation. Demands will be set on measuring accuracy, measuring instruments, the placing of measuring instrument and measuring points.

It should be possible to use the proposal both for "routine" follow-ups and research projects.

Time Schedule

The project is being carried out in cooperation with the Swedish National Road Administration and will be finished at the end of 1993.

F2. Settlement follow-up of Main road No. 50 in Sweden

A four-way junction just outside Askersund was rebuilt and moved about 100 metres. The soil in this area consists of 1 m peat on top of about fifteen metres of soft silty clay. The original road was improved with vertical drains and combined with a temporary load. The stratigraphy of the soil was derived by CPT test, which indicated a continuous layer of silt and sand. Settlement calculations were made, taking into account the fact that the water could be drained to these layers. According to the calculations, it should be possible to eliminate most of the settlements during a construction period of 18 months with only a preload. The settlement has been measured since 1987 and agreement with the calculated settlement is so far very good.

F3. Settlement follow-up of European highway E6, section between the villages of St. Höga and Ödsmål in Sweden

In designing this project, lightweight fill material (expanded clay pellets) has been used mainly to achieve more even settlements along and across the highway, as well as to reduce the total settlements. At one location, a section 300 metres in length, the settlements after
twenty years have been calculated to be up to 0.5 metre. Here, instrumentation has been installed to measure the total settlement across the road, the settlement at different depths and the variation in pore water pressure with depth.

The measurements were started in 1988 and so far the results are in good agreement with the calculated settlements.

**Time schedule:**
Project start: July 1988.

**Documentation:**
Paper/article in a Swedish magazine and journals such as "Ground Engineering". Report in the "Vägledning" series (published jointly by the Swedish National Road Administration and the SGI).

**F4. Foamed concrete in ground, road and railroad construction**

The purpose of the project is to study the suitability of using foamed concrete in connection with the construction of roads and railroads. Preliminary directions on different applications for foamed concrete have been proposed. The directions concern planning, performance, demands on material, design and quality control.

**Documentation**
A textbook (in Swedish) has been presented, which includes examples of applications in addition to the above directions. A careful follow-up has been made on a number of objects where foamed concrete has been used in connection with repair work on damaged roads.

The project has been run in cooperation between different state-owned and private companies, such as the Swedish Road Administration, the Swedish Geotechnical Institute, contractors (NCC) and the cement industry (Cementa).

**F5. Soil improvement with the Lime-column method. Improvement of design methods for settlement for embankment**

Calculation of settlements and their time sequence currently requires many simplifications. In this project, a theoretical approach to the problem is taken to understand the influence of different parameters on settlements.

Extensive measurements have been performed in a test field and the results have been analyzed and compared with calculation methods.

An important aspect of the project is to permit better understanding of the performance of soil stabilization with limecolumns for an embankment.
Design methods are presented and their applicability is discussed.

**Time schedule**

A final report on the project will be published in December 1993.

The calculation of the settlements and the time process of the settlements is today made with a lot of simplifications.

In this project a theoretical approach to the problem is taken to understand the influence of different parameters on the settlement.

At a test field extensive measurements have been performed and the results are analyzed and compared with calculation methods.

An important aspect of the project is to give a better understanding of the performance of a soil stabilization with lime-column for an embankment.

Design methods are presented and their applicability is discussed.

**Time schedule**

The project is going to be finally reported in December 1993.

**F6. Prediction and performance of reinforced soil as retaining structures**

The technique of using reinforced soil as a retaining structure is common in other countries but has not really been established in Sweden yet. The reasons are many. However, the technique is in many cases both economically and technically a better solution than conventional concrete walls, even for Swedish conditions. In Linköping, three different retaining walls have been built during summer 1993. SGI and a number of other bodies are involved in the project, which will provide a reference object for expanding the use of the method in Sweden.

Follow-up studies are being made of the economic results and performance and movements in the structures. Measurements of elongation are being made in the back anchorages consisting of plastic grids and steel bars respectively. A comparison will be made between measured values of forces and design values to determine the validity of present theory.

**Time schedule**

The main part of the results will be presented in December 1993, although the measurements will continue at least until December 1995.
F7. Follow-up of settlements of bridges

The need for systematic follow-up of settlements has increased with the introduction of Nybyggnadsreglerna (the Swedish building code). It should be possible to use the follow-up in contexts such as improving the calculation models.

The present knowledge of the actual settlements needs to be improved.

The aim of the project is to propose a selection method and a systems for following up settlement of bridges. The proposal should describe the method for selecting the appropriate object, measuring methods and routines, field instrumentation, design of the presentation and the relevant documentation. Demands will be set on measuring accuracy, measuring instruments, the placing of measuring instrument and measuring points.

It should be possible to use the proposal for both "routine" follow-ups and research projects.

Time Schedule

The project is being carried out in cooperation with the Swedish National Road Administration and will be finished at the end of 1993.

F8. The use of cement and cement-lime in deep stabilisation of soft soils

The aim of the project is to study the effect of various additives in different types of soft soils. The project is expected to lead to a better understanding of the behaviour of stabilised soil and to provide guidelines for the design of future deep stabilisation projects.

The stabilising agents used in the project consist of four different types of cement, four different mixes of cement and quick lime and also quick lime alone. Different amounts of additives are used. In all, ten different types of soils, ranging from peat and gyttja to different types of soft clays and clayey silt, have been stabilised in the laboratory. Testing has been performed at time intervals up to 270 days after mixing. The samples have been tested in regard to undrained and drained shear strength, compression modulus and permeability. Theoretical and experimental studies of the chemical reactions are also included in the project. Field tests have been performed at two test sites, where columns have been installed in clay and clayey gyttja. The tests comprise measurements of temperature, permeability and shear strength.

Time schedule

The project started in 1990 and is expected to be finished by the early spring of 1994.

Documentation

Results are to be presented in an SGI Report. Results from the project as well as the project
itself have been briefly described in a paper to the XIII ICSMFE 1994 in New Delhi, entitled "The use of different additives in deep stabilisation of soft soils" by Åhnberg, Holm, Holmqvist & Ljungcrantz.

F9. Lime-cement columns, textbook on design, performance and inspection

Soil stabilisation with lime and lime-cement columns is the dominating stabilisation method for cohesive soils in Sweden. Among the advantages are the possibilities for adapting the method to the distance, length and diameter of the columns. The method is also economical compared to other techniques in connection with the construction of roads and railroads.

Plans have been made to carry out a major modernisation of Sweden’s infrastructure over the coming years. Deep stabilisation with lime-cement columns is considered very suitable for the types of construction involved. The method is fairly new and many consultant engineers, contractors and administrators unfamiliar with such projects are involved. Knowledge of the advantages and disadvantages of the method is insufficient. There is also a need to modernise the design methods.

Documentation

The results of the project will be presented in a textbook (in Swedish). It is likely that the textbook will later be translated to English.
Planned Activities

F101.   Consequences of foundation works for buildings

The aim of the project is to study a number of buildings with shallow and deep foundations in ten cities in Sweden. Settlements and effects in surrounding areas will be measured, analysed and reported.

The project will provide information on the actual settlements of buildings. Methods used for design in serviceability limit stage can be calibrated. Effects on and from surrounding areas will be presented. The project will also provide guidelines on how to follow up, manage and report settlements affecting buildings.

Time schedule

The project will start in late 1993 and is planned to continue at least to mid-1995.

F 102. Development of regulations for material testing and design of soil reinforcement

Material testing of geosynthetics used as soil reinforcement is very important. Today, many manufactures use different testing methods and it is therefore very difficult to compare different products with each other and to know how they will change with time. SGI has presented a number of suggestions for testing methods to the Swedish National Road Administration and these were published in 1992. Attention has to be paid to the European work that is in progress. SGI will monitor this development.

The project "Prediction and performance of reinforced soil as a retaining structure" will provide information on the validity of current design theory. This project and other work elsewhere in the world may lead to a new design method suitable for use in Sweden.

Time schedule

The work will take place continuously over a three-year period.

F103.   Ground water in excavations

The aim of this project is to determine how ground water conditions are affected by excavations and/or road cuts in different natural soils. Various numerical analyses will be used to simulate the excavations and to forecast changes in ground water conditions. The results from numerical calculations will be compared with field measurements. The project is expected to provide valuable experience on the behaviour of the soil/water system in large excavations and its influence on stability, drainage, erosion, bearing capacity, frost
depth and frost heave.

The project will lead to a thesis for the degree of Doctor of Science at the Department of Geotechnical Engineering, Chalmers University of Technology.

Time schedule:
Project finish: June 1994.

Documentation:
Final report (thesis) in CTH or/and an SGI report. Articles in soil mechanics journals. Paper in the "Vågledning" series (published jointly by the Swedish National Road Administration and the SGI).

F104. Application inventory of new, international methods for soil improvement

New methods of soil improvement, such as soil nailing, are being suggested for use in Sweden. SGI is receptive to new methods and is willing to introduce them if they are economically and technically advantageous. Follow-up studies are being planned when the methods are introduced in Sweden, since it is important to determine how well they perform in Swedish conditions.

F105. Settlement follow-up of lime/cement column improvement for railway embankments

In autumn 1993, an embankment will be built for the "Svealandsbanan" railway. Lime/cement columns will be used to improve the ground. The soil consists of 25 m clay over friction material. The columns will be installed in three different layers (5, 10 and 15 m depth) in the same sections.

Settlements will be followed both with horizontal hose settlement gauge and with bellows-hose settlement gauge. Horizontal movements will be measured with inclinometer. Pore water pressure gauges will also be installed. A number of different methods for determining the shear strength in the columns will be used. Measurements will be made over a period of three years.

The design method used today is empirical and is based on the design of the lime columns. Design values will be compared with measured values to determine how the design method works for lime/cement columns. Since most settlements occur in the upper part of the clay layer, the columns will be installed in three different layers. The quantity of lime/cement will then be three times more in the upper five meters than in the clay between 10-15 m. De-escalation behaviour will be studied.
Time schedule

The results of the project will be presented at the end of a three-year period.
Environmental Geotechnics

Ongoing Projects

E1. Water protection in areas along roads

Highways are often situated close to water resources because geological conditions favourable for ground water supplies are often also suited for road construction. Storm water and liquid spillages from vehicle accidents are therefore a threat to water resources in many places.

The project describes Swedish laws to protect water resources and takes up questions of responsibility. Environmental influences from roads are discussed. A strategy for identifying and classifying areas in regard to vulnerability is presented. Measures to protect water resources are proposed corresponding to the vulnerability classification.

A list of functional demands for protecting water resources is proposed.

Time schedule and documentation

The project will be finished in 1993 and presented in a report.

E2. Environmental impact on soil in areas along roads

Every 5-20 years, road ditches have to be cleaned from soil and vegetation that has invaded them and decreased their hydraulic capacity. Material from ditches is dumped or used as fill in building projects. The material is likely to contain contaminants from traffic and road maintenance. An international review has shown that there are no publications on the chemical composition of the soil in road ditches or on the risks associated with the disposal of such material.

The aim of the project is to provide recommendations on how to deal with soil from road ditches. The methods include soil sampling from ditches in the vicinity of eight roads and surrounding areas, soil characterisation, analysis of total chemical composition and leach tests (batch and column). The chemical analysis includes 18 inorganic compounds, mineral oil and PAH.

The preliminary results show that the concentrations of lead, zinc, copper and mineral oil are elevated in the ditch soil. Concentrations of other heavy metals and PAH are not elevated. The concentration of metals is a function of the traffic intensity, especially in the case of zinc and copper. The concentration in the soil decreases dramatically with the distance from the road and with depth. The leaching tests will show how much of the metals can leach out and be transported to the groundwater. The project will probably show that it is justified to perform tests on the soil in the vicinity of highways with high traffic intensity before disposal of the soil.
Time Schedule and documentation

The project will finish this autumn with a report for the Swedish National Road Administration. Some of the results will also be published in the international press during 1994.


Preliminary study

Environmental risk assessments are an important basis for decisions in land use planning, as well as in the road planning process. Questions of geology, geotechnics and geohydrology have so far not been described in a satisfactory way. In a preliminary study, a checklist will be used to ensure that aspects of soil, rock and ground water are taken into account.

A small number of environmental risk assessments of roads will be studied, in particular with regard to the description of the geotechnically related consequences.

Time schedule


Documentation

The checklist will be published in the SNRA series of reports.

E4. Treatment of highway stormwater

The most commonly used method for removing surface runoff from highways in Sweden is through direct discharge to the nearest watercourse. Such a system pays little attention to the potential pollution loads generated from rainfall runoff or their possible impact on receiving waters.

The aim of this project is to identify and specify designs for treatment of highway runoff. The study is intended to help define management practices and cost-effective options, together with design and operational guidance.

As a part of the project, existing and planned treatment systems for highway stormwater in Sweden are being surveyed. A literature review is being performed to evaluate the effectiveness of existing management practices in reducing and treating highway runoff. Treatment systems commonly used in other countries will be studied, since Sweden has very little experience in this field.

Time schedule

The project started in June this year and is planned to continue for three years. The first
year will be concentrated on literature studies and inventory of treatment systems. During the second and third years, a small number of treatment systems will be studied in order to recommend the best management practices.

**Documentation/presentation of results**

As the project is at an early stage, no documentation has yet been presented. Annual reports will be presented at the end of each year and a final report presenting the recommendations will be issued at the end of 1995.

**E5. Durability of geosynthetics**

A preliminary study has been made during 1993 on the durability of geosynthetics. This work is being performed for four Swedish authorities, apart from SGI. Each type of geosynthetic for different kinds of applications has been investigated by literature study and contact with foreign institutes. Important properties of different kinds of geosynthetic for special applications have been identified. Recommendations on the type of test method for these properties have been made. There are also suggestions for further research.

SGI will follow the European work in progress and will also take part in a joint working group between CEN TC 189 and 254 concerning geomembranes.

Nordic co-operation in this field is planned, but funding of the project has not yet been finalised. Other projects on behalf of various authorities in Sweden concerning rules that affect durability will probably start during the next three years, but have not yet been specified.

**E6. Groundwater modelling as a tool for risk assessment and remediation of contaminated groundwater**

Contaminated soil and groundwater is an increasing problem throughout the world. Site investigations, chemical analysis, risk assessment and remedial actions are often complicated and expensive. The most effective way of using the collected data is to enter it in a computerised model of the area. Today, there are many different models on the market with varying capabilities.

The aim of the project is to give an introduction to practical groundwater modelling in connection with contaminated sites. Different types of models will be presented and evaluated, the modelling methodology will be reviewed and modelling of flow and contaminant transport will be performed for two investigated areas. Methods of modelling remedial actions such as pumping and treatment, drainage, slurry walls etc. will be described.

**Time schedule and documentation**

The project will be completed during spring 1994 and presented in a report for the Swedish
Building Council.

**E7. Utilisation of coal ash for compensated foundations on European highway E4**

The SGI is co-operating with several of the major municipal owners of thermal power and district heating plants. The purpose of this co-operation is to determine the chemical and physical properties of residual products, as well as the environmental impact of the utilisation of such products.

In the construction of a new section of European highway E4, close to the city of Norrköping, Sweden, a volume of 50,000 cubic metres of bottom ash has been used for embankments. The SGI has determined the properties of the coal ash and designed the embankment. In a follow-up study, settlements, bearing capacity and possible contamination of soil and ground water will be investigated. The SGI co-operates in a similar way with other industries, such as the steel industry.

**Time schedule**


**Documentation**

Research reports and design documents.
Planned Activities

E101. Increased utilisation of waste products in infrastructure construction

About 45 million metric tonnes of different materials are used every year in Sweden for road construction. Only a few per cent of this amount consists of residual products, whereas natural materials are used to cover the remaining need. The reason for the low usage of residual products is due to uncertainty regarding when and how alternative materials can be used, as well as what environmental requirements should govern such usage.

The list of residual products that can be used for road construction is extensive, even though geographical factors, such as transport distances, may influence the economy of usage. For several years, the SGI has been working to optimise the utilisation of residual products from energy production (ashes) and the metal industry (slag).

The planned work will be carried out with a wide approach through research projects determining the technical and environmental properties of the products. Furthermore, the practical use of the research results will be applied and used in demonstration projects, where great importance will be attached to follow-ups and environmental controls.

Time schedule

September 1993 - December 1995

Documentation

Research reports and conference papers

E102. Testing and usage of field instruments for environmental investigations

Problems with areas containing contaminated soils and sediments have increased strongly during the last decade. In areas with such problems, soil, gas or ground water may be contaminated by organic or inorganic substances. From experience it is known that the substances are heterogeneously spread to a very high extent in the area concerned. Investigating such areas in a cost-effective may causes considerable difficulties.

The study is divided into a preliminary study and a main study (1994-1996). The objective is to examine and test a field instrument for environmental investigations, alternatively to develop or improve existing instruments. The purpose of the preliminary study is to define the specifications for a "Swedish" field instrument, to conduct a market inventory and to formulate proposals for the main study. Specifications will have to be defined for physical stability, chemical detection ability, compatibility with geotechnical standard equipment, manageability, economy etc.
A market inventory regarding existing instruments will be carried out, as well as a study of different gauges for environment-related parameters. The emphasis will be placed on fibre optic gauges, FOCS, since these have proved to be most useful, especially in analyses of organic substances.

**Time schedule**

Project start: 1 January, 1994  
Project finish: 31 December, 1994

**Documentation**

Presentation of results: Research report, seminar
Use of Geotechnical Knowledge

To ensure that the necessary geotechnical knowledge is available for planning and construction, research needs to be completed with continuous information activities. The SGI has a natural role as the country’s information centre by disseminating knowledge to various groups in the community.

The Institute’s Literature Service, which furnishes library services, retrieval and evaluation of literature, performs continuous surveillance of foreign literature and exchange in liaison with some 300 foreign organizations. Special attention is given to the EC and the conditions that will influence the Swedish construction market in the future.

The Institute is represented in various committees and working groups, both in Sweden and internationally. The SGI has joined with the industry in intensifying work on European rules and environmental geotechnics. The aim of the Institute’s participation is to make geotechnical skills more widely known, accepted and standardized.

The SGI organizes courses and conferences for municipal authorities and government departments, as well as consultants and contractors, the aim being to increase awareness for geotechnics and geotechnical applications.

The Institute communicates its research results and practical experience through reports, manuals and other publications. In addition, research results are presented through participation in Swedish and international conferences, as well as contributions to these.

The publications in the series SGI Report are listed in the enclosed publications list.
Serien "Rapport" ersätter våra tidigare serier: "Proceedings" (27 nr), "Särtryck och preliminära rapporter" (60 nr) samt "Meddelanden" (10 nr).

The series "Report" supersedes the previous series: "Proceedings" (27 Nos), "Reprints and Preliminary Reports" (60 Nos) and "Meddelanden" (10 Nos).

RAPPORTE/REPORT

No.  År  Pris kr  exkl moms

1  Grundvattensänkning till följd av tunnelsprängning. 1977  20:-
P. Ahlberg & T. Lundgren
(Ground water lowering as a consequence of tunnel blasting. In Swedish with an English summary)

2  Påhängskrafter på långa betongpålar. 1977  20:-
L. Bjerin
(Negative skin friction on long precast piles driven in clay. In Swedish with an English summary)

3  Methods for reducing undrained shear strength of soft clay. 1977  20:-
K.V. Helenelund

4  Basic behaviour of Scandinavian soft clays. 1977  20:-
R. Larsson

5  Snabba ödometerförsök. / 1978  20:-
R. Karlsson & L. Viberg
(Rapid oedometer tests. In Swedish with an English summary)

6  Skredriskbedömningar med hjälp av elektromagnetisk fältstyrkemätning - provning av ny metod. 1978  20:-
J. Inganas
(Landslide risk estimation by means of electromagnetic field strength measurement - test of a new method. In Swedish with an English summary)
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<td>7</td>
<td>Förebyggande av sättningar i ledningsgravar - en förstudie.</td>
<td>U. Bergdahl, R. Fogelström, K.-G. Larsson &amp; P. Liljekvist</td>
<td>1979</td>
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<td></td>
<td>How to limit the settlements in fills above pipe line systems - a preliminary survey. (In Swedish with an English summary)</td>
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<td>B. Carlsson</td>
<td>1979</td>
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<td>Horisontalarmerade fyllningar på löst jord.</td>
<td>J. Belfrage</td>
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<td>(Horizontal reinforcement in embankments on soft soils. (In Swedish with an English summary)</td>
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<td>Tuveskredet 1977-11-30</td>
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<td>R. Larsson</td>
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<td>Long term consolidation beneath the test fills at Väsby, Sweden.</td>
<td>Y.C.E. Chang</td>
<td>1981</td>
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<td>Bentonittätning mot lakvatten.</td>
<td>T. Lundgren, L. Karlqvist &amp; U. Qvarfort</td>
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<td>(Beatonite sealants in the pollution control of sanitary landfills. (In Swedish with an English summary)</td>
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<td>Kartering och klassificering av lerområdets stabilitetsförutsättningar.</td>
<td>L. Viberg</td>
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<td>(Mapping and classification of the stability conditions within clay areas. (In Swedish with an English summary)</td>
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<td>17</td>
<td>Symposium on slopes on soft clays, Linköping, March 8-10, 1982</td>
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<td>18</td>
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<td>19</td>
<td>Släntstabilitetsberäkningar i lera. Skall man använda totalspänningsanalys, effektivspänningsanalys eller kombinerad analys? R. Larsson</td>
<td>1983</td>
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<td>20</td>
<td>Portryckssvariationer i leror inom Göteborgs-trakten. J. Berntson</td>
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<td>21</td>
<td>Tekniska egenskaper hos restprodukter från kolförbränning - en laboratoriestudie. B. Möller &amp; G. Nilson</td>
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<td>Geoteknisk terrängklassificering för fysisk planering. L. Viberg</td>
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26 | 1984 | 20:-
Large diameter bored piles in non-cohesive soils. Determination of the bearing capacity and settlement from results of static penetration tests (CPT) and standard penetration test (SPT).
K. Gwisdala

27 | 1985 | 95:-
Bestämning av organisk halt, karbonathalt och sulfidhalt i jord.
R. Larsson, G. Nilson & J. Rogbeck
(Determination of organic content, carbonate content and sulphur content in soils. In Swedish with an English summary)

27E | 1987 | 95:-
Determination of organic content, carbonate content and sulphur content in soils.
R. Larsson, G. Nilson, & J. Rogbeck

28 | 1986 | 73:-
Deponering av avfall från kol- och torveldning.
T. Lundgren & P. Elander

28E | 1987 | 73:-
Environmental control in disposal and utilization of combustion residues.
T. Lundgren & T. Elander

29 | 1986 | 125:-
Consolidation of soft soils.
R. Larsson

30 | 1987 | 110:-
Kalkpelare med gips som tillsatsmedel.
G. Holm, R. Tränk & A. Ekström
Användning av kalk-flygaska vid djupstabilisering av jord.
G. Holm & H. Ahnberg

31 | 1986 | 100:-
Kalkpelarmetoden. Resultat av 10 års forskning och praktisk användning samt framtida utveckling.
H. Ahnberg & G. Holm

32 | 1988 | 150:-
Two stage-constructed embankments on organic soils. Field and laboratory investigations - Instrumentation - Prediction and observation of behaviour.
W. Wolski, A. Szymanski, J. Mirecki, Z. Lechowicz, R. Larsson, J. Hartén, K. Garbulewski & U. Bergdahl
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<td>33</td>
<td>Dynamic and static behaviour of driven piles. Diss. Nguyen Truong Tien</td>
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<td>34</td>
<td>Kalksten som fylningsmaterial. J. Hartlén &amp; B. Åkesson (Limestone as fill material)</td>
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<td>35</td>
<td>Thermal properties of soils and rocks. Diss J. Sundberg</td>
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<td>Full-scale failure test on a stage-constructed test fill on organic soil W. Wolski, A. Szymanski, Z. Lechowicz R. Larsson, J. Hartlén, &amp; U. Bergdahl</td>
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<td>37</td>
<td>Pore pressure measurement - Reliability of different systems M. Tremblay</td>
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<td>38</td>
<td>Behaviour of organic clay and gyttja R. Larsson</td>
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<td>125:-</td>
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<td>Shear moduli in Scandinavian clays. R. Larsson &amp; M. Mulabdic'</td>
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<td>Piezocone tests in clay. R. Larsson &amp; M. Mulabdic'</td>
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<td>43</td>
<td>Footings with settlement-reducing piles in non-cohesive soil Phung Duc Long</td>
<td>1993</td>
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</table>
SWEDISH NATIONAL ROAD ADMINISTRATION

Geotechnical Design

Ongoing R&D-projects

- research areas

- name of research projects including a short description of objectives and time schedules

Analysis of consistency NAD-EC7. The project includes:

1) Calculation of actual probability for failure for constructions being built today

2) Discussion about the achieved probability - is it OK?

3) Suggestions about how to determine material properties to reach desired probability for failure

The project will be finished during 1996

Planned R&D-activities

- problems in infrastructure related to soil, rock and ground water

  Varying safety level for different geotechnical constructions due to e.g. uncertainty while choosing characteristic properties for soil

- planned R&D-projects in a 1-3 years period

  Methods for determining properties for special materials, e.g. rest products
- co-operation with other organizations

Swedish Geotechnical Institute

Swedish Road and Transport Research Institute

Technical Universities
Foundation Engineering

Planned R&D-activities

- problems in infrastructure related to soil, rock and ground water

  Uncertainty in calculation models

  (Antagen beräkningsmodell stämmer ej med verkliga uppmätta resultat. Ofta kan det vara svårt att avgöra om det felaktiga resultatet beror på fel i modellen eller fel antagna material egenskaper)

- planned R&D-projects in a 1-3 years period

  Development of analytical methods for calculation of settlements for embankments and footings. Development of an empirical model for determination of size of settlements and increase by time in a compacted earth fill.

  Development of a model for design of soil improvement with lime/cement columns including methods for determining characteristic properties.

  Development of methods for design of retaining constructions including loads from earth pressure.

  Requirements for overall stability for existing roads including a method for locating roads with low stability

- co-operation with other organizations

  Swedish Geotechnical Institute

  Swedish Road and Transport Research Institute

  Technical Universities
Environmental Geotechnics

Ongoing R&D-projects

- research areas

- name of research projects including a short description of objectives and time schedules

  Model for describing effects on water from chemical and physical aspects. The project will be finished during 1994

  Method for inventory, description of properties and planning of resources for rock, natural gravel and course morain. The project will be finished during 1994

  Technics for protection against environmental effects. The project will be finished during 1995

- preliminary research results

Planned R&D-activities

- problems in infrastructure related to soil, rock and ground water

  Effects on the environment and spread of pollutions via "road water" to water and soil have to be considered more systematically while planning, building and maintaining roads.

- planned R&D-projects in a 1-3 years period

  Method for handling old road materials
- co-operation with other organizations

Swedish Geotechnical Institute

Technical Universities

Sveriges geotekniska institut

Universities
National Report

R&D activities

United Kingdom
Dear Mr Rydell,

SEMINAR ON SOIL MECHANICS AND FOUNDATION ENGINEERING R&D FOR ROADS AND BRIDGES.

In response to your request for a description of the geotechnical R&D programme in the UK, I am pleased to enclose the following information:

1. Annex 1 which is my reply to your questionnaire;

2. Annex 2 which is a list of current research projects of the UK Department of Transport (DoT);

3. Annex 3 which is a list of current research projects of the UK Department of the Environment (DoE);

4. Annex 4 which is a list of current research projects of the Construction Industry Research and Information Association (CIRIA), a body part-funded by subscriptions from industry;

5. Annex 5 which is a list of the current research programme funded by the Science and Engineering Research Council (SERC).

You may find it helpful to know a little about how geotechnical R&D is funded in the UK. The UK Government is by far the greatest source of funding; funding is provided as follows:

(a) Via the DoT, for highway and bridge application; most of the work is carried out at the Transport Research Laboratory, or managed by them at Universities.

Contd...
(b) Via the DoE, for application in the construction of buildings and other essentially non-transport matters such as embankment dams, ground contamination etc. Most of the DoE funding goes to the Building Research Establishment (BRE) and to CIRIA, who receives 50% support for projects with the remainder coming from the Construction Industry.

(c) Via the SERC, who allocate R&D funds to University and other Higher Education Institution research groups.

While only the work funded by the DoT is directly aimed at road and bridge application, much of the work supported by the other bodies applies indirectly to transportation engineering: for example, BRE investigates in situ ground property testing which has relevance to highway as well as building projects. Therefore, you will find in the Questionnaire answers that contain contributions from several different sources.

I hope the information I have provided will be sufficient for your purposes, but please contact me if you require further help.

I look forward to meeting you at the Seminar in November.

Your sincerely,

[Signature]

R Driscoll
Head, Geotechnics Division
ANNEX 1: QUESTIONNAIRE

The answers given below have taken information from the attached R&D programmes and categorised them under the headings given; this process has not always been easy, since some projects do not fall neatly into the categories given.

The author has some knowledge of the detail of about 50% of these extensive programmes; it is, therefore, not always possible to know how much EC involvement exists, or where joint execution is involved. Where EC involvement is known to exist, the item has been marked thus: X_EC; where projects are known to be jointly executed by more than one of the 4 UK organisations listed, the project has been marked thus: X_GT.

All the projects listed in the Annexes are either ongoing or scheduled to begin within one year. Generally, the University-based projects (S) last for three years, while the other projects range in duration between 2 and 5 years; some long-term, monitoring projects have continued for 15 years, but this is rare.

The author is able to make an oral presentation only for those projects funded by the DoE; he can make brief statements about some parts of the other programmes.

The projects listed in Annexes 1 - 4 have been assigned to the Plenary Sessions as follows:

Session 1 - Geotechnical Design

1 (a) - Advanced numerical methods: T.17; T.20; T.28; S.6; S.7; S.14; S.15; S.16; S.26; S.36; S.38; S.41; S.42.

1 (b) - Statistical methods: E.7; S.1.

1 (c) - Field and laboratory investigations: T.7; T.13; T.39; T.47; E.1_EC; E.5; E.8_EC; E.9_EC; E.10; E.15; S.2; S.3; S.8; S.9; S.10; S.11; S.12; S.18; S.19; S.20; S.21; S.22; S.23; S.25; S.27; S.28; S.29; S.30; S.31; S.32; S.33; S.34; S.37; S.39; S.40; S.43; S.44; S.45; S.46.
Session 2 - Foundation Engineering

2 (a) - Deep and shallow foundations: T.5; T.6; T.8; T.10; T.11; T.12; T.14; T.15; T.16; T.21; E.2; E.4; E.6EC; E.12; E.13; S.4; S.24.

2 (b) - Soil reinforcement/improvement: T.19; T.25; T.26; T.29; T.30; T.31; T.44; T.45; T.48; T.50; T.59; E.10; E.14;

Session 3 - Environmental Geotechnics

3 (a) - Vibrations:

3 (b) - Slope stability: T.41; T.42; T.51; T.52; T.55; T.57; T.60; E.17; S.17.

3 (c) - Ground water - contamination and protection: T.37; E.3JT; S.5; S.13.

Session 5 - Use of Geotechnical Knowledge

5 (a) - Dissemination of R&D results: C.1 - C.17*

5 (b) - Technology transfer: C.1 - C.17*; S.35.

* Note: CIRIA exists primarily to disseminate and transfer geotechnical knowledge from Research Institutes and Universities into industrial practice).
ANNEX 2. DEPARTMENT OF TRANSPORT (DoT) - Reference T.

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<td>Predicting settlements caused by tunnelling.</td>
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<td>Review of design methods for tunnel linings.</td>
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<td>T.5</td>
<td>Behaviour of piled foundations under lateral loading.</td>
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<td>T.7</td>
<td>Centrifuge studies of long-term pressures on multi-propped retaining structures.</td>
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<td>Economics of alternative construction methods for accommodating soil induced lateral loading on piled foundations.</td>
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<td>T.11</td>
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<td>T.14</td>
<td>Behaviour of diaphragm walls during construction.</td>
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<td>Behaviour of bored pile retaining structures during construction.</td>
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<td>Limit state design of sub-structures and foundations.</td>
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<td>Assessment of soil-structure interaction using plasticity and limit state analysis methods.</td>
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<td>Expert systems for design of retaining structures.</td>
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<td>T.35</td>
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<td>T.36</td>
<td>Dry stone retaining walls.</td>
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T.37 Site investigation techniques, environmental factors and investigation of derelict, marginal and polluted land.
T.38 Effects of construction activity on adjacent property.
T.39 Cost benefit analysis of site investigation.
T.40 Site investigation specification.
T.41 Study of the mechanism of highway slope failures.
T.42 The design of slopes for highway cuttings and embankments.
T.43 Design of buried pipes under highways.
T.44 Re-appraisal of earthwork compaction.
T.45 Monitoring of earthwork compaction.
T.46 Compaction of trenches and other confined spaces.
T.47 Classification of chalk fill.
T.48 Lime stabilisation.
T.49 Use of re-cycled and waste materials and industrial by-products.
T.50 Ground improvement techniques.
T.51 Use of polystyrene in embankments.
T.52 Use of vegetation for slope stability.
T.53 Structural implications of pipeline backfill materials.
T.54 Earthworks drainage.
T.55 Slope repair and maintenance.
T.56 Reviews of methods of sub-surface drainage.
T.57 Slope and basal reinforcement with geotextiles.
T.58 Methods of widening existing motorways.
T.59 Development of CEN test procedures for geotextiles.
T.60 Reinforced slopes and embankments.
T.61 Trenchless construction of pipelines.
T.62 Highway crossings by hazardous pipelines.
ANNEX 3. DEPARTMENT OF THE ENVIRONMENT (DoE) - Reference E

E.1 Assessment of Eurocode 7 and inputs to associated soil testing standards.
E.2 Instability of foundations on shrinking and swelling clay soils.
E.3 Performance of geotechnical barriers to contaminant flow.
E.4 Development of methods of assessing collapse compression in fills.
E.5 Calibration of in-situ methods with laboratory methods of ground property determination.
E.6 Development of empirical procedures to relate small-scale site tests to observed foundation behaviour.
E.7 Database of subsidence damage to buildings.
E.8 Measurements of the Liquid Limit - differences in European practice.
E.9 Factors affecting triaxial soil testing - differences in European practice.
E.10 Use of dynamic probing, geophysics and other in-situ tests to assess effectiveness of ground improvement techniques.
E.11 Design and performance of soakaways for small developments.
E.12 Monitoring the performance of foundations and the effects of excavation on foundations of buildings.
E.13 The long-term settlements of filled ground - monitoring and assessing the consequences for buildings.
E.14 Improving the quality assurance procedures for the placement of engineered fills and post-placement fill treatment.
E.15 Engineering properties of soils with organic content.
E.16 Effects of collapse of abandoned mine-workings and appropriate treatments.
E.17 Review of incidence of clay slope creep damage to buildings.
E.18 Assessment of the threats to buildings from unexpected ground instability.
E.19 Field and laboratory studies of the behaviour of embankment dams.
ANNEX 4. CONSTRUCTION INDUSTRY RESEARCH AND INFORMATION ASSOCIATION (CIRIA) - Reference C.

C.1 Groundwater levels in the Thames Gravels.
C.2 Rising groundwater levels in Birmingham, and the engineering implications.
C.3 Methane and associated hazards to construction: (i) Protection of new and existing development; (ii) Nature, origins, occurrence; (iii) Detection, measurement; monitoring; (iv) Procedures for investigation.
C.4 Remedial treatment of contaminated land.
C.5 Ground movements from tunnelling.
C.6 SPT.
C.7 Review of ground improvement techniques.
C.8 The use of geotextiles in ground engineering: soil reinforcement.
C.9 Role of integrity and other non-destructive testing in the evaluation of piled foundations.
C.10 The use of grouting techniques for ground improvement.
C.11 Engineering properties of major UK soils and rocks: Chalk.
C.13 Monitoring, maintenance and rehabilitation of boreholes.
C.14 Sale and transfer of land which may be affected by contamination.
C.15 Management and disposal of dredgings.
C.16 Geotechnical sampling and testing.
C.17 Improvement of ground using deep compaction techniques.
ANNEX 5.

SCIENCE AND ENGINEERING RESEARCH COUNCIL (SERC) - Reference S.

S.1 Establishment of a National pavement performance database and relationships.
S.2 Model-based measurement of road defects using structured lighting and stereoscopic images.
S.3 A study of small-strain stiffness of Cambridge Gault clay.
S.4 Compensation grouting to prevent subsidence due to tunnelling.
S.5 Study of pollutant spillage and electro-kinetic soil remediation.
S.6 Numerical modelling of pollutant spillage and cleanup operations.
S.7 Dynamic (boundary elements) analysis of non-destructive impact tests on layered soils and pavements.
S.8 True triaxial tests to investigate stress-induced anisotropy of sand.
S.9 The Bothkennar pile downdrag study: monitoring and analysis.
S.10 Soil microstructure and cementation in the Bothkennar clay.
S.11 The direct measurement of soil moisture suction in the laboratory and the field.
S.12 Development of site investigation and rock mass classification methodology for rock engineering.
S.13 Electro-kinetic decontamination of soils.
S.14 Finite element analysis of stochastic soils.
S.15 Finite element analysis of slope liquefaction.
S.16 Three-dimensional finite element analysis of reinforced and nailed soil.
S.17 Assessment of acoustic emission techniques for monitoring the stability of slopes.
S.18 Permanent strain response of granular materials subjected to cyclic loading in the hollow cylinder apparatus.
S.19 Field and laboratory testing of concrete jacking pipes.
S.20 Analysis and design of reinforced concrete jacking pipes.
S.21 The influence of sedimentation history on the behaviour of re-sedimented Bothkennar clay.
S.22 The effect of electrical and chemical potentials on the state of soil slurries.
S.23 Validation of a critical state model for unsaturated soils.
S.24 The design of earth berms for the temporary support of embedded retaining walls.
S.25 Analysis of pavements and their structural maintenance requirements based on non-destructive test techniques.
S.26 Numerical modelling of rising groundwater.
S.27 Location of solution features in chalk.
S.28 Compressibility of weak rock masses.
S.29 Field studies of a cantilever retaining wall singly propped at formation level.
S.30 Automatic image analysis of soil microfabric.
S.31 Micromechanics of granular material under 3D states of stress.
S.32 The application of dynamic geotechnical modelling to design.
S.33 Application of soil mechanics principles to the behaviour of soft rocks.
S.34 Movement of foundations on clays due to rising groundwater.
S.35 A knowledge-based system for the interpretation of site investigation information.
S.36 Numerical modelling of pavements (incorporating reinforcing elements) subjected to dynamic wheel load.
S.37 The application of dynamic geotechnical modelling to design.
S.38 Analysis of regularly inhomogeneous soils - application to stone column reinforced foundations.
S.39 Classification of clay soils.
S.40 Ground deformations induced by transient loading.
S.41 Development of efficient algorithms for 3D analysis of non-linear soils.
S.42 Three-dimensional finite element modelling.
S.43 In situ measurement of permeability.
S.44 The behaviour of unsaturated soils.
S.45 Changes in lateral earth pressure and ground movements associated with diaphragm walls in clay.
S.46 Surface-wave geophysics for the prediction of foundation settlements.
2. GEOTECHNICAL DESIGN

2.1 Technical papers

In situ characterisation of deformation behaviour of soils and pavements
by special analysis of surface waves
Prof Dr Ir W F van Impe, Ir Wim Haegeman

2.2 Literature and references (not included)

Settlement analysis of compacted granular fill
K Rainer Massarsch

Consolidation of soft soils. SGI Report No 29
Rolf Larsson

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IN SITU CHARACTERISATION OF DEFORMATION BEHAVIOUR OF
SOILS AND PAVEMENTS BY SPECTRAL ANALYSIS OF SURFACE WAVES

Prof. Dr ir W. F. VAN IMPE 1 - ir Wim Haegeman 2

ABSTRACT

The Spectral-Analysis-of-Surface-Waves (SASW) method is an in-situ, seismic method for determining the shear velocity (or shear modulus and young modulus) profile of soil and pavement sites. Field measurements are performed for the purpose of measuring surface wave dispersion at a site. This dispersion is expressed in terms of a dispersion curve which is a plot of propagation velocity versus wavelength. Once the field dispersion curve has been determined, it is used to calculate the stiffness profile at the site using an inversion algorithm. Inversion allows detailed profiles of shear wave velocity to be determined at sites with very simple to very complex stiffness profiles. Descriptions of the test configuration, field procedure and inversion methodology are presented in the following paragraphs.

Introduction

Methods employing wave propagation principles in determining elastic properties are either intrusive or non-intrusive. Intrusive methods require penetration or borehole drilling and determine compression ($V_p$) and shear ($V_s$) wave velocities directly. Tests included in this category are the uphole, downhole, crosshole, bottomhole, and seismic cone penetration. Non-intrusive methods determine body wave velocity profiles from the ground surface. These include surface reflection, surface refraction, steady state vibration and the Spectral Analysis of Surface Waves. Surface reflection and refraction are generally more suited to geophysical exploration work and do not generally provide accurate profiles of seismic veloci-

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ties for geotechnical engineering purposes. To varying extents, the last two non-intrusive methods use surface wave dispersion to indirectly determine compression and shear wave velocities. The advantages of these measurements are that they are made at known levels of stress and strain (<10^{-3}%)) and they involve essentially no soil disturbance. The SASW involves no boreholes since the equipment is placed on the ground surface, is extremely rapid and uses the latest technology in waveform analysis.

Theoretical background

Surface waves used in the SASW method are the vertically polarised Rayleigh waves. These are seismic waves that travel along the exposed surface of any solid system. These waves have particle motion that decreases with depth into the system. The depth of wave motion is determined by the wavelength (or frequency) of the wave. Low frequency, hence long wavelength, waves extend deeper into the system than high frequency, hence short wavelength, waves. This property is illustrated in Fig. 1.

Rayleigh waves with short wavelengths propagate through the surface layer; their velocity will only be determined by the properties of that layer. On the other hand, longer wavelength Rayleigh waves propagate through the top several layers, and their velocities will be determined by the combined properties of the layers through which they propagate. Conclusion, in layered media, the velocity of propagation of a surface wave depends on the frequency (or wavelength) of the wave. This variation of velocity with frequency is called dispersion. Therefore, all layers in the profile can be sampled simply by generating surface waves over a wide range in wavelengths (i.e. a wide range in frequencies) and the velocities will vary with the stiffnesses and thicknesses of the layers in the system. The objective in the field testing of the SASW method is to measure this surface wave dispersion.
Surface wave velocity ($V_R$) of a uniform material is closely related to the shear wave velocity ($V_s$) of the material. The surface wave propagates at a velocity slightly less than the shear wave velocity. The relationship between surface wave velocity, shear wave velocity and poisson's ratio ($v$) can be approximated by (Achenbach, 1973):

$$V_R = \frac{0.862 + 1.14v}{1 + v} \times V_s$$

(1)

For values of Poisson's ratio between 0.1 and 0.3, surface wave velocity can be approximated by:

$$V_s \approx 0.9 \times V_s \quad \text{for } 0.1 < v < 0.3$$

(2)

By using the theory of elasticity, values of modulus can be calculated from shear wave velocity and mass density ($\rho$) as follows:

$$G = \rho \times V_s^2$$

(3)

$$E = 2 \times G \times (1 + v)$$

(4)

where $G$ is shear modulus and $E$ is Young's modulus.

**Testing procedure**

The general configuration of the source, receivers, and recording equipment is shown in Fig. 2. Surface waves are generated by applying a dynamic vertical load to the ground surface. The propagation of these waves along the surface is monitored with two receivers placed at distances of $d_1$ and $d_2$ from the source. Additionally, distance $d_2$ is usually equal to two times $d_1$. In principle it should be possible to use a single source/receiver spacing for the entire test. However, practi-
cal considerations such as attenuation and near-field effects dictate that several different source/receiver spacings must be used. Typically distances between receivers of 1, 2, 4, 8 and 16 m are used if the soil profile is to be evaluated to a depth of about 10 m. For pavement sites, these distances range from 0.15 m to 1 m. The spacing between the receivers is varied according to the wavelengths ($\lambda_n$) being measured. At each spacing, one can typically measure wavelengths ranging from twice the receiver spacing to less than one-third of the receiver spacing (Rix et al, 1990).

Surface wave frequencies used in SASW testing range from several Hertz to about 500 Hz for most soil sites. In this case, velocity transducers with natural frequencies between 1 and 4.5 Hz work well as receivers. When rock or some other stiff material such as concrete is at the surface, frequencies on the order of 10 to 50 kHz must be generated, and accelerometers are used as receivers. In both instances, the most common types of sources are either simple hammers (small, hand-held hammers or sledge hammers) or dropped weights weighing from 200 to 1500 N. Electromagnetic vibrators in conjunction with sinusoidal or random input motion can also be used as sources.

A dual channel Fast Fourier Transform (FFT) dynamic signal analyser is used to record and analyse the motions at any two velocity transducers. The ability to calculate transforms rapidly in the field, is an essential part of the SASW method, allowing operators to immediately assess the quality of the data being collected and, if necessary, modify the arrangement of source and receivers or other test parameters accordingly. This data can be easily transferred to a microcomputer for further analysis as desired.
Analysis procedure

For each source/receiver spacing, the time histories recorded by the two receivers, \( x(t) \) and \( y(t) \), are transformed to the frequency domain resulting in the linear spectra of the two signals, \( X(f) \) and \( Y(f) \). The cross power spectrum of the signals, \( G_{xy}(f) \) is then calculated by multiplying \( Y(f) \) by the complex conjugate of \( X(f) \). In addition to the cross power spectrum, the coherence function and auto power spectrum of each signal are also calculated. It must be emphasized that all of these frequency domain quantities are calculated in real time by the waveform analyzer. The key data, consisting of the phase of the cross power spectrum and the coherence function, are shown in Fig. 3. The coherence function represents a signal-to-noise ratio and should be nearly one in the range of acceptable data (e.g. 27 to 100 Hz in Fig. 3).

The phase of the cross spectrum represents the phase difference of the motion at the two transducers. The surface wave velocity \( (V_s) \) and the wavelength \( (\lambda) \) can be determined from the phase of the cross spectrum \( (\Theta_{xy}(f)) \) using the following expressions:

\[
t(f) = \Theta_{xy}(f)/2\pi f
\]

where the phase angle is in radians and the frequency, \( f \), is in Hertz. The surface wave phase velocity, \( V_s \), is determined using:

\[
V_s(f) = (d_2 - d_1)/t(f)
\]

and the corresponding wavelength of the surface wave is calculated from:

\[
\lambda = V_s/f
\]
The result of these calculations is a dispersion curve \( V_s \) versus \( L_i \) for one receiver spacing. Individual dispersion curves from all receiver spacings are assembled together to form the composite dispersion curve for the site. An example of a composite dispersion curve is presented in Fig. 4. The portion of the curve determined from the phase record shown in Fig. 3a is outlined by the dashed box in Fig. 4.

For a layered system in which stiffness changes with depth, an inversion process is required to obtain the stiffness profile from the measured dispersion curve. This requires that a velocity profile be assumed, and a theoretical dispersion curve be calculated for that profile (Nazarian, 1984). The theoretical dispersion curve is then compared to the measured curve, and the assumed profile is adjusted in an attempt to improve the match. This procedure is repeated until the theoretical and measured dispersion curves closely match at which time the assumed profile is taken to represent the stiffness profile in the material system. The comparison of theoretical and field dispersion curves after the final adjustment of velocities and thicknesses is illustrated in Fig. 5. The final profile is assumed to represent the actual site conditions accurately. Application of inverse theory to surface wave testing has increased the accuracy of resulting wave velocity profiles and has significantly expanded the variety of sites at which the SASW method can be successfully used.

**SASW Research at the University of Ghent**

The SASW Research at the University of Ghent is performed in close cooperation with Prof. K.H. Stokoe. From his work we took the following summary (Bay and Stokoe, 1990).
Fast determination of deformation characteristics

When the only layer of interest is a homogeneous surface layer, as is generally the case for a concrete slab, then a simplified inversion procedure can be employed. Only the initial portion of the curve is of concern where wavelengths shorter than the thickness of the top layer propagate as if they were in a uniform half-space. In other words, the short wavelengths are not influenced by the material beneath the slab. Thus, by only using wavelengths shorter than the thickness of the concrete slab, the stiffness of the slab can be determined directly from the dispersion curve using Eqs. 1-4.

Integrity Testing

The SASW method has the potential to be used for integrity testing of members in the field. Both velocity and attenuation measurements can be used. However, velocity measurements are easier to perform and quantify.

Specific applications under study will be:
1. identification of zones of deteriorated material with a lateral extent greater than the thickness of the concrete member and
2. identification of localized flaws.

In the first case, the zone (layer) can be mapped vertically within the concrete member by the velocity or stiffness profile. This effort generally requires complete inversion of the dispersion curve. In the second case, void detection in curing concrete can be studied. Voids may create recognizable patterns in the phase diagram and dispersion curve. In these applications as well as others, the nonintrusive characteristic of the method make it's application to concrete slabs particularly attractive.
Variations in Seismic Velocities with Time

Any technique used to monitor the stiffness of curing concrete must be effective over an extremely broad range of stiffnesses. The SASW method has proven to be very suitable in this regard. A series of dispersion curves from successive tests performed on one slab during curing is shown in Fig. 6. The velocities range from a few hundred feet per second, typical of soft soils, to nearly 8000 ft/sec, representative of cured concrete or hard rock.

One important point to note in Fig. 6 is that each dispersion curve is essentially horizontal. This means that the slab is uniform in stiffness and has no voids or flaws in the area of testing. The minor fluctuations in the curves are due to the presence of body wave reflections off the bottom of the slab and surface wave reflections off the sides of the slab which arrive too quickly to be eliminated with the windowing function.

The resulting variation in surface wave velocity with time is shown in Fig. 7. Also shown in the figure is the variation in compression wave velocity with time. Both velocity-time relationships show the same trend, indicating that surface waves can be useful in measuring the response of concrete during and after curing.

Elastic Properties of Curing PCC

By applying the relationship in Eqs 1-4 to the measured wave velocities, the elastic properties of the curing concrete were determined. Values of shear modulus, Young's modulus, and Poisson's ratio are shown in Fig. 8. The uncured concrete is initially more "fluidlike", with Poisson's ratio nearly equal to 0.5. As the concrete cures, the stiffness in shear greatly increases, causing Poisson's ratio to decrease and become about 0.25 in this case.
Case studies

Another promising result of the SASW method is the measurement on municipal waste disposals. Some preliminary tests were performed to have an idea on the density of the waste. Fig. 9 shows the results of these measurements. On one place, 4 tests with different receiver spacings give dispersion curves which are increasing with wavelength (or depth). A few meters further two tests give dispersion curves which are decreasing with depth which indicates a lower density on this spot. Further measurements will be performed to investigate the waste disposal soil improvement by heavy tamping.

SASW-tests were performed on a compacted gravel bed besides a building. This gravel bed served as a foundation for a crane which collapsed on the building. Three tests were performed on 0.3 m, 1 m and 2 m from the wall (Fig. 10). These tests indicated that the gravel was less compacted (and gave a lower Young modulus) near the wall which caused excessive settlements of the crane base just besides the wall.
FIGURES
Distribution of Vertical Particle Motion with Depth for Two Surface Waves of Different Wavelengths. (after Rix and Stokoe, 1989)

Fig. 1

Source-Receiver Configuration

Fig. 2
a. Phase of the Cross Power Spectrum

b. Coherence Function

Key Records for One Receiver Spacing

Fig. 3

Field Dispersion Curve Developed from All Receiver Spacings. (after Stokoe et al. 1989)

Fig. 4
Comparison of Theoretical and Field Dispersion Curves (after Stokoe et al 1989)

**Fig. 5**

Examples of dispersion curves measured on one curing concrete slab at various times after the addition of water to the concrete mix.

**Fig. 6**
Variation in measured compression and surface wave velocities during curing of concrete slab

Fig. 7

Variation in Young's modulus, shear modulus, and Poisson's ratio during curing of concrete slab

Fig. 8
3. FOUNDATION ENGINEERING

3.1 Technical papers

Base capacity of bored piles in sands from in situ tests
V N Ghionna, M Jamiolkowski, R Lancellotta, S Pedroni

Reduction of vertical stresses on rigid pipes by the use of soft inclusions under the invert
N S D Liedberg

3.2 Literature and references (not included)


Base capacity of bored piles in sands from in situ tests

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Technical University of Turin, Italy
Pedroni, S.
Enel Cris, Milano, Italy

ABSTRACT: The paper examines the evaluation of the base capacity of large diameter bored piles in sands. After an experimental confirmation that the base resistance at failure is independent from the pile installation, the concept of the critical base resistance $q_{bc\text{crit}}$ correlated to a value of critical settlement of the pile is reviewed. The empirical rules correlating the value of $q_{bc\text{crit}}$ to the results of penetration tests are given. An approach referring to the theory of elasticity and making use of the operational stiffness of the sand for the assessment of $q_{bc\text{crit}}$ is suggested.

INTRODUCTION

Stress relief and loosening of the soil under the pile tip, both associated with drilling operations of bored piles, are responsible for the large relative displacement ($s/D$) necessary for the full mobilization of the ultimate base resistance ($Q_{ub}$), much larger than for driven piles.

Based on the data collected by De Beer (1984, 1988), Reese and O'Neill (1988) and Hirayama (1990) it is possible to figure out, in sands and gravels, the following ratios of $Q_{b}(B)$ to $Q_{b}(D)$ mobilized at a given value of $s/D$:

$$
\begin{array}{c|c}
\text{s/D} & Q_{b}(B)/Q_{b}(D) \\
\hline
0.05 & 0.15 \text{ to } 0.21 \\
0.10 & 0.30 \text{ to } 0.50 \\
0.25 & 0.30 \text{ to } 0.70 \\
\rightarrow \infty & \equiv 1.0 \\
\end{array}
$$

Table 1 - Ratio of $q_{b}(B)/q_{b}(D)$ as function of $s/D$.

where:

$Q_{b}(B)$ - base resistance mobilized by a bored pile at a given value of $s/D$

$Q_{b}(D)$ - base resistance mobilized by a displacement pile at the same value of $s/D$

$s$ - settlement of the pile tip

$D$ - diameter of the pile tip.

The data exposed in Table 1 implicitly postulate that there is a unique ultimate pressure $q_{ub}$ hence, unique value of $Q_{ub}$ under the tip of a single pile embedded in cohesionless soil, which is independent from the method of pile installation.

However, the relative displacements required to mobilize a fraction of $q_{ub}$ are strongly dependent on the method used for the pile installation and in case of bored piles even on minor construction details, e.g. O'Neill et al. (1992).

In support of the above, convincing evidences are brought by De Beer (1984) as a result of full scale load tests on bored and driven piles embedded in sands.

A verification of the independence of $q_{ub}$ from the pile installation procedure and its substantial coincidence with the cone resistance $q_c$ measured during a steady penetration has been attempted by the writers in the ENEL CRIS calibration chamber [Bellotti et al. (1982, 1988)] via "deep" plate loading tests (PLT's). The essential features of these experiments are shown in Fig.1.

Two types of tests have been carried out: aimed at corroborating the idea of uniqueness of $q_{ub}$ and its trend to equal $q_c$: during the first series of tests, the flat plate D=35.7 mm, constituted by the CPT shaft without conical point, has been pre-installed in the CC specimen during its formation.

Thereafter, the specimen has been subject to one-dimensional compression assigning to it the desired stress history. Once the conso-
Validation phase was completed, the plate was loaded and when relative settlement of 3 to 4 was attained, the flat CPT rod has been pushed at a constant rate of 2 cm/s measuring the $q_c$.

An example of such test (n°350), performed in very dense normally consolidated (NC) predominantly silica Ticino sand, (TS) is shown in Fig.2. The same figure shows also the $q_c$ measured by pushing a flat CPT tip into the CC specimen n°351 having very similar relative density ($D_R$) and boundary stresses, at the rate of 0.001 cm/s. The results of PLT n°350 simulated the mobilization of $q_{ub}$ of a non displacement pile, i.e. an ideally installed bored pile. During the second series of tests the flat CPT tip has been pushed into the CC specimen to a depth where a plateau of $q_c$ has been attained. At this point the steady penetration was arrested, the cone rod unloaded and the PLT performed. After the $q_{ub}$ was fully mobilized the steady penetration was resumed till the bottom of the CC specimen. The results of CC test n°355 performed in the very dense, NC specimen of TS is reported in Fig.3. The PLT stage of such test simulated the mobilization of $q_{ub}$ of a displacement pile.

The results of the experiments summarized in Fig.s 2 and 3 allow the following comments:
- The physical ultimate bearing pressure $q_{ub}$ of a deep foundation in TS is independent from the installation method and is controlled by the relative density $D_R$ and...
by the state of the effective stresses.
- For a given test condition (D-plate - D-penetrator) the value of \( q_{ub} \) is for all practical purposes equal to the measured \( q_c \).
- For the non-displacement model pile (test \#350) Fig.2 shows that the \( q_{ub} \) has been mobilized at \( s/D - 3 \) to 4 while, for the displacement model pile (test \#355) the \( q_{ub} \) has been mobilized at \( s/D \leq 0.5 \).
- The penetration rate has a negligible influence on the \( q_c \) of TS.

Fig.4 reports the relationship between \( q_{ub} \) and \( s/D \) as obtained by O'Neill (1992) during a load test on an instrumented bored pile D=760 mm having the tip embedded in a very dense silty sand with standard penetration resistance \( N_{spr} \) - 100 blows/foot. The same figure reports the average unit shaft friction \( f_u \) mobilized during the load tests also as function of \( s/D \).

The data reported in Fig.4 confirm that to achieve the \( q_{ub} \) a very large relative displacement of the pile tip is required also for full scale bored piles.

On the contrary, a displacement of only 10 to 20 mm is sufficient to mobilize the ultimate shaft friction load.

This implies that a rational assessment of the design load \( Q_u \) cannot consist in a summation of the two components, ultimate shaft resistance \( Q_{us} \) and \( Q_{ub} \), and their division by a global safety factor \( F \).

On the contrary, the settlement dependency of the \( Q_{us} \) and \( Q_{ub} \) should be taken into account in order to satisfy the specific performance criteria.

According to Franke (1991), the following three situations can be envisaged for large bored pile, having the tip embedded in cohesionless soil:

- Shear failure takes place at the soil-shaft interface and beneath the pile tip, i.e. the physical bearing capacity failure:
  \[ Q_u = Q_{us} + Q_{ub} \]
  is reached.
- Due to excessive total and differential settlement of piles an collapse mechanism takes place in the supported structure.
  In this case the maximum load \( Q_{crit} \) on the pile is defined with reference to a some critical value of the settlement \( s_{crit} \) whose magnitude depends on the type of structure in question.
  The value of \( Q_{crit} \) is defined as follows:
  \[ Q_{crit} = Q_{us} + Q_{bcrit} \]
  where:
  - \( Q_{crit} \) - load on the pile head at \( s = s_{crit} \)
  - \( Q_{us} \) - ultimate shaft resistance
  - \( Q_{bcrit} \) - base resistance mobilized at \( s = s_{crit} \).
  The above formula implies that the settlement of the pile required in order to mobilize \( Q_{us} \) is less than \( s_{crit} \).

- Serviceability limit state (SLS) which corresponds to a situation where the excessive total and/or differential settlements determine the loss of the serviceability of the facility supported by piles.

The determination of the load on the pile corresponding to the SLS involves a soil-pile-structure interaction analysis whose discussion is beyond the scope of the present work.

Within the above scenario it is obvious that for large diameter bored piles, owing to the very high values of \( s/D \) at which the \( q_{ub} \) is achieved, only the evaluation of \( Q_{crit} \) is of practical interest.

Therefore, the following part of the paper will be devoted to the evaluation of the unit base resistance \( q_{bcrit} \) at a pre-established value of \( s_{crit} \).

CRITICAL LOAD OF A SINGLE BORED PILE: EXISTING APPROACHES

At present, the evaluation of the \( q_{bcrit} \) is based on one of the following two types of approaches:

- Reference is made to the semi-empirical bearing capacity formulae allowing to assess the \( q_{bcrit} \) as function of the effective overburden stress \( \sigma_0 \) and of the friction angle \( \phi \), e.g. Vesic (1967) and Berezantzev (1970).

- Said formulae that combine the classical
bearing capacity solutions with the results of the model tests lead to the values of \( q_{\text{bcrit}} \) mobilized at values of \( s/D \) around 0.15 to 0.25, see Berezantzev (1970).

The \( q_{\text{bcrit}} \) is established using the empirical rules linking its value either to the measured penetration resistance, \( q_c \) from CPT or to the \( N_{\text{SPT}} \) from SPT, e.g. Reese and Wright (1977), Bustamante and Gianessi (1982), de Beer (1984, 1988), Jamiołkowski and Lancellotta (1988), Reese and O'Neill (1988), Franke (1989) and Van Impe (1986, 1991).

These rules worked out on the basis of results of full scale load tests, define the \( q_{\text{bcrit}} \) as the unit soil resistance mobilized under the pile tip at a relative displacement ranging between 0.05 and 0.10.

For example Reese and O’Neill (1988) adopt the \( s_{\text{crit}} = 0.05 \) \( D \) suggesting the following relationship between \( N_{\text{SPT}} \) and \( q_{\text{bcrit}} \):

\[
q_{\text{bcrit}} = 0.06 N_{\text{SPT}} \text{ MPa}
\]

This relationship holds for \( 0 < N_{\text{SPT}} \leq 5 \) blows/foot, for \( N_{\text{SPT}} > 75 \) blows/foot a constant value of \( q_{\text{bcrit}} = 4.5 \) MPa is adopted in the design. Should the value of \( s_{\text{crit}} = 0.05 \) \( D \) be considered excessive to keep the \( s/D \) within the desired value the value of \( q_{\text{bcrit}} \) can be reduced making reference to the load transfer curves given by Reese and Wright (1977) and shown in Fig. 5. The results of the load tests on bored piles performed by Caputo and Viggiani (1988) support the range of \( q_b \) vs \( s/D \) displayed in Fig. 5.

Jamiołkowski and Lancellotta (1988) based on the results of 15 load tests on bored piles suggested the empirical relationship:

\[
q_{\text{bcrit}} (\text{at } s/D = 0.05) = 0.2 q_c
\]

Franke (1989) suggests a more conservative relationship:

\[
q_{\text{bcrit}} (\text{at } s/D = 0.1) = 0.2 q_c
\]

With the aim to investigate the relationship between \( q_{\text{bcrit}} \) and \( q_c \), a series of 34 PLT’s in medium dense and very dense dry TS have been carried out in the ENEL-CRIS CC [Bel- lotti et al. (1982)] using a rigid plate having \( D \)=104 mm. The schematic configuration of these PLT’s is shown in Fig.1 and the obtained values of \( q_b \) at \( s/D \) equal 0.02, 0.05 and 0.10 respectively, are shown in Table 2. The values of \( q_{\text{bcrit}} \) inferred from the CC test \#350, previously, expounded, are also reported in the same table.
Table 3 - Influence of boundary conditions imposed during CC tests on $q_b$ in very dense Ticino sand.

FEM analyses have been also carried out with the aim to ascertain if the finite dimensions of the CC, see Fig. 1, influence the results of the PLT's. Based on these analyses, for which the non-linearity in stiffness of the TS has been modelled adopting the hyperbolic stress-strain relationship [Duncan et al. (1980)] the results obtained showed that up to the relative settlement $s/D = 0.25$ the load-settlement relationship inferred from the PLT's in the CC is almost identical to that modelled numerically in an infinite cylinder, see the example shown in Fig. 6 [Bocchio (1993)]. In such analyses the maximum shear modulus $G_0$ from the resonant column tests has been used to evaluate the initial stiffness of TS, it was made dependent on the mean effective stress $p'$.

From what above stated, one can argue that the values of $q_b/q_c$ function of $s/D$ given in Table 2 are not significantly influenced by the size of the CC specimen and by the imposed boundary conditions.

Both $q_{bcrit}$ vs $N_{PT}$ and vs $q_c$ relationships reported in the literature and those described in this paper apply to bored piles installed under well controlled conditions in medium dense to very dense cohesionless soils, with the relative embedment in the bearing layer not less than 8D. For piles with a smaller relative embedment the values of $q_{bcrit}/q_c$ given in this paper should be reduced multiplying by the coefficient $a_c$ given in Fig. 7, see also De Beer (1963, 1965, 1971).

The problem of the scale effect intended as the possible influence of the $D$ on the $q_{bcrit}$ is not well understood yet. It appears that there is a trend of $q_{bcrit}/q_c$ ratio to decrease with increasing the pile diameter, especially when $D > 1$ m, see Meyerhof (1983), Franke (1984) and Reese and O'Neill (1988). The only way allowing to take into account the effect of $D$ is to refer to a continuum approach.

CRITICAL LOAD OF A SINGLE BORED PILE: SUGGESTED APPROACH

Despite the experience gained with the above mentioned approaches the writers believe that the relevant factors affecting the relationship between the mobilized $q_b$ and the corresponding magnitude of $s/D$ can be properly evaluated by means of the elasti-

\begin{table}[h]
\centering
\begin{tabular}{|c|c|c|c|c|c|c|}
\hline
Test & $D_R$ (%) & $p'$ (kPa) & $q_b(0.02)$ (MPa) & $q_b(0.05)$ (MPa) & $q_b(0.1)$ (MPa) & Boundary conditions \\
\hline
324 & 90.9 & 41 & 1.09 & 1.99 & 2.74 & $\sigma_v' = \text{const.}$ \hspace{1cm} $\sigma_h' = \text{const.}$ \\
326 & 90.9 & 38 & 1.01 & 1.92 & 2.90 & $\sigma_v' = \text{const.}$ \hspace{1cm} $\sigma_h' = \text{const.}$ \\
327 & 90.6 & 39 & 1.08 & 2.07 & 3.25 & $\sigma_v' = 0$, $\sigma_h' = 0$ \\
328 & 90.6 & 39 & 1.15 & 1.98 & 2.71 & $\sigma_v' = 0$, $\sigma_h' = \text{const.}$ \\
\hline
\end{tabular}
\caption{Base resistance mobilized at relative settlement $s/D$ equal to 0.02, 0.05 and 0.1 respectively.}
\end{table}

$q_b$ = Base resistance mobilized at relative settlement $s/D$ equal to 0.02, 0.05 and 0.1 respectively.
city theory. In other words, it is suggested that for a given value of \( s_{\text{crit}}/D \), the \( q_{\text{crit}} \) should be evaluated using the following formula:

\[
q_{\text{crit}} = \frac{8 G z_{\text{crit}}}{\pi D (1-v) f(z/D)}
\]

where:
- \( G \) - equivalent secant shear modulus function of the assumed value of \( s_{\text{crit}}/D \)
- \( f(z/D) \) - non-dimensional factor ranging for a rigid plate from 1 to 0.85 for surface \((z/D=0)\) and deeply \((z/D=8)\), embedded plate respectively
- \( v \) - Poisson coefficient
- \( z \) - depth of the plate embedment

The key issue in the use of the above formula for the preliminary assessment of the \( q_{\text{crit}} \) is related to the ability of the designer to select in a reliable manner:
- the value of \( s_{\text{crit}} \) corresponding to the achievement of the ULS of the given structural system;
- the value of \( G \) which takes properly into account the non-linearity of the soil stiffness.

Within this frame, Fig. 8 shows the degradation of \( G/G_0 \) normalized with respect to the initial shear modulus \( G_0 \), as function of the thickness \( s/D \) inferred from the PLT's performed in the 15 m deep lined shafts, which were dug out in the slightly cemented sand of miocene age and in the almost uncemented sand and gravel of pleistocene age at the site proposed for the Messina Strait Crossing by a suspended bridge, see Crova et al. (1992). The diameter of the shafts was 2500 mm and that of the plate 800 mm while the \( G_0 \) has been inferred from the shear wave velocity measured using the cross-hole technique. The values of \( G \) from the PLT's under question and those performed in the CC discussion of which follows were computed by means of the formula of the theory of elasticity:

\[
G = \frac{\sigma}{\varepsilon} \cdot \frac{\pi}{8} D (1-v) f (z/D)
\]

adopting \( v=0.2 \) and being \( \sigma \) equal to the pressure acting on the plate.

It results, see Fig.8, that assuming \( s_{\text{crit}}=0.05 D \) the equivalent stiffness reduces up to 5.1% and 7.2% of its initial value \( G_0 \), in miocene and pleistocene deposits respectively.

### Figures

**Figure 8** - Degradation of \( G/G_0 \) at Messina Strait site.

Similar curves have been compiled for the TS based on the results of the PLT's performed in the CC using the values of \( G_0 \) obtained from the resonant column tests by Lo Presti (1987). These curves, shown in Figs.9 and 10, exhibit a degradation of \( G/G_0 \) which trend is similar to that observed in the natural deposits at the Messina Strait site.

**Figure 9** - Degradation of \( G/G_0 \) with \( s/D \) in medium dense NC Ticino sand.

**Figure 10** - Degradation of \( G/G_0 \) with \( s/D \) in NC and OC Ticino sand.
An overall analysis of Figs.8 to 10 allows the following comments:

- No matter how the $s_{cr}$ is defined, the operational stiffness available under the pile tip is only a small fraction of the initial stiffness.

- Tentatively, for the assessment of $q_{cr}$ one can refer to the following values of the $G/G_0$ ratio:

$$s_{cr}/D = 0.05 : 0.07 \leq G/G_0 \leq 0.11$$

$$s_{cr}/D = 0.10 : 0.05 \leq G/G_0 \leq 0.08$$

where the lower and upper limits refer to loose and very dense sand respectively. On average it results that the ratio of $G/G_0 \leq 1$ is proportional to $(s/D)^n$ with $n = -0.5 \pm 0.05$.

- The mechanical overconsolidation influences in a negligible manner $G/G_0$ at the values of $s/D$ of practical interest.

- Comparing the degradation of soil stiffness at the Messina Strait site (Fig.8) with that of the pluvially deposited TS (Figs.9 and 10), it appears that the phenomenon is more pronounced in natural deposits than in sands reconstituted in laboratory. This agrees with the findings presented by Thomann and Hryciw (1992) and Ishihara (1993).

**DESIGN LOAD**

In order to evaluate the design load $Q_d$ the following factors need being considered:

- Spatial variability of strength and deformation properties of cohesionless deposits. For example, Burland and Burbridge (1984) found that in apparently constant geological conditions the observed settlements of foundations scattered around $\pm 50\%$ of the average value.

- According to Berardi and Lancellotta (1993), in addition to the soil variability the model uncertainty linked to the evaluation of $s_{cr}$ hence of $q_{cr}$ should also be taken into account when selecting the $F$.

In particular, when $q_{cr}$ is evaluated on the basis of an estimate of settlement, the accuracy and reliability of the calculation method used should be assessed making reference to the basic variable, the ratio of the computed $(s_c)$ to the measured $(s_m)$ settlement. As an example taken from Berardi and Lancellotta (1993), Fig.11 reports the percentage of cases where this ratio is less than a given value for the number of methods presently used to predict the settlements of founda-

- The category of the engineering facility classified in terms of its importance and years of life, see Reese and O'Neill (1988).

- Quality of control exercised during the execution of the bored piles, see Wright (1977) and Reese and O'Neill (1988).

For permanent structures and with the normal controls during construction the designers are accustomed to use $2 \leq F \leq 3$, to obtain the allowable load.

The use of separate safety factors $F_s$ and $F_b$ applied to shaft and base resistance respectively even if attractive, at a first sight, lack of an adequate experience among the designers. As evidenced by Franke (1989, 1991), for a given set of $F_s$ and $F_b$ values the value of is not constant but it depends on the ratio of $Q_{us}$ over $Q_{cr}$. In most cases this renders very uncertain, a rational selection of $F_s$ and $F_b$.

A rational way of overcoming the problem is to link the design load $Q_d$ to a design capacity $Q_{cr}$ through $\gamma_m$ factors which account for the above mentioned uncertainties.

Some examples with this respect may be found in works by Wright (1977), Reese and O'Neill (1988) and Franke (1989, 1991).

**CONCLUSIONS**

On the basis of what has been already exposed the following conclusions can be drawn:

1. The plate loading tests performed in the CC confirm that the tip bearing capacity
Q_{ub} of a pile embedded in sand is virtually independent from the installation and equals the cone resistance q_{c}. However, the values of the relative displacement s/D at which the Q_{ub} is mobilized are one order of magnitude larger in case of non displacement piles if compared to those of displacement piles.

2. Due to the above, the physical failure of a single large diameter bored pile is of no practical interest and the ULS is referred to the critical load Q_{cri}. Such load corresponds to the sum of the ultimate shaft resistance Q_{um} plus the base resistance Q_{bcrit} mobilized at a certain critical value of the settlement s_{cri} which is likely to induce the collapse in the supported structure. Usually, the Q_{bcrit} is referred to q_{bcrit} corresponding to the s/D equal to 0.05 or 0.10.

3. The results of 34 "deep" plate loading tests performed in the CC yielded the following range of the q_{bcrit} expressed as fraction of the q_{c}:

\[
\frac{s/D}{0.05} = 0.14 \leq \frac{q_{bcrit}}{q_{c}} \leq 0.19
\]

\[
\frac{s/D}{0.10} = 0.20 \leq \frac{q_{bcrit}}{q_{c}} \leq 0.23
\]

Even if such range falls within that resulting from the full scale load on tests instrumented bored piles [Bustamante and Gianeselli (1982), Jamiolkowski and Lancellotta (1988), Franke (1989)], the effect of the dimensions of the pile diameter on this type of empirical rules is far from being clarified.

4. Based on the field and CC tests data a method to assess the q_{bcrit} combining the experimental degradation curves of the G/G_{0} vs. s/D with the formula of the theory of elasticity is tentatively suggested. This approach, at least in principle, allows, knowing the field value of G_{0}, to enter into the degradation curves like those exposed in Figs.8 to 10 with a given value of s_{cri}/D in order to assess the operational stiffness G with help of which q_{bcrit} is computed from formula given at page 6.

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REDUCTION OF VERTICAL STRESSES ON RIGID PIPES BY THE USE OF SOFT INCLUSIONS UNDER THE INVERT

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ABSTRACT

One of the main concerns in soil-pipe interaction problems is the magnitude and distribution of vertical stresses at the crown and at the invert. Stress concentrations at the invert, due to improper bedding, is one of the main reasons for bending failures of rigid pipes. These kind of failures will cause longitudinal cracks along the pipe at the invert, crown, and at the spring lines. Therefore, the induced trench concept has been used in various ways in order to reduce the vertical stresses on the pipe by providing for a positive arching effect in the back-fill above the pipe.

In an extensive series of full-scale field tests in sand including various types of bedding geometries, together with numerical simulations by the use of the direct design program SPIDA (Soil-Pipe Interaction Design and Analysis), it has been found that a soft inclusion under the invert is more favourable for the pipe than the traditional location above the pipe.

The larger the settlement of the pipe into the soft inclusion, cushion, the larger the area over which the reaction force at the bottom will be distributed. Furthermore, due to the favourable positive arching above the crown the lateral stresses supporting the pipe at the spring lines will increase.

The arching effect will increase with increasing cushion width, since the pipe is allowed to settle more freely without large point loads at the haunches. Hence, the weight of the soil above the crown, carried by the pipe will decrease with increasing cushion width. Due to this favourable load reduction the pipe wall moments and thrusts will decrease drastically.

In a parameter study performed with SPIDA the maximum moment was found to decrease with about 70% for a case when the width of the cushion had the same width as the external diameter of the pipe. For the same case the weight of the soil carried by the pipe was only 60% of the weight of the overburden above the crown.

The cushion can be easily designed by the use of a simple elastic, analytical approach. Furthermore, the cushion can be looked upon as a prefabricated bedding which will lead to time savings and cheaper pipe designs.
REDUCTION OF VERTICAL STRESSES ON RIGID PIPES BY THE USE OF SOFT INCLUSIONS UNDER THE INVERT

DIMINUTION DES TENSIONS VERTICALES SUR LES CANALISATIONS RIGIDES PAR L'INSERTION DE COUSINS MOUX SOUS CELLES-CI

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SYNOPSIS

Stress concentrations at the invert, due to improper bedding, is one of the main reasons for bending failures of rigid pipes. The induced trench concept has been used in various ways in order to reduce the vertical stresses on the pipe by providing for a positive arching effect in the backfill above the pipe. In an extensive series of full-scale field tests in sand including various types of bedding geometries, together with numerical simulations by the use of the direct design program SPIDA, it has been found that a soft inclusion under the invert is more favourable for the pipe than the traditional location above the pipe. Furthermore, it is shown how such inclusions can be designed by the use of a simple elastic, analytical approach. Some of the advantages with the method are pointed out together with information about the first successful cushion installation.

INTRODUCTION

In soil-pipe interaction problems the magnitude and distribution of the vertical stresses at the crown and at the invert are of special interest. Stress concentrations at the invert, due to improper bedding, is one of the main reasons for bending failures of rigid pipes. These kinds of failure will cause longitudinal cracks along the pipe at the invert, crown, and at the spring lines. Therefore, the induced trench concept has been used in various ways in order to reduce the vertical stresses on the pipe by providing for a positive arching effect in the backfill soil above the pipe.

The induced trench method, involving compressible layers of various materials above the crown of the pipe, has been used in the USA since the 1960s. In various studies performed by Larsen (1979), Jacobson et al. (1984), Vatsetz (1990) and Liedberg (1991) the advantages with the method have been demonstrated. In these studies various inorganic matters or soft soils, such as mineral wool insulation mats, geotextile, and EPS (expanded polystyrene), have been used as compressible layers. The positive load reducing effects for these materials were all quite pronounced. However, the load reduction using geotextile was somewhat time delayed due to consolidation processes within the geotextile (Jacobson et al., 1984).

The standard concept for an induced trench installation has been a compressible layer installed above the crown of the pipe. However, it is well known that it is favourable to install a pipe on a "soft" bedding. Such a bedding will distribute the reaction force at the invert over a relatively large area and reduce the negative effects of large stress concentrations at the invert. Larsen (1979) performed a series of centrifuge model tests. Both soft inclusions above and below a pipe were tested. The tests showed a striking reduction of pipe-wall moments for both of the test geometries. However, the results clearly showed that a soft inclusion under the pipe results in a larger stress reduction in the pipe than in the case of a soft inclusion above the crown of the pipe.

Hegg (1988) suggested a soft soil bedding under the middle one-third of the external pipe diameter in order to reduce pipe-wall bending moments. He also stated, "If a sufficiently soft zone is provided below the invert, the moment and shears in the vicinity of the crown, rather than the invert, will govern the design of the pipe. Also, variations in the level of compaction below the

Fig. 1. Deformed cushion under the invert in test section K, in the full-scale field tests, during the excavation 12 months after the test start.

FULL-SCALE FIELD TESTS

The field tests consisted of ten various installations of a concrete pipe of type GERMAX, with an internal diameter of 600 mm, and an external diameter of 740 mm. Each test section consisted of five pipes with a total length of 12 meters. All measurements were performed for one central pipe in each test section. These measurements included radial earth pressures along the circumference of the pipe, by means of pneumatic Glaedt earth pressure cells, readings of changes in pipe diameter in four different directions, and
sediments in the surrounding soil, by the use of bench marks extended through the fill. The central pipe was equipped with eight earth pressure cells equally spaced around the pipe, and installed in the pipe wall so that the active membrane of the cells were flush with the concrete surface as shown in Fig. 2. All pressure cells were carefully calibrated for temperature and method of installation in the pipe wall.

![Configuration of earth pressure cells in the pipe section](image)

**Fig. 2. Configuration of earth pressure cells in the pipe section**

Good as well as poor bedding and side-fill conditions were studied. The basic test geometries for the four tests discussed herein are given in Fig. 3 and can be described as:

**Section A:**
- Pipe founded directly on the firm in situ soil
- No compaction of side- and back-fill

**Section E:**
- Pipe founded directly on the firm in situ soil
- Compaction of side-fill and soil under the haunches
- Compaction of back-fill up to the ground surface
- Soft inclusion above the crown (see Fig. 3)
- No compaction of soil above the soft inclusion

**Section F:**
- Pipe founded directly on the firm in situ soil
- Compaction of side-fill and soil under the haunches
- Soft inclusion above the crown

**Section H:**
- Pipe founded directly on the firm in situ soil
- No compaction of side- and back-fill

The mean radial earth pressures measured during the test period as well as the values calculated with SPIDA are given in Fig. 4 for the four test sections discussed herein. It can be clearly seen that the pipe in section A is exposed to an unfavourable stress distribution with an extreme stress concentration at the pipe support under the invert. This is mainly due to the fact that the soil under the haunches and the side-fill was not compacted, resulting in a poor lateral support, and that the reaction force under the invert is distributed over a very narrow bedding angle. This is clearly shown by the fact that the contact pressure between soil and pipe under the haunches was measured to zero.

The pipe in section E was, however, exposed to a more advantageous stress distribution, due to the more favourable distribution of the reaction force under the invert, due to well compacted soil under the haunches. The pipe installation in section E generally fulfilled the demands, regarding the degree of compaction of the backfill, that are normally put when ground surface settlements are critical.

The use of the classical induced trench concept, by installing a soft inclusion above the crown as in section F, improved the stress distribution for the pipe so that the vertical stresses carried by the pipe were much less than for the pipes in sections A and E. Due to positive soil arching above the crown the vertical stresses in the side-fill increase, and, hence, the lateral stresses against the side of the pipe increase as well. This can be clearly seen from both field measurements and SPIDA simulations. The reduction of vertical stresses and increase in lateral side support will result in a favourable reduction of pipe-wall bending-moments.

![Basic test geometries for test sections A, E, F, and H](image)

**Fig. 3. Basic test geometries for test sections A, E, F, and H.**

The pipe in section F was, however, exposed to an advantageous stress distribution, due to the more favourable distribution of the reaction force under the invert, due to well compacted soil under the haunches. The pipe installation in section F generally fulfilled the demands, regarding the degree of compaction of the backfill, that are normally put when ground surface settlements are critical.

The use of the classical induced trench concept, by installing a soft inclusion above the crown as in section F, improved the stress distribution for the pipe so that the vertical stresses carried by the pipe were much less than for the pipes in sections A and E. Due to positive soil arching above the crown the vertical stresses in the side-fill increase. The lateral stresses against the side of the pipe increase as well. This can be clearly seen from both field measurements and SPIDA simulations. The reduction of vertical stresses and increase in lateral side support will result in a favourable reduction of pipe-wall bending-moments.

However, the stress situation can be improved even more by installing the soft inclusion as a cushion under the invert of the pipe as in section H. The dramatic stress decrease at the pipe invert is very advantageous for the pipe. The stress distribution is very close to what would be an idealistic situation with a more or less hydrostatic pressure distribution. Such a distribution would only result in thrusts in the pipe wall and no moments. However, that is not possible to achieve. In general three positive effects are achieved by the cushion. These are:

**Numerical Simulations**

The measured earth pressures in the field were compared with those calculated by the direct design program SPIDA (Soil-Pipe Interaction Design and Analysis). The SPIDA program, which is based on a finite element concept, has been jointly developed by Simpson, Compere and Cooper Inc. and the University of Massachusetts (see Heger 1982). Simulations by the use of SPIDA were performed for all bedding geometries tested in the field. These numerical simulations were performed with the soil parameters obtained from the different tests mentioned above, and should, therefore, not be considered as a parameter study.

**Results**

The mean radial earth pressures measured during the test period as well as the values calculated with SPIDA are given in Fig. 4 for the four test sections discussed herein. It can be clearly seen that the pipe in section A is exposed to an unfavourable stress distribution with an extreme stress concentration at the pipe support under the invert. This is mainly due to the fact that the soil under the haunches and the side-fill was not compacted, resulting in a poor lateral support, and that the reaction force under the invert is distributed over a very narrow bedding angle. This is clearly shown by the fact that the contact pressure between soil and pipe under the haunches was measured to zero.

The pipe in section E was, however, exposed to a more advantageous stress distribution, due to the more favourable distribution of the reaction force under the invert, due to well compacted soil under the haunches. The pipe installation in section E generally fulfilled the demands, regarding the degree of compaction of the backfill, that are normally put when ground surface settlements are critical.

The use of the classical induced trench concept, by installing a soft inclusion above the crown as in section F, improved the stress distribution for the pipe so that the vertical stresses carried by the pipe were much less than for the pipes in sections A and E. Due to positive soil arching above the crown the vertical stresses in the side-fill increase. The lateral stresses against the side of the pipe increase as well. This can be clearly seen from both field measurements and SPIDA simulations. The reduction of vertical stresses and increase in lateral side support will result in a favourable reduction of pipe-wall bending-moments.

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**Fig. 4. Basic test geometries for test sections A, E, F, and H.**
1. The pipe is resting on a high quality bedding which is distributing the reaction force of the invert over a large bedding angle. The larger the bedding angle, the larger the bedding angle.

2. Vertical stress reduction due to positive arching effects in the soil.

3. Larger lateral side support as an effect of this soil arching.

### DESIGN OF SOFT CUSHION UNDER THE INVERT

When the induced trench concept is to be used with a soft cushion under the invert, the pipe installation has to be designed taking the vertical settlements into the soft foundation into account. The pipe installation must be designed so that it settles down into its intended level. The magnitude of this settlement has to be calculated and, hence, the depth of the level of the bottom of the excavation, or the level of the installation, has to be reduced by the expected pipe settlement. The magnitude of the settlement of the pipe into the cushion should be divided into two parts, one part that is due to self-weight of the pipe and internal weight of water per unit length of the pipe, and one part that is only due to the weight of the soil carried by the pipe. The total settlement of the pipe into the cushion can then be calculated by Eq. 1.

\[ s_{\text{set}} = s_{\text{g}} + s_{\text{p}} \]  

(1)

Based on the geometry given in Fig. 6, the settlement can be calculated according to Eq. 2. The radial pressure distribution along the contact area between the cushion and the pipe for which there is vertical equilibrium between the weight of the pipe \( Q_p \), including internal weight of water, and the radial contact pressure \( \sigma_r \) can then be calculated, where \( \sigma_{\text{r}} = 0 \) at an angle \( \alpha_0 \) at the contact boundary.

\[ \sigma_r = \sigma_p \cdot \left( 1 - \cos \alpha_0 \right) \]

(2)

Fig. 5. Distribution of moments as calculated with SPIDA.

Fig. 6. Geometry for the calculation of the settlement \( s_\text{p} \).
The cushion method is suitable for situations when large vertical loads are expected on rigid pipes. For example, when the pipe is placed over firm soil or rock, the pipe is installed under higher embankments, or where the pipe is embedded in silty soils where large frost pressures might be expected. The method can also be used when just a higher quality for the pipe installation is wanted.

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K R Massarsch

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St. Louis, June 1 - 5, 1993
MAN-MADE VIBRATIONS AND SOLUTIONS

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MAN-MADE VIBRATIONS AND SOLUTIONS

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SYNOPSIS: The generation and propagation of man-made vibrations in the ground is discussed. Emphasis is placed on a simplified approach which is used to assess the most important factors, such as wave attenuation, refraction focusing and vibration amplification as a result of resonance. Practical guidelines are presented which can be used to predict vibrations and settlements in the ground. A semi-empirical relationship for the assessment of permissible vibration levels for buildings is proposed. Finally, a new ground vibration isolation method, the gas cushion screen is presented.

INTRODUCTION
During the past two decades, research in the area of soil dynamics has mainly been directed towards the solution of problems related to earthquakes and off-shore installations. Sophisticated computer programs and advanced testing methods have been developed which make it possible to analyse and predict the effect of complex ground motions on different types of structures such as nuclear and hydro-electric power plants, high-rise buildings and off-shore structures. Significant progress has also been made with respect to the determination of dynamic soil properties in the field and in the laboratory. This has led to improved quality of analytical results, the accuracy of which is strongly dependent on the selection of appropriate input parameters. It is thus surprising that few of the advances in the area of earthquake engineering have been utilised to solve "conventional" man-made vibration problems, caused by traffic, construction activities or industries.

Man-made vibration problems can be divided into two categories. The first category concerns small-amplitude vibrations, i. e. the effect on the human environment or on sensitive instruments. Acceptable vibration levels can be very low and may be chosen on a subjective basis. The second category concerns vibration problems which can cause or contribute to the damage of structures. These cases are not common but can be of concern in densely populated areas and near vibration sensitive structures, such as historic monuments or buildings on poor foundations.

In spite of the fact that damage caused by man-made vibrations is rarely spectacular, the direct and indirect cost to society can be substantial. Unfortunately, the complexity of many vibration problems makes it difficult to properly identify the main cause of damage, which may be overshadowed by a variety of other factors. Thus, vibration problems are many times decided in the courtroom on a non-technical basis or by arbitration. Many interesting case histories with valuable practical information can not be published because of legal restrictions. Unexpected damage to structures caused by vibrations, as well as over-conservative restrictions concerning vibration threshold levels can have significant economical consequences.
The present report does not attempt to give an extensive review of all recent developments related to man-made vibrations. Rather, emphasis is placed on the fundamental understanding of vibration problems. The main factors which need to be considered for a comprehensive analysis of ground vibration problems are discussed. The relative importance of various factors is assessed, using relatively simple analytical tools. The numerical results of this approach are not as exact as sophisticated analytical procedures such as the Finite Element or Boundary Element method. However, a simple model lends itself to a more critical study of important phenomena, which can be overlooked in a complex analysis.

Vibration problems caused by man-made activities can be divided into several categories. However, they all are controlled by the following common factors: 1) the dynamic characteristics of the vibration source, 2) the propagation of vibrations in the ground and 3) the effect of ground vibrations on buildings, sensitive installations or human perception, Fig. 1.

For the analysis of vibration problems it is necessary to consider the combined effect of several factors:

- the dynamic characteristics of the vibration source,
- the interaction between the dynamically loaded foundation and the soil/rock,
- the propagation of surface and body waves in the ground,
- the interaction between the ground and the affected structure, and
- the response of the structure and elements of the structure to ground excitation.

In addition, the effect of air-borne vibrations can contribute to the excitation of a structure, which under unfavourable circumstances can be as significant as vibrations transmitted through the ground.

**VIBRATION SOURCE**

The main sources of man-made vibrations are traffic (road and railway), construction activities, vibrating machines from heavy industries and construction activities (blasting, pile driving, soil compaction). Figure 2 shows typical frequency ranges and strain levels for different vibration sources, covering a wide range. It is apparent that these large variations of frequency and amplitude must be taken into consideration when assessing vibration problems.

![Fig. 2. Typical range of vibration amplitudes and frequencies for different vibration sources](image-url)
The life time of conventional engineering structures can be assumed to be about 30 years. Figure 3 presents the estimated number of significant vibration cycles (amplitudes larger than about 0.5 mm/s) for different vibration sources. Since damage to structures depends on the number of vibration cycles, this factor needs to be taken into consideration. Although, the values proposed are only of a qualitative nature, they convey the important message that a clear distinction needs to be made between different types of vibration sources (transient loading, repeated loading and steady-state vibrations).

In some cases, the location of the vibration source may be easily identified (industrial vibrations), while in other cases the vibration source can move continuously (construction activities) or pass at high speed (vehicle and train traffic). Complex vibrations can be generated by heavy trains, passing at varying speeds and from different directions. Extensive vibration measurements may be required to identify the direction of a travelling wave field. The depth of the vibration source can also change, as is the case when piles or sheet piles are driven. Blasting in tunnels can in some cases be difficult to locate. In the following sections, vibrations caused by pile driving will be discussed in detail, Massarsch (1992).

**PILE DRIVING - AN EXAMPLE**

Vibrations caused by pile driving are influenced by three main factors which govern the dynamic characteristics of the vibration source: 1) the pile hammer, 2) the pile and 3) the pile-soil interaction.

**The hammer**

Piles can either be driven by impact hammers or by vibratory hammers. In the case of an impact hammer, the pile is subjected to separate blows from a hammer, which is raised by a rope, compressed gas or air and falls back by gravity. Impact hammers are not rigidly connected to the pile and the driving energy is transmitted through a hammer cushion, positioned on the head of the pile or in the base of the hammer. A significant part of the impact energy can be dissipated in compressing the cushion blocks when driving on the head of the pile. At every blow of the hammer, the inertia of the pile and the resistance of the soil must be overcome.

Vibrating hammers function in a fundamentally different way than drop hammers. A vibratory hammer consist of a static mass, which is connected to the oscillating part by steel or plastic springs. The oscillator can be driven by an electric motor or hydraulically. The oscillating part of the vibratory hammer is rigidly attached to the head of the pile and the pulsating force, which acts along the entire pile, forces it to continuously move in the up-ward and down-ward direction. The frequency of the pulsating force can vary and ranges typically between 10 and 50 Hz. Modern vibrators can be very powerful and permit the continuous variation of vibration frequency and amplitude.

The energy transfer from a vibrator to the pile is efficient as no energy is lost in the connection between the pile and the vibrator. The pile is kept in an oscillating motion which minimises the bonding between the pile and the soil.

**The pile**

The energy delivered by the impact of the hammer generates time-dependent stresses and displacements in the pile. The pile behaves as an elastic bar in which the stresses travel...
longitudinally as waves. The velocity of longitudinal wave propagation is constant for a given material and can be calculated from

\[ C = \left( \frac{E}{g} \right)^{1/2} \]  \hspace{1cm} (1)

where \( E \) is the modulus of elasticity of the pile material and \( g \) is the acceleration of gravity. The force \( P \), transmitted across a section of the pile, is obtained from

\[ P = \rho C A \cdot v \]  \hspace{1cm} (2)

where \( \rho \) is the density of the material, \( A \) is the cross-sectional area of the pile and \( v \) is the vibration velocity. The capability of the pile to transmit the longitudinal force is determined by its impedance \( I \),

\[ I = \rho C A \]  \hspace{1cm} (3)

The force exerted on the pile head must be at least so high that the resistance of the soil acting along the shaft and the toe of the pile can be overcome. The impedance of the pile limits the force that can be transmitted by the pile, independent of the energy that is applied to the pile head. The impedance can vary significantly and some typical values for different pile types are given in Table 1.

Table 1 indicates that piles with approximately the same exterior dimensions but of different materials have widely varying impedance values. The impedance of a steel pipe pile is almost 10 times higher than that of a wooden pile.

The energy transmitted from the pile to the soil depends mainly on the type and efficiency of the hammer, the nature of the impulse (transient or steady-state) and the impedance of the pile. The intensity of ground vibrations is related to the energy imparted to the pile by the pile hammer. In an interesting study of case histories related to vibration problems due to pile driving, Heckman and Hagerty (1978) showed the significance of pile impedance for the transmission of vibration energy from the pile to the surrounding soil.

### Table 1. Impedance of different pile types (after Peck et al., 1974)

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>AREA (cm²)</th>
<th>IMPEDANCE (kNs/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wood</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kiln dry</td>
<td>506</td>
<td>137</td>
</tr>
<tr>
<td>Southern pine</td>
<td>506</td>
<td>160</td>
</tr>
<tr>
<td><strong>Concrete</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10 inch (25.4 cm)</td>
<td>506</td>
<td>421</td>
</tr>
<tr>
<td>20 inch (50.8 cm)</td>
<td>2027</td>
<td>1680</td>
</tr>
<tr>
<td><strong>Steel</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HP 10 x 57</td>
<td>108</td>
<td>446</td>
</tr>
<tr>
<td>HP 12 x 53</td>
<td>100</td>
<td>434</td>
</tr>
<tr>
<td>HP 14 x 117</td>
<td>222</td>
<td>959</td>
</tr>
<tr>
<td>Pipe 10/3 x 0.188</td>
<td>40</td>
<td>166</td>
</tr>
<tr>
<td>Pipe 10/3 x 0.279</td>
<td>59</td>
<td>257</td>
</tr>
<tr>
<td>Pipe 10/3 x 0.365</td>
<td>77</td>
<td>316</td>
</tr>
<tr>
<td>Pipe 10/3 x 0.188 (mandrel driven)</td>
<td>344</td>
<td>1416</td>
</tr>
<tr>
<td><strong>Steel-Concrete</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pipe 10/3 x 0.279</td>
<td>576</td>
<td>634</td>
</tr>
</tbody>
</table>

In evaluating the effect of pile driving, they used the semi-empirical relationship proposed by Attwell and Farmer (1973) to estimate the ground vibration amplitude, \( v \)

\[ v = K \left( \frac{W}{D} \right)^{0.5} \]  \hspace{1cm} (4)

where \( K \) is an empirical coefficient, \( W \) is the energy applied to the pile head and \( D \) is the distance from the pile. Figure 4 shows the variation of the K-Factor as a function of pile impedance. The findings of Heckman and Hagerty are important as they show that at the same site and at the same distance from a pile, a reduction of the pile impedance by 50% (from 2000 to 500 kNs/m) can increase the ground vibration amplitude by a factor of 5. It is thus apparent that the determination of pile impedance is an important factor when assessing the transmission of vibration energy from the pile to the surrounding soil.
Pile-Soil Interaction

The capacity of a pile at a given depth in the ground depends on the force that can be exerted to achieve the downward movement. Enough force must be transmitted to the pile head to overcome the shaft and toe resistance. Part of the energy, transmitted through the pile is transferred to the soil along the pile shaft and part to the toe. The displacement of the soil by the penetrating pile generates both plastic and elastic deformations. Beyond a short distance from the pile (about one pile radius), most of the energy is propagated in the form of elastic waves. These elastic waves comprise body waves, which radiate energy in all directions in the ground, and surface waves, which transmit the energy along a zone close to the ground surface. Two types of body wave can occur, compression (P-) and shear (S-) waves. These waves propagate at different velocities, depending on the soil properties.

The surface (R-) wave propagates close to the ground surface at a speed slightly slower than the shear wave. Another wave type, the Love (L-) wave can occur at the interface of two soil layers, and has a large transverse amplitude component. Figure 5 shows a schematic representation of the wave mechanism, created in the ground by the impact at the head of a pile, Martin (1980).

It can be concluded from Figure 5 that, although the loading force is generally in the vertical direction during impact pile driving, the maximum ground vibrations can contain horizontal and vertical components of motion.

The location of the origin of energy transfer from the pile to the soil depends strongly on the soil layers through which the pile is driven. In a dense, homogeneous sand deposit, a large part of the driving energy can be transferred along the pile shaft, generating friction-induced conical waves. In the case of a stiff layer at the toe of the pile, two types of wave fields may be created, one originating from the toe of the pile and another one induced by the bending (flexing) of the pile.
WAVE PROPAGATION IN THE GROUND

The main factors which influence the propagation of vibrations in the ground are 1) wave attenuation, 2) vibration focusing and 3) resonance. These phenomena are complex and can thus be discussed only in a simplified way.

Wave Attenuation

Elastic waves, which are generated by a vibration source, attenuate as they propagate through the soil. Wave attenuation is caused by two different effects: 1) enlargement of the wave front as the distance from the source increases (geometric damping), and 2) internal damping of the wave energy by the soil. The attenuation of waves in the soil due to geometric damping can be described by the following general relationship

\[ \frac{A_2}{A_1} = \left( \frac{R_2}{R_1} \right)^{-n} \]  \hspace{1cm} (5)

where \(A_1\) and \(A_2\) are the vibration amplitudes at distances \(R_1\) and \(R_2\), respectively. The exponent depends on the wave type as follows:

<table>
<thead>
<tr>
<th>Wave type</th>
<th>Exponent (n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P- and S-waves</td>
<td>1.0</td>
</tr>
<tr>
<td>P- and S-waves at surface</td>
<td>2.0</td>
</tr>
<tr>
<td>R-waves</td>
<td>0.5</td>
</tr>
</tbody>
</table>

As waves pass through the soil, part of the energy is consumed by friction and cohesion and this reduction of the vibration amplitude is called material damping. Although the process of material damping is only partially understood, it is possible to include this effect in the relationship, Eq. (5).

\[ \frac{A_2}{A_1} = \left( \frac{R_2}{R_1} \right)^{-n} \cdot e^{-\alpha(R_2-R_1)} \]  \hspace{1cm} (6)

The coefficient \(\alpha\) is called coefficient of attenuation and reflects the damping properties of the soil. The relationship of Eq. (6) is shown in Fig. 6 for the case of Rayleigh waves \((n = 0.5)\) with a reference distance \(R_1 = 10\) m.

Material damping has a strong influence on the attenuation of ground vibrations. For instance, at a distance of 50 m from the vibration source, the vibration amplitude is at least 10 times higher when soil damping is not considered than when a coefficient of attenuation of 0.10 is assumed.

![Coefficient of attenuation (1/m)](image)

It is apparent that the assessment of the coefficient \(\alpha\) is of great importance for a reliable prediction of wave attenuation. In spite of this, little relevant information is available in the literature (Richart et al., 1970, Massarsch, 1983). The coefficient of attenuation \(\alpha\), varies with soil type and vibration frequency according to following approximate relationship (after Haupt, 1986),

\[ \alpha = \frac{(2\pi Df)}{C} \]  \hspace{1cm} (7)

where \(D\) is material damping, \(f\) the vibration frequency and \(C\) the wave propagation velocity. This relationship is shown in Figure 7 for a
Fig. 7 Variation of the coefficient of attenuation \( \alpha \) \((m^{-1})\) as a function of wave velocity and frequency.

Material damping value of 5%, which is typical for the elastic range of soil deformations. The coefficient of attenuation decreases with increasing wave velocity and with decreasing vibration frequency. While the frequency of vibration is often measured in the field, the significance of the wave propagation velocity of the subsoil is often neglected.

Table 2 summarises typical values of the attenuation coefficient \( \alpha \) for different soil types and frequencies. These values are only approximate and should be verified by field measurements. However, they do agree well with values reported elsewhere (Barkan, 1962, Richart et al. 1970, and Haupt, 1986).

A variety of field and laboratory methods can be used to determine wave propagation velocities. Also, empirical relationships are often sufficient for preliminary analyses. It should be noted that for most practical purposes the propagation velocity of R- and L waves can be assumed to be equal to the S-wave velocity.

**TABLE 2. Approximate values of the coefficient of wave attenuation \( \alpha \) \((m^{-1})\) for saturated soils and typical ranges of vibration frequency**

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>WAVE VELOCITY (m/s)</th>
<th>Coeff. of Attenuation (15 Hz)</th>
<th>Coeff. of Attenuation (30 Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft Clay</td>
<td>75</td>
<td>0.09</td>
<td>0.13</td>
</tr>
<tr>
<td>Loose Sand</td>
<td>100</td>
<td>0.05</td>
<td>0.09</td>
</tr>
<tr>
<td>Medium sand</td>
<td>150</td>
<td>0.03</td>
<td>0.06</td>
</tr>
<tr>
<td>Dense Sand</td>
<td>200</td>
<td>0.02</td>
<td>0.05</td>
</tr>
<tr>
<td>Gravel</td>
<td>300</td>
<td>0.02</td>
<td>0.03</td>
</tr>
<tr>
<td>Till</td>
<td>400</td>
<td>0.01</td>
<td>0.02</td>
</tr>
</tbody>
</table>

**Vibration Focusing**

The propagation of body waves and surface waves in the ground is also strongly influenced by soil layering and the location of the ground water table. Reflection and refraction of body waves can occur at changes of wave propagation velocity. These effects are well-documented by case histories in the literature but rarely taken into consideration in practice. Bodare (1981) pointed out the importance of wave focusing by refraction which can be caused by a gradual increase of wave propagation velocity with depth. This results in a focusing effect at the ground surface at some distance from the wave source, where the effect of surface wave propagation is superimposed by the emergence of refracted body waves, Fig. 8.

**Fig. 8. Illustration of focusing effect caused by wave refraction.**

The refraction focusing problem can be studied analytically, Bodare (1981). If it is assumed that the wave propagation velocity \( C \) has a minimum
value $C_0$ at the ground surface and increases with depth $z$ according to the following function

$$C = C_0 \cosh \left( \frac{\pi z}{x_0} \right)$$

(8)

then theoretically, wave focusing by refraction occurs at a distance $x_0$ from the wave source. This relationship between wave propagation velocity, depth and focusing distance can be used to determine critical zones surrounding a vibration source. Figure 9 shows the increase of wave propagation velocity $C$ with depth $z$ at which refraction focusing can occur at a distance of $x_0 = 15$ m from the vibration source. Different values of wave velocities at the ground surface $C_0$ have been assumed. It is apparent that the assumed variation of wave propagation velocities is quite realistic and that the effect of refraction focusing should be taken into consideration. Refraction focusing can be illustrated by the following practical example, assuming a soil deposit with the following values of wave velocity, Table 3. This variation of wave propagation velocity with depth is quite common for many naturally deposited soils

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Depth, $z$ (m)</th>
<th>Wave Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>0 - 5</td>
<td>50 - 100</td>
</tr>
<tr>
<td>Sand</td>
<td>7 - 12</td>
<td>100 - 300</td>
</tr>
<tr>
<td>Till</td>
<td>12 - 14.5</td>
<td>300 - 500</td>
</tr>
<tr>
<td>Rock</td>
<td>&gt; 17</td>
<td>&gt; 800</td>
</tr>
</tbody>
</table>

TABLE 3. Assumed variation of wave propagation velocity with depth, focusing at $x_0 = 15$ m distance

Resonance Effects
Waves, which propagate through soil layers, can be amplified as a result of soil resonance. This phenomenon occurs when the dominant frequency of the propagating wave coincides with the natural frequency of one or several soil layers. The relationship between the natural frequency $f_n$ of an elastic, homogeneous soil layer with thickness $H$, resting on a rigid base can be estimated according to Roesset (1977)

$$f_n = \frac{(2n - 1) C}{(4 H)}$$

(9)

where $C$ is the compression or shear wave velocity and $n$ represents the number of frequency modes. The resonance frequency of a soil deposit, which is composed of several soil layers with the respective thickness $H$ and density $\rho$ can be approximated according to Dobri et al (1976) from Fig. 10.

As a first step, the resonance periods of the respective soil layers are determined according to Eq. (9). Then, starting from the ground surface, the dominant period of two layers, $T_0$ can be estimated from Fig. 10. The resonance period of a multi-layered deposit can be estimated by this iterative process, starting with the top layer and gradually correcting for the effect of the underlying layers.
Fig. 10. Determination of resonance period (T = 1/f) of a soil deposit with several layers

Obviously, soil stratification has a significant influence on the dynamic response of soil deposits subjected to vibration excitation. Experience gained from a variety of vibration problems suggests that resonance effects can occur already after a few loading cycles, as from impact pile driving. Bodare and Erlingson (1993) describe a case history where resonance effects have led to excessive ground vibrations and amplification effects in a large concrete structure. Resonance can also occur during the starting-up of vibratory hammers, Fig. 11. When vibrators are operated at a frequency, which corresponds to the resonance frequency of a soil deposit or of a structure in the vicinity, significant vibration amplification can occur. This resonance effect is used by the MRC system to achieve improved soil compaction, Massarsch (1991). The dynamic characteristics (frequency and amplitude) of modern vibrators can be varied and monitored continuously with electronic process control systems, Massarsch and Heppel (1991).

Soil or building response can be measured by vibration sensors, which are placed on the ground surface or on vibration-sensitive structures. The frequency and amplitude of the vibrator can then be controlled by a computerised system to avoid excessive vibrations, still obtaining optimal driving performance.

Fig. 11. Resonance of a soil layer during the starting-up of a vibratory pile hammer

Figure 12 shows a fully automated, electronic vibrator monitoring system (MPC system), which records and analyses simultaneously the performance of the vibrator and the dynamic response of the ground. Figure 13 shows a print-out by the MPC field computer, which was obtained on site during installation and extraction of a steel pile. The left part of the diagram displays as a function of time (in the vertical direction) 1) the penetration depth of the pile, 2) the hydraulic pressure and the 3) operating frequency of the vibrator (thick line).

Fig. 12. Electronic vibrator and ground response control system (MPC System)

On the right side of the diagram, 4) the penetration speed (dotted line) and the 5) vibration velocity of the ground (thick line) are recorded.
Continuous recording of vibrator performance and ground response provides valuable information concerning the efficiency of the pile driving process. The two thick lines in Fig. 13 show the variation of vibrator frequency (left) and ground vibration velocity (right). During the initial phase of pile driving, the vibrator is operated at a higher frequency (24 Hz). The penetration rate of the pile is relatively high (dotted line to the right) and the ground vibration intensity is low (thick line to the right). After about 1.5 min the vibrator frequency was reduced from 23 to 16 Hz. The penetration rate of the pile slowed down significantly and ground vibrations increased (almost by a factor of 3). After 2 minutes of pile driving (at a depth of 7 m), the pile was withdrawn, illustrating the change in ground response. Although the vibration amplitude remains unchanged (at 16 Hz), ground vibrations decrease during extraction by about 50%.

A simplified concept can be used to illustrate pile installation by a vibratory hammer with variable frequency. At a very high frequency (e.g. the resonance frequency of the pile), a large part of the vibration energy is used to overcome soil resistance. The pile penetrates rapidly and ground vibrations are relatively low, as the vibration energy is consumed at the pile-soil interface. When the vibrator frequency is gradually lowered and approaches the resonance frequency of the soil layer, pile penetration slows down and ground vibrations increase. At soil resonance, the pile oscillates with the surrounding soil and the loss of vibration energy between the pile and the soil is small. As most of the vibration energy radiates into the surrounding soil, strong ground vibrations are observed. From this simplified model it can be concluded that for efficient pile installation, the vibrator should operate at a high frequency, while soil compaction is best achieved at a low frequency, which corresponds to the resonance frequency of the soil.
SETTLEMENTS

Ground vibrations can also cause permanent deformations in the ground. When piles are installed in loose granular soils, settlements can occur even at relatively low vibration levels. Water-saturated loose soils liquefy initially as a result of excess pore water pressure. It is generally accepted that ground acceleration is the most relevant parameter to assess the risk of settlements in the ground. The degree of ground settlements depends also on the initial density of the soil.

Brumund and Leonards (1972) have suggested that the parameter that governs the ultimate residual settlements of a vibratory footing resting on the surface of a granular soil deposit is the steady-state transmitted energy. Subsidence is due partly to densification and partly to irrecoverable shear strains and densification is more likely related to the logarithm of transmitted energy (rather than directly proportional to it).

Settlements (as percentage of soil layer thickness) can range between 10% in very loose sand and silt to 1% in dense sand and gravel. Figure 14 shows an empirical relationship for estimating ground settlements, observed during vibratory compaction. The density of the soil is expressed in terms of initial cone penetration resistance. These settlement values should give a conservative settlement estimate. As the vibration amplitude is usually highest close to the ground surface, settlements will be largest in the upper soil layers, where also the confining stress is low. In many cases, large differential settlements have been observed when part of a building was founded in an excavation and part on the uncompacted fill, removed from the excavation. This uneven foundation situation can lead to relatively large differential settlements below light-weight buildings, even at very low vibration levels (on the order of 1-2 mm/s).

Fig. 14 Settlements caused by vibratory pile driving and vibratory soil compaction

DAMAGE TO BUILDINGS

Damage to structures from ground vibrations is usually attributed to "dynamic effects", such as vibration amplification and resonance effects in structures. Existing codes and regulations are generally empirical and based on observations of damaged structures. The results of such investigations are strongly affected by the local soil conditions and these criteria are therefore difficult to apply elsewhere.

It can be shown both theoretically and by a review of the relevant literature that ground distortion caused by "pseudo-static" ground movements (resulting from the passage of waves below a building) is an important factor which controls building damage, Massarsch and Broms (1991).

Figure 15 illustrates the effect of wave propagation below a building, subjected to surface waves of different wave length. From figure 15 can be concluded that the most critical situation arises when the building length corresponds to about half the length of the propagating wave. The wave length L can be estimated from the following relationship

\[ L = \frac{C}{f} \]  

where C is the wave propagation velocity and f the dominant frequency of the propagating wave. Vibration frequencies on the order of 10 - 30 Hz from waves propagating in soft soils have wave lengths on the order of 5-20 m. These
values are just within the critical range of many residential buildings.

While during static ground deformations, the soil is either distorted by gradual sagging or hogging motion, a large number of upward and downward oscillations can occur during the passage of waves travelling below a building, cf. Fig. 3. Massarsch and Broms (1991) have, based on a review of existing vibration codes in different countries, suggested that damage to buildings caused by "pseudo-static" ground distortions occur at a critical deflection ratio \( d/L = 1.5 \times 10^{-5} \). A simple relationship for the estimation of a critical vibration velocity, \( v \) was obtained

\[
v = 4.7 \times 10^{-5} \cdot C \tag{11}
\]

where \( v \) is the vertical component of ground vibration velocity and \( C \) is the wave propagation velocity in the soil below the structure. This relationship is strictly valid only for the case when the building length corresponds to half the wave length, i.e. for wave propagation in soft soils.

A general solution of the ground distortion problem was proposed by Massarsch and Bodare (1993), which is applicable over a wide range of vibration frequencies and wave propagation velocities. Thus, also the allowable vibration velocity for long wave lengths can be analysed. The general solution shows that the ground vibration velocity \( v \) depends on the wave propagation velocity \( C \) and the two non-dimensional numbers: ground distortion \( (d/L) \) and relative building length \( (B/L) \), c.f. Fig. 16:

\[
v = 2 \cdot \Pi \cdot C \cdot (d/B \cdot (B/L)) / (\cos(\Pi B/L)) \tag{12}
\]

This relationship is shown graphically in Figure 16 for soft soils and in Figure 17 for dense soils and rock. The above relationship can be applied to transient loading and must be corrected for the case of repeated or stationary loading by a factor \( A_1 \). Empirical factors have also been proposed to take into account the building category \( A_2 \) and the degree of acceptable damage \( A_3 \), Massarsch and Broms (1991).

### TABLE 4. Correction factors which take into account the type of vibration source, building type and degree of damage

<table>
<thead>
<tr>
<th>Vibration source</th>
<th>( A_1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impulse</td>
<td>1.0</td>
</tr>
<tr>
<td>Repeated</td>
<td>0.6</td>
</tr>
<tr>
<td>Stationary</td>
<td>0.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Building Category</th>
<th>( A_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very sensitive structures, historic monuments etc.</td>
<td>0.5</td>
</tr>
<tr>
<td>Vibration-sensitive buildings (with masonry walls and plaster), conventional foundations</td>
<td>1.0</td>
</tr>
<tr>
<td>Buildings with good foundations, concrete walls, structures not vibration sensitive</td>
<td>1.5</td>
</tr>
<tr>
<td>Steel or reinforced concrete structures, industrial premises</td>
<td>2.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Degree of Damage</th>
<th>( A_3 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negligible</td>
<td>0.7</td>
</tr>
<tr>
<td>Slight</td>
<td>1.0</td>
</tr>
<tr>
<td>Moderate</td>
<td>2.0</td>
</tr>
<tr>
<td>Severe</td>
<td>4.0</td>
</tr>
</tbody>
</table>
Fig. 16 Critical vibration velocity for typical wave propagation velocities and vibration frequencies in soft soils

Fig. 17 Critical vibration velocity for typical wave propagation velocities and vibration frequencies in stiff soils and rock
GROUND VIBRATION ISOLATION

Three different methods can be used to reduce ground vibrations: 1) restrictions on the source of vibration, 2) screening of wave propagation in the ground or 3) changes of the dynamic properties of the structure. The most effective measure is usually to change the conditions at the source of vibrations (active vibration isolation), e.g. by limiting the speed of vehicles, by modifying the operating frequency of machines or by improving the dynamic response of the foundation of vibrating machines. However, active vibration isolation is not always possible. Another effective and often cheap measure at the planning stage of a project is to increase the distance between the vibration source and the affected structure, cf. Fig. 6.

Vibrations can also be reduced by isolation barriers in the ground (passive vibration isolation). The concept of wave barriers is based on reflection, scattering and diffraction of wave energy. Different types of isolation barriers have been used, such as open or slurry-filled trenches, sheet piles and concrete walls (Barkan, 1962, Woods, 1968, Richart et al, 1970 and Haupt 1980). The isolation effect of a wave barrier depends on the impedance $I$,

$$I = C \cdot \rho$$  \hspace{1cm} (13)

where $\rho$ is the density of the isolation material and $C$ the wave velocity. The energy reflection ratio $R_n$ is a function of the differences of impedance

$$R_n = \frac{(I_2 + I_1)^2}{(I_2 - I_1)^2}$$  \hspace{1cm} (14)

where $I_1$ is the impedance of the soil and $I_2$ is the impedance of the wave barrier. The isolation effect ($R_n$) of a barrier can be estimated from Table 5.

From this relationship it is apparent that a large impedance change between the wave barrier and the soil is required in order to achieve significant isolation. An open trench (with an impedance close to zero) is more effective than a stiff barrier.

<table>
<thead>
<tr>
<th>Impedance Ratio, $I_2 / I_1$</th>
<th>Reflection Ratio, $R_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.00</td>
</tr>
<tr>
<td>0.5</td>
<td>0.11</td>
</tr>
<tr>
<td>0.10</td>
<td>0.67</td>
</tr>
<tr>
<td>0.05</td>
<td>0.81</td>
</tr>
<tr>
<td>0.01</td>
<td>0.96</td>
</tr>
</tbody>
</table>

Massarsch (1986, 1991) described a new type of ground vibration isolation screen, the "gas cushion method". The objective is to create a permanent vertical barrier with low impedance. This is achieved by installing a continuous wall of flexible, gas-inflated cushions. The gas cushion screen consist of horizontally placed flexible tubes, manufactured of a thin-walled plastic laminate. The composition of the laminate assures complete gas tightness and high mechanical as well as chemical resistance (de Cock and Legrand, 1990). The inflation pressure of the gas cushions can be chosen with respect to the depth of installation and is always lower than the ambient ground pressure after installation. Thus, after a slight compression of the cushions, a pressure equilibrium will be achieved between the external earth pressure and the internal gas pressure in the cushions, reducing diffusion rates to very low values.
The gas cushions are installed in a slurry-filled trench, Fig. 18. After installation of the screen in the trench, the bentonite slurry is replaced by a self-hardening cement/bentonite grout, similar to ground water cut-off barriers. The plastic cement-bentonite cake forms a flexible, watertight layer on either side of the gas cushions, providing an additional gas-tight layer on either side. After installation, the surface of the trench above the gas cushion screen must be properly protected by a layer of styrofoam (for temperature isolation of the cushions) and by a surface cover.

The gas cushion screen was used successfully on several projects in Sweden, Belgium and Germany, Massarsch and Erson (1985), Massarsch and Corten (1988), de Cock and Legrand (1992). Figure 19 shows the installation of the gas cushion screen at Düsseldorf, Germany, where ground vibrations from a high-speed railway line caused excessive vibrations in a two-story residential building.

The vibration isolation effect of the gas cushion screen is approximately the same as that of open trenches, de Cock and Legrand (1991, 1992). If the screen extends at least to a depth corresponding to one wave length, then an isolation effect in excess of 50 - 80% of the ground vibration velocity before screening can be expected. It should be pointed out that the geometrical arrangement of the isolation screen is of great practical importance, especially in the case of railway traffic, where the propagating wave field of a passing train can be complex.

The gas cushion screen has the advantage compared to open trenches that after installation, the screen is no longer visible, Fig. 20.

CONCLUSIONS

In the present paper an attempt has been made to describe the process of energy transfer from the vibration source to the surrounding soil. An important aspect, which has not been taken into consideration in the past, is the vibration amplification effect which is caused by the low impedance of piles.
The transfer of vibration energy from a pile to the soil depends on the type of dynamic excitation. In the case of impact loading, the pile is subjected to transient pulses, while during vibratory driving, the pile is kept in a continuous motion, which influences the pattern of energy dissipation at the pile-soil interface.

Wave propagation in the ground is governed by the dynamic soil properties, such as soil damping and wave propagation velocity. A simple chart is proposed which permits the estimation of the attenuation coefficient $\alpha$, based on wave propagation velocity and vibration frequency.

Vibration amplification in the ground as a result of refraction focusing and resonance of soil layers is discussed. A simple solution is proposed for estimating the distance of refraction focusing from the vibration source.

Ground distortion, which is caused by the "pseudo-static" passage of waves below buildings and structures, can cause damage due to repeated sagging and hogging motion. This aspect of wave propagation can be a main reasons for building damage. A simple relationship is proposed by which permissible vibration levels can be determined for transient vibrations. Also the dynamic characteristics of the vibration source, the effect of type of building type and the degree of damage can be accounted for.

Finally, a new method of ground vibration isolation, the gas cushion screen is presented. This passive isolation screen can be installed to great depth in most soils between the vibration source and the affected structure. The expected isolation effect is comparable to that of open trenches.

ACKNOWLEDGEMENTS
The research presented in this report is the result of a team effort by several research groups. Most of the theoretical studies were carried out at the Department of Soil and Rock Mechanics at the Royal Institute of Technology (KTH), Stockholm. The suggestions and valuable contributions by Dr. Anders Bodare throughout this work are especially acknowledged.

Most of the practical development of the gas cushion screen was performed at Franki International and in co-operation with Franki Grundbau, Germany. The contributions of Mr. W. Brieke and Mr. F. de Cock are appreciated.

Research work was sponsored by the Swedish Building Research Council (BFR) and the Belgian Building Research Institute (CSTC).

The permission by Mr. G. Heppel of Müller Bautechnik, Alsfeld Germany and Müller GeoSystems, to publish data recorded by the Müller Process Control System, is gratefully acknowledged.

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5. MISCELLANEOUS

5.1 Technical papers

Leaning tower of Pisa - updated information 1)
M Jamiołkowski, F Levi, R Lancellotta, C Pepe

5.2 Literature and references (not included)

The SPRINT programme Contract RA 216 bis. Quality Assurance in Geotechnical Testing. Phase 2, Recommended Practice in Geotechnical Laboratory Testing.

1) Synopsis and closing remarks included. A copy of the paper can be ordered from the Swedish Geotechnical Institute, S-581 93 Linköping.
SYNOPSIS: The paper is aimed at giving information on the present situation of the Leaning Tower of Pisa and on the activities undertaken for its safeguard by the International Committee appointed in May 1990 by the Italian Government. After a brief review of the subsoil conditions of the structural features and of the observed movements of the Tower, the activities undertaken by the Committee are also summarized.

INTRODUCTION

This paper is aimed at presenting updated information on the Leaning Tower of Pisa. This world-famous curiosity part of the beautiful historical complex of Piazza dei Miracoli in Pisa - see Fig. 1, has been subject, since its erection in 1173, to a progressive tilt reaching nowadays the alarming value of 5°'28"09" (~10%). Such phenomenon has been thoroughly studied by the International Geotechnical Community (e.g. Ministero dei Lavori Pubblici (1971, 1979), Mitchell et al. (1977), Croce et al. (1981), Berardi et al. (1991), AGI (1991), Di Stefano and Viggiani (1992)) but in the last five years great concern has also grown over the structural integrity of the Monument. This problem, rather than the increase of the inclination has prompted the Committee chaired by professors R. Jappelli and P. Pozzati, to close the Tower to visitors in 1989. Following this decision, which caused great sensation, the Italian Government appointed, in October 1990, a 15-member multidisciplinary Commission charged with taking all necessary actions to safeguard the Tower.

In the first part of the paper some hints on the history of the Monuments will be given, thereafter the main issues concerning subsoil conditions, structural features and the observed movements of the Tower will be addressed. In the last part of the paper an attempt will be made to clarify the mechanism at the base of the constant increase of tilting and an account of the work carried out so far by the present Committee will be provided.

CLOSING REMARKS

In the preceding sections of this paper geotechnical and structural aspects of the leaning Tower Pisa as well its movements have been summarized. These latter aspects are recalled in Fig. 26 and Table 3. Fig. 26 reports the evaluation of the Tower settlement using a simple elastic perfectly plastic soil model and geotechnical data that have been gathered since 1989 by Giunta (1988) and Costanzo (1989). The obtained results are in reasonable agreement with those
postulated by Leonards (1979) on the basis of the shape of the settlement bowl encountered at the contact between formations A and B. See Fig. 9. Table 3 shows the evolution of the Tower inclination and of the related overturning moment with time; the displayed data have been high qualitative till 1758, while since the measurements performed by Taylor and Cresy (1829) they reflect in a quantitative manner the evolution of the Tower tilt. As shown in Fig. 25 the inclination of the Tower is growing and the increase is at present around 5 to 6 second of arc' per annum excluding perturbations due to the environment. The upward concavity of the curve representing the rigid tilt versus time can be ascribed to the second order effects.

The phenomenon similar to that which deals in structural mechanics with slender columns recently has recalled the attention of Abghari (1987), Hambly (1990), Cheney et al. (1991) and Lancellotta (1993) in relation to the stability of tall structures seated on soft compressible soils. In case of Pisa Tower the mechanism which might have triggered the initial tilt i.e. the leaning instability should be linked to the inclination which occurred suddenly during the second construction stage, see Fig. 5 and to the subsequent differential settlements, both related, in some manner, to the pronounced spatial variability of the mechanical properties of formation A, e.g. Fig. 8.

With regards to the above mentioned phenomenon a parametric study by Lancellotta (1993) and geometrical analysis by Levi and Lancellotta (1992) have led to a safety factor against overturning of the Tower referred to the actual vertical load of order of 1.09. A more elaborated approach considering the Tower foundation as a rigid disk resting on a discrete support of non-linear springs and dash-pots by Napoli (1992), showed that the increase of tilt required to reach the instability is about 1.5 to 2% of the present inclination.

In view of what above stated, the present Committee appointed to safeguard the leaning Tower of Pisa, considering:

• the high but non quantifiable risk of a structural collapse which is increasing with the increase of inclination,
• the very low safety factor against overturning instability evidenced by progressive increase of rigid tilt at increasing rate,
• the absolute need to avoid that the geotechnical and structural stabilisation works become too intrusive or lead to a heavy visual impact and in order to preserve at any cost the artistic and historical value of the monument and of the whole Piazza dei Miracoli,
• the need, to carry out a series of multidisciplinary studies involving: archaeology, history of medieval arts, architecture geotechnical and structural engineering whose completion requires at least two years,

has resolved upon the strategy hereunder outlined:

• to design and to implement the temporary, and completely reversible, local reinforcement of the most critical cross-section of the structure at the level of the 1st cornice in order to improve the structural safety of the monument. This accomplishment has already been put in place, see Fig. 27. It consists in post-tensioned cables aimed at preventing local buckling in compression of the marble stones forming the external facing;
• to improve the foundation stability against overturning by placing 6 MN lead counterweight of the north rim of the Tower base as shown in Fig. 28. This temporary and reversible intervention will be implemented very gradually in the next future, keeping Tower movements and the possible changes of the pore water pressure in the foundation soil under constant monitoring.

The highly controlled application of the counterweight will hopefully lead to a reduction of the inclination of the Tower by few minutes, producing a situation analogous to that existing 30 to 50 years ago. In addition, the application of the lead counterweight will represent a valuable full scale test of the response of the Tower to the effect of small scale stabilising moment;
• in order to mitigate the possible influence of the subsidence of Piazza dei Miracoli on the present rate of tilt of the Tower, it was resolved to close a number of water wells in the area within 1 km of the Tower. Such decision, although causing some social problem to the municipality, has recently been approved by the Mayor of Pisa and will be enforced in the near future.

After the above mentioned actions which slightly improve the safety of the monument, the Committee has started examining the feasibility of different possible solutions to stop or even to reduce, by no more than one degree, the Tower tilting. Of the different possible approaches, the one leading to a controlled settlement of the ground at the soil-structure interface on the North side of the Tower is being considered by the Committee. If feasible, it will allow, without touching the monument, to stop the increase of inclination and with a reduction of the tilt of order of 30' to 60' to modify positively the state of stress in the critical sections of the structure.

In order to achieve this goal two alternative solutions are taken into account:

• inducing the settlement of the Tower at the North side by causing the reduction of the volume in the top most part of the Pancone clay by means of a properly devised electrosmotic consolidation treatment, see Fig. 29 adapted from Mitchell (1991);

• using the underexcavation technique, to obtain a similar effect, as postulated by Terracina (1962) and as presently used with success to reduce the extremely large differential settlements to which the XVI Century Metropolitan Cathedral of Mexico City has been subject, see Tamez et al. (1992).

The underexcavation technique is envisaged as a very controlled removal of small volumes of soils from the sandy silt formation of horizon A in Fig. 7 beneath the North side of the foundation. A possible scheme of such intervention is shown in Fig. 30.

While the Committee starts trial fields and numerical modelling to ascertain the feasibility of the above mentioned intervention methods, the solution with the ground anchors characterized by a longitudinal deformability as low as possible shown in Fig. 31, is also being developed. It consists in the use of ten ground anchors designed for the working load of 1 MN and connected to the Tower by means of a prestressed concrete ring that is evidenced in the same figure. This intervention is also thought as a very effective means to assure the contact between settling soil and foundation, hence the stability of the Tower when implementing any type of controlled subsidence under the North side.

The above mentioned intervention can be envisaged in relation to one of the following scenarios:

• during implementation of electrosmosis or underexcavation it might result necessary to apply to the North rim of the Tower base a load in addition to the lead counterweight in order to assure the contact between settling soil and the foundation;

• the feasibility of the two above mentioned methods of controlled subsidence will not be demonstrated. In this case the solution with the ground anchors after possible positive response of the Tower to the application of the lead counterweight might become an alternative measure for the permanent stabilisation of the Tower foundation.