Session - 1

Geo-technical and Geophysical Investigations

Evolution of Bridge Foundations for Constructability, Economy, Sustainability and Safety

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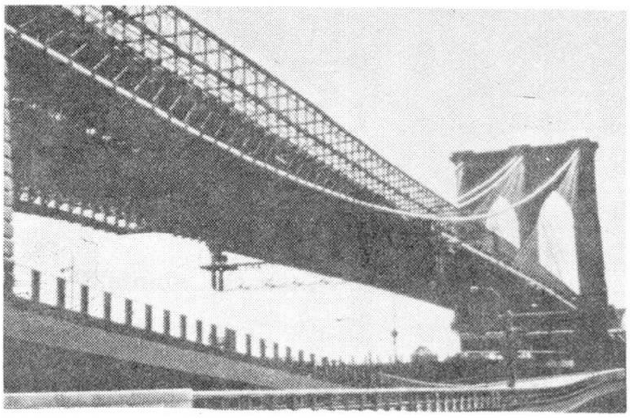
SUMMARY

Bridge foundation has been subjected to significant evolutions from earlier risky techniques with high losses of human lives to modern safe and economical foundation engineering. The use of float-in caissons, large diameter piles in concrete or steel, adapted from the offshore oil industry, and grouting techniques have reduced material quantities, energy consumption and been beneficial to environmental impact. This evolution is described by examples from resent major bridge projects.

1 INTRODUCTION

A bridge component, which has been subjected to significant development and innovation over the last 30 years, is bridge foundation. The foundations were traditionally the most complicated and difficult part to built of any major bridge structure. Unexpected difficulties caused delays, extra costs and sometimes required project changes caused by the need for altered positions. Loss of human lives was normal rather than the exception. The compressed air chamber caissons for the Brooklyn Bridge claimed the lives of many workers at the end of the last century because the unknown effects from compressed air, the bends, were believed to be some sort of a bacterial decease. This technique has also resulted in many accidents and difficulties several times later. It was regularly used, however, up to the mid 1960'es, when new techniques appeared.



The Brooklyn Bridge.

2 NEW TECHNIQUES

Important progress within the offshore industry has led to vast improvements in foundation techniques for bridges over the last 30 years.

Two fundamentally different techniques, inherited from offshore, dominate modern bridge building today:

- Offshore gravity base structure method (GBS).
- Large diameter steel pile foundations, driven to refusal, or large diameter prestressed concrete bored piles.

The first technique was initially developed for offshore oil and gas fields in the North Sea, pioneered by the Ekofisk tank in 1973 for 70 m water depth. The second has been developed for offshore steel jacket structures, initially in the Mexican Gulf, later in the North Sea on medium water depths up to 40 m, and many other places.

Common for these techniques is that they eliminate complicated, weather dependent and sometimes very risky manned operations in the water under pressure. This risky work is substituted by onshore fabrication of reinforced concrete caissons in the GBS case and steel or concrete fabrication onshore, to be installed offshore by floating, and then sinking respective driving or boring techniques with heavy equipment from above water level.

These techniques have proved their reliability in a number of major bridge projects, and have reduced unexpected delays considerably and minimised technical and financial difficulties in the substructure work.

Further, significant progress of analysis and design has made it possible to make use of the potential detailed knowledge of soil structure interaction, including interface behaviour.

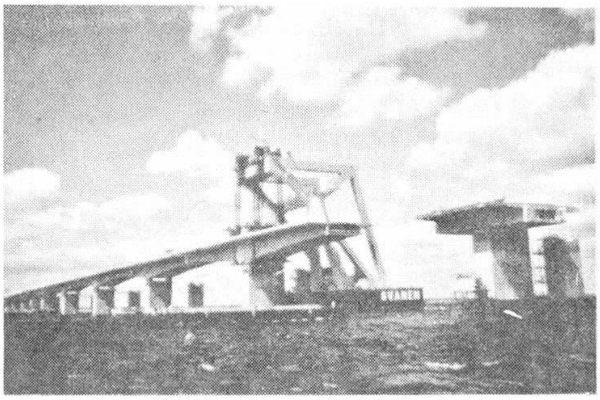
This progress in foundation technologies has, in keeping with time needs of today, considerably reduced quantities of material required for adequate foundations, as well as energy consumption and environmental impact, and thereby contributed to sustainability of modern bridges.

It is often seen that optimum spans for certain applications have been reduced compared to previous more risky and material consuming foundation techniques.

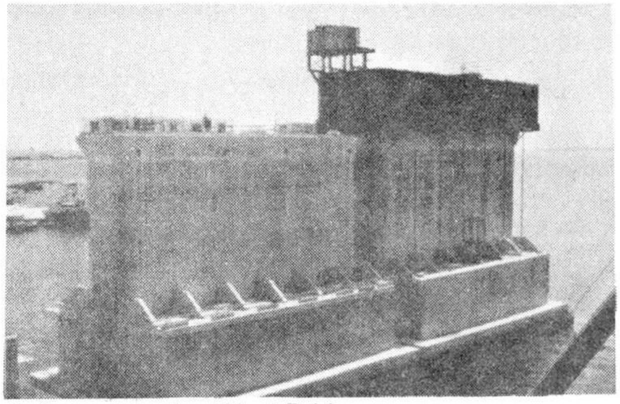
3 THE STOREBÆLT WEST BRIDGE

One of the most recent major bridge projects, where the GBS method has been used to its full potential, is the Storebælt West Bridge, a 6.6 km multi-span bridge, comprising 51 identical spans of 110 m, built as a huge prefabricated building block system.

The bridge contains altogether 324 concrete elements which have been pre-fabricated on shore close to the bridge site and without use of heavy gantry cranes or dry docks. Sliding surfaces were used to move the units.



Installation of girder on the West Bridge.



Caissons for the West Bridge.

The caisson fabrication was carried out parallel in two lines. A base plate was cast first, followed by slipforming of the walls and casting of the cover slabs and plinths. The dimensions were 34 m x 22.5 m maximum and 31.1 m x 6.4 m. Only one slipform was used, suspended from a special gantry and served each line alternatively. A catamaran crane vessel lifted out the caissons from a jetty and transported them to their position in the bridge site.

Each bridge pier is resting on a stonebed which has been constructed after excavation to a prefixed level, determined by detailed site investigations and subsequent cleaning and inspection. The stonebed consists of a lower, well-compacted about 1.1 m, 5-70 mm thick layer, followed by an upper about 0.4 m thick screeding layer with a grain size of 70 mm. During construction, great care was taken to produce a plane surface, and after placing the stones, substantial checks were carried out to ensure that this goal had been met. As the caissons were constructed with a plane base plate, no grouting underneath was carried out.

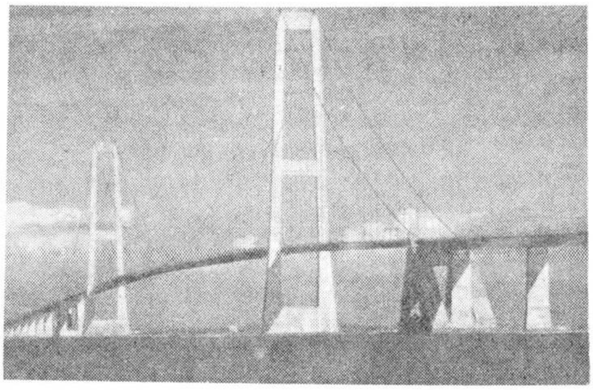
All piers were installed within a few centimetres of accuracy laterally, and vertical settlements have been within a few centimetres as predicted. Further, the method substituted complicated and time-consuming work on the critical path offshore. The West bridge was completed in 1997 and subsequently opened for rail traffic in June. A year later the motorway part was opened.

4 THE STOREBÆLT EAST BRIDGE

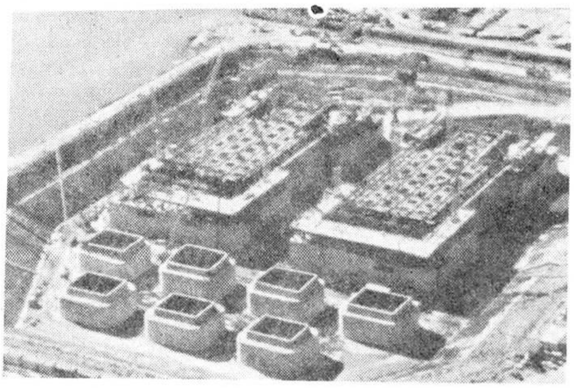
The main bridge across the navigation channel of the Storebælt has similarly employed modern foundation techniques following the success of the West Bridge.

The caissons for the pylons, the anchor blocks and the approach spans were constructed at a prefabrication site about 30 nautical miles from the bridge site. The larger caissons were cast in two dry docks, using the GBS technique, and the smaller caissons on a nearby quay area. The pylon caisson weighed 32,000 tonnes and the anchor block caissons 36,000 tonnes when they were towed by tug boats to their position in the bridge alignment.

The pylon caissons are equipped with 0.5 m high skirts. A very high friction angle for the crushed stone bed materials gave rise to some concern about whether sufficient skirt penetration would be achieved. However, by placing a screed, looser layer at the top of the stone bed, the predicted load penetration response was received with full skirt penetration and the desired base contact prior to grouting of the interface.

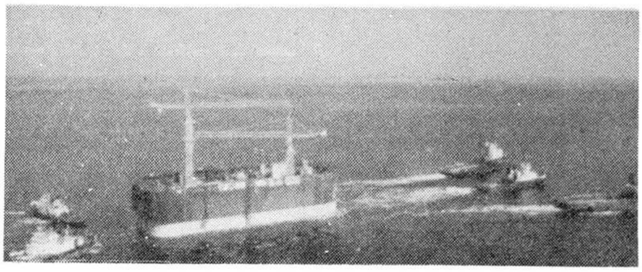


The East Bridge.



Anchor block caissons for the East Bridge.

Each anchor block, which must be able to resist a cable force of 600 MN, has a 121.5 m x 54.5 m rectangular base which is divided into three parts: a front pad of 41.7 m, a middle part of 39.1 m, and a rear pad of 40.7 m. Only the front and the rear pads are in contact with the supporting soils. As a result of excavation, the top part of the clay till was expected to be disturbed and to have a reduced sliding surface. This problem was compensated for by introducing a wedge shaped fill of compacted crushed stone below each of the two pads.



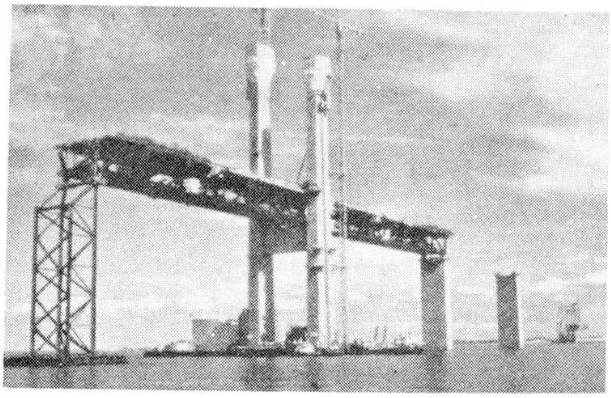
Pylon caisson towed by tug boats.

Settlements and movements after force transfer have been carefully monitored and have been within predicted limits. The bridge was inaugurated in 1998.

5 THE ØRESUND BRIDGE

For the 8 km long Øresund Bridge between Denmark and Sweden, which is presently under construction, the concept has been further refined and optimised. All piers have been prefabricated, including the pylon foundations. All pier foundations have been completed, and the bridge is scheduled for inauguration in year 2000.

The accidental load from ship collision is one of the governing load cases in the design of the pylon foundations for the cable-stayed bridge with has a main span of 490 m. The pylon caissons have a base area of 36m × 38m and are founded directly on Copenhagen limestone at a water depth of 18m. The caissons are buried 2m into the limestone to obtain the necessary horizontal bearing capacity as a combination of base shear and passive resistance in front of the caisson.



The Øresund Bridge.

The quasi-static design ship collision force of 550-600 MN, acting 2m above sea level, was determined from a dynamic global analysis. Several finite element models with advanced elasto-plastic models describing soil behaviour were defined. By means of quasi-static push-over analyses, the horizontal bearing capacity and the associated movements were calculated. These results were compared to the maximum force, and the expected permanent displacement was determined.

The use of an advanced soil model, which was carefully calibrated, made it possible to obtain a very competitive foundation design by avoiding excessive conservatism for this rather unlikely event of collision from a large vessel.

6 THE JAMUNA RIVER BRIDGE, BANGLADESH

The projects mentioned above have all employed derivatives of the GBS offshore foundation technique, because the subsoil condition did not require piled foundation.

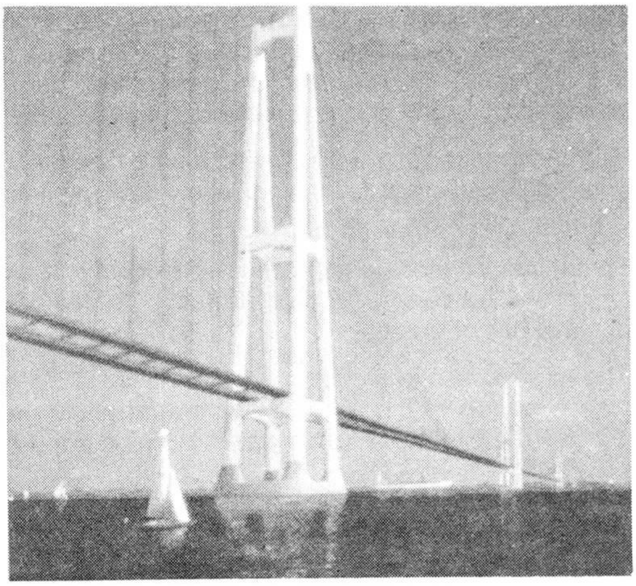
Ground conditions for the Jamuna Bridge in Bangladesh were far so friendly as for the Storebælt and Øresund bridges. The geological conditions at the site consist of about 70 m deep deposits of sands which have a low capacity to resist lateral loads. The bridge is furthermore built in a seismically active zone.

Thus the foundation conditions were extremely difficult, and previous studies in the 1970'es indicated that huge and very deep well caisson foundations would be required. Such foundations would, of course, be extremely costly, and the optimum span would consequently be of a magnitude which would require cable stayed or even suspension bridge spans. The total cost of the project would be considerably beyond a financially feasible level and could therefore not be justified.

During the subsequent evaluations in the World Bank expert panel of the technical and financial feasibility of the project, the experience from large diameter piled foundations of offshore steel jackets were brought in as an element that completely changed the concept and thereby the cost of the bridge.

Substituting huge caissons of concrete to very large depths - more than 80 m - by tripod-like, large diameter steel pile foundations, connected by a pile cap at the bridge deck soffit level, saves the overall per foundation unit costs considerably, whereby the optimum span length decreased to the use of well proven pre-stressed concrete or steel girder techniques. The final optimisation of the project resulted in an overall cost reduction of the bridge project by more than 50% compared to the previous studies using 100 m spans and approximately 3 m diameter steel piles, driven to 60-80 m depth using large capacity offshore pile driving equipment.

At the same time this foundation type reduced the use of resources considerably and was beneficial to environmental impact. The long piles cater for the risk of liquifaction of the uniformly grained, fine sand of the Jamuna River bed during earthquakes as well as allowing for the sand wave risk and very deep scour characteristic of the changing river bed of the Jamuna River bed.

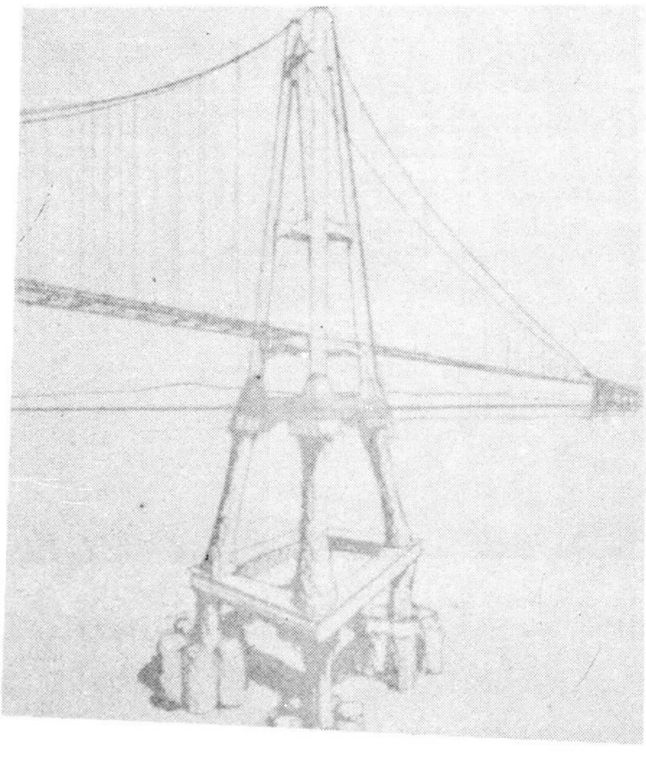


Artist's view of a Gibraltar Strait Bridge.

7 THE GIBRALTAR STRAIT BRIDGE

Since 1981, the United Nations have been interested in investigating the feasibility of a fixed link across the Strait of Gibraltar. The strait is characterised by large water depths, at the narrowest crossing between 500-1000 m, and 250-350 m at a slightly longer alignment along a subsea sill.

The extrapolation of the Ekofisk GBS technique to the Troll Field in Norway with more than 300 m water depth has made it conceivable to build a bridge on fixed foundations across the Strait. Costs of foundations would, of course, be very high as for the offshore platform, and the optimum spans therefore very long. 10 years of studies of alternative configurations have resulted in a technically feasible concept of a series of suspension spans 3500 m each with 450 m tall A-frame pylons founded on a GBS-type prefabricated concrete structure, installed on the sea bottom by towing out and subsequent sinking using skirts and underbase grouting.



Pylon for a Gibraltar Strait Bridge.

The concept would enable initial base construction to take place close to the shore in dry docks, float out and continued construction above water with simultaneous sinking of the caisson as the load increased until completion in a floating condition at an intermediate construction site. After completion the foundation would be towed to position and sunk, where after pylon construction could take place.

The concept being physically transparent in its nature with few cylindrical or conical legs, allows a relatively free and unhindered water flow, thereby minimising wave and current forces as well as reduces the waterforces from earthquakes and minimising environmental impact. The structure would be optimised structurally for minimum material consumption and thereby use of energy and resources.

The Gibraltar Bridge brings to an extreme the huge potential of GBS-based bridge foundations for a case where a bridge on fixed foundations would be unthinkable without such a technique.

8 THE RICHMOND-SAN RAFAEL BRIDGE

Another field within foundation safety is retrofitting. As part of the seismic retrofit of the Richmond-San Raphael Bridge, a concept of using precast concrete jackets to strengthen the existing concrete piers has been developed by Ben C. Gerwick Inc.

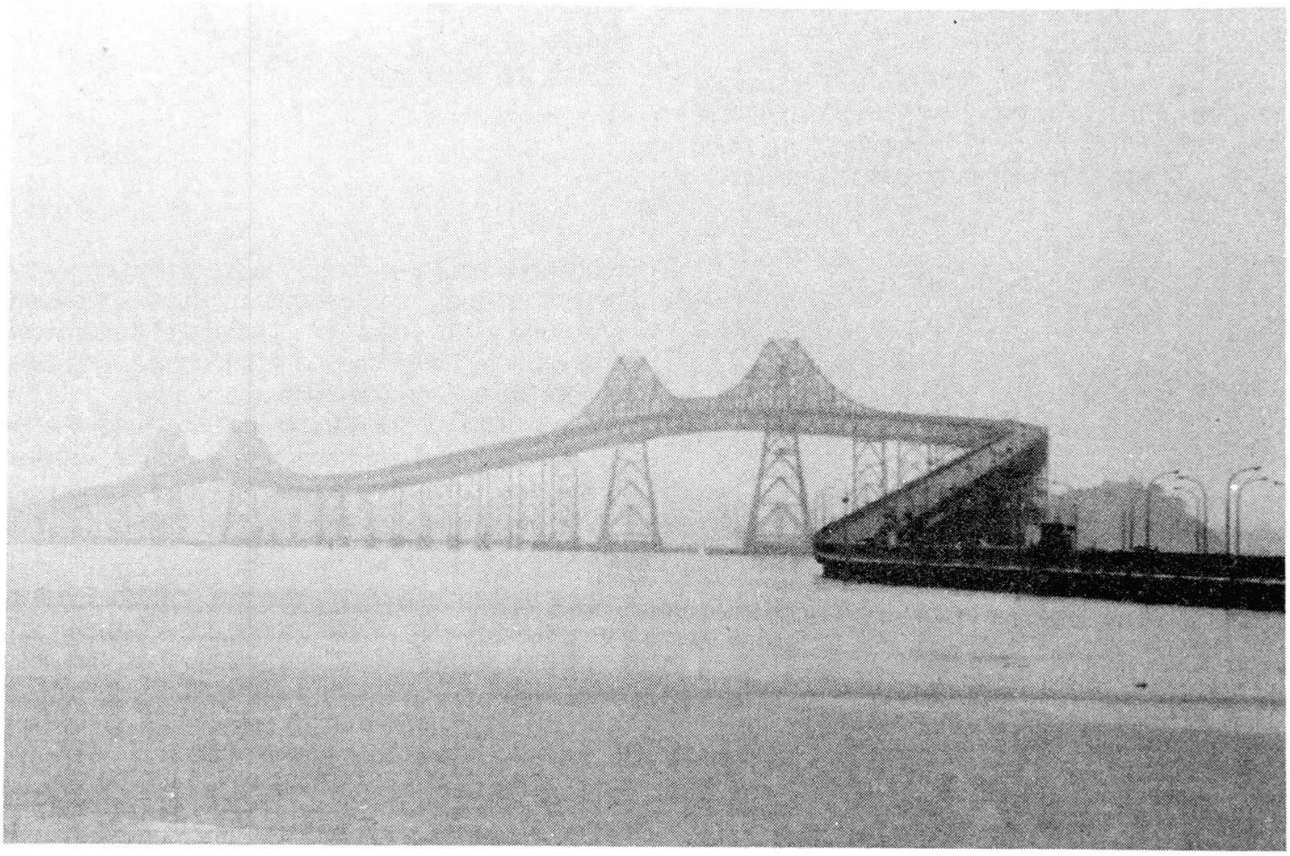
Unlike steel jackets, concrete jackets can be designed to resist corrosion in the aggressive tidal-splash zone for the remaining 100-year life expectancy of the bridge.

A concrete mix with a low water-cement ratio, fly ash, and moderate amounts of an active pozzolan has been specified to allow the jackets to be constructed with a 6.35 cm minimum cover with un-coated reinforcement. In the splash and tidal zone, the concrete will be polyurea-coated which reduces cover impedes micro cracking and reduces the weight of the jackets.

The concrete jackets will be matchcast horizontally and placed around the existing shafts, spandrel beam and diaphragm wall in halves, connected by transverse HS rods which will also connect the precast segments vertically.

Two thirds of the precast concrete jackets will be submerged when placed in their final position. The precast jacket concept allows for a high degree of off-site prefabrication followed by a wet erection with a minimal use of divers.

The existing shafts are cleaned by high-pressure jets before the jackets are placed. An erection frame is placed on top of the existing shafts that allow two precast segments to be placed on each side of the concrete substructure above water. HS rods are used to connect the segments on the outside of the shafts.



The Richmond-San Rafael Bridge.

9 CONCLUSION

The examples illustrate how huge potential in new construction techniques and methods, often generated in other fields, can be employed in bridges with many advantages for the owners, and sometimes even making unprecedented projects feasible.

Application of geophysical techniques for major bridge projects in Denmark

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Henrik Olsen, born 1957, got his M.Sc. and Ph.d. in geology at the University of Copenhagen in 1984 and and 1987, respectively. For ten years he was involved in geological and geophysical research and university teaching. Since 1994 Henrik Olsen has been employed at RAMBØLL in Denmark where he has been involved in several major construction projects.

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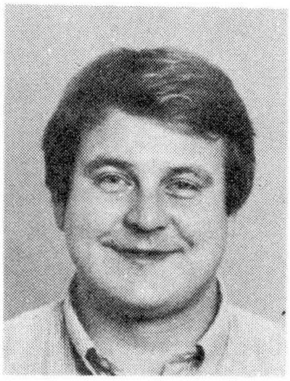

Carsten Ploug, born 1957, got his M.Sc. and Ph.d. in applied geophysics at the Technical University of Denmark in 1983 and 1991, respectively. He has been involved in reflection seismic research for a number of years. Since 1992 Carsten Ploug has been employed at RAMBØLL in Denmark where he has been involved in several major construction projects. Dr. Ploug is head of Geophysical Department.

SUMMARY
Geophysical techniques have been an integratal part of the soil investigation programs for all major bridge construction projects in Denmark during the last two decades. The projects include the 18 km Storebælt Link in central Denmark, the ongoing 15 km Øresund Link between Denmark and Sweden and the planned 17 km Fehmarn Bælt Link between Denmark and Germany. The present paper describes the increasing implementation of geophysical techniques in the feasibility studies and lessons learned in the major projects.

1 INTRODUCTION

Geophysical techniques have been integrated in the feasibility studies for all major bridge construction projects in Denmark during the last two decades, including the Storebælt Link in central Denmark, the Øresund Link between Denmark and Sweden and the planned Fehmarn Bælt Link between Denmark and Germany (Fig. 1).

A range of geophysical metods have been applied to these studies and the experiences from the early projects have been used to further develop the soil investigation programs towards a multiple technique appoach.

The advantage of integrated geophysical investigations is outlined as well as the interpretation approaches applied to the different projects.

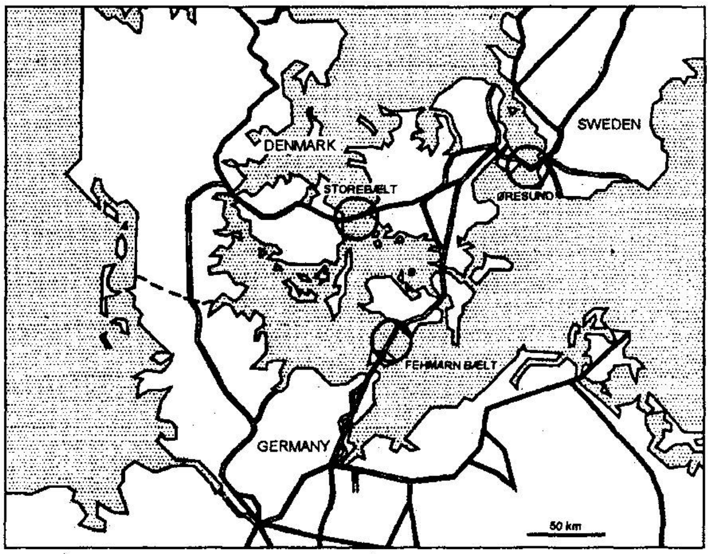

Fig. 1. The locations of the three major infrastructure projects in Denmark: the Storebælt Link, the Øresund Link and the Fehmarn Bælt Link.

2 THE STOREBÆLT LINK

2.1 General

The 18 km fixed link across the Storebælt sound in central Denmark is composed of three main components: 1) The West bridge, a 6.6 km low bridge for rail and road; 2) The East Bridge, a 6.8 km suspension bridge for road; and 3) The East tunnel, a 7.4 km bored tunnel for rail. The construction was completed in June 1998.The geological succession is generally (from the top): Quaternary Post/Late-glacial fines, Quaternary glacial till, Tertiary marl and Tertiary limestone.

2.2 Geophysical investigations

The geophysical methods applied in the feasibility studies were mainly restricted to analoque reflection seismic techniques. A total of more than 3,000 km seismic profiling was carried out by the Danish Geotechnical Institute /1/. At a very early stage, in 1962, reflection seismic investigations were carried out in the entire suggested alignment corridor. The method applied was single channel boomer seismic. In 19977-78 renewed boomer seismic investigations were carried out in the entire corridor with improved technology.

The boomer seismic data from the 60'es and 70'es formed the geophysiscal basis for the realization of the bridge projects commencing in 1986. In the tunnel alignment, however, additional single channel sparker seismic data were acquired in 1987 to obtain information of deeper geological features. The investigations were carried out simultaneously with or subsequent to geotechnical drilling operations.

2.3 The role of geophysics
The reflection seismic data formed a very important input for the 3D geological/geotechnical data base - Geomodel Storebælt /2/ - established especially for the Storebælt project. The early phase in which the seismic data were acquired made it possible to optimize the geotechnical drilling programme for the bridge projects.

In contrast, optimum benefit from the seismic data was not obtained in the tunnel project because the seismic data were not available prior to the drilling operations. Accordingly, the seismic data were not used for identifying possible problem areas for subsequent direct sampling but as means of interpolating the major lithological boundaries between borehole data points /3/. The resulting understanding of the geology was, however, governing for the tunnel alignment and profile.

3 THE ØRESUND LINK

3.1 General
The 15 km fixed link across the Øresund sound between Sweden and Denmark (Fig. 1) is currently under construction. The construc tion will consist of a 3.8 km immersed tunnel in the west, a 7 km suspension bridge in the east and a 4 km low bridge across an artificial island in the central part ofvthe sound. The construction works for the Øresund Link commenced in 1995 and the fixed link iis expected to be completed in the year 2000. The general geological conditions in the Øresund alignment corridor is a Quaternary glacial till succession on top of a Tertiary limestone succession.

3.2 Geophysical investigations
The feasibility studies for the Øresund Link included reflection seismic studies, refraction seismic studies, vertical seismic profiling and wireline/borehole logging.

3.2.1 Reflection seismic
Reflection seismic investigations were carried out in 1993-1995 in the entire alignment corridor and in selected areas of special interest. The investigations were performed by DGI and comprised single channel boomer as well as six channel water gun seismics /4/. In contrast to the Storebælt investigations, the acquisition in Øresund were digital, providing basis for postprocessing. The seismic data were interpreted mainly in order to provide a structural geological model for the limestone succession.

3.2.2 Refraction seismic
Refraction seismic investigations were carried out by Geomap in 1992-93 along the western part of the alignment /5/. The seismic data were interpreted in order to provide information about the velocities of the geological layers and to define possible glacially disturbed limestone successions in the upper part of the limestone.

3.2.3 Vertical seismic profiling (VSP)
VSP investigations were carried out by RAMBØLL in 1993-1994 along the entire alignment /6/. VSP was carried out in boreholes by use of a hydrophone streamer with 12 hydrophones. Vertical data point spacing was 0.5 m. The seismic data were interpreted in order to define the seismic velocities of the geological units.

3.2.4 Borehole logging
Borehole logging was carried out in 1992-1994 by RAMBØLL in boreholes along the entire alignment and in boreholes situated in areas of special interest. The borehole logging programme included a large suite of advanced logging techniques, including: natural gamma, induction conductivity, guard resistivity, neutron porosity, gamma density, sonic, fluid conductivity and fluid flow. The logs were interpreted in order to establish a detailed log stratigraphy and to provide information of physical properties and ground water flow characteristics of the limestone /6, 7/.

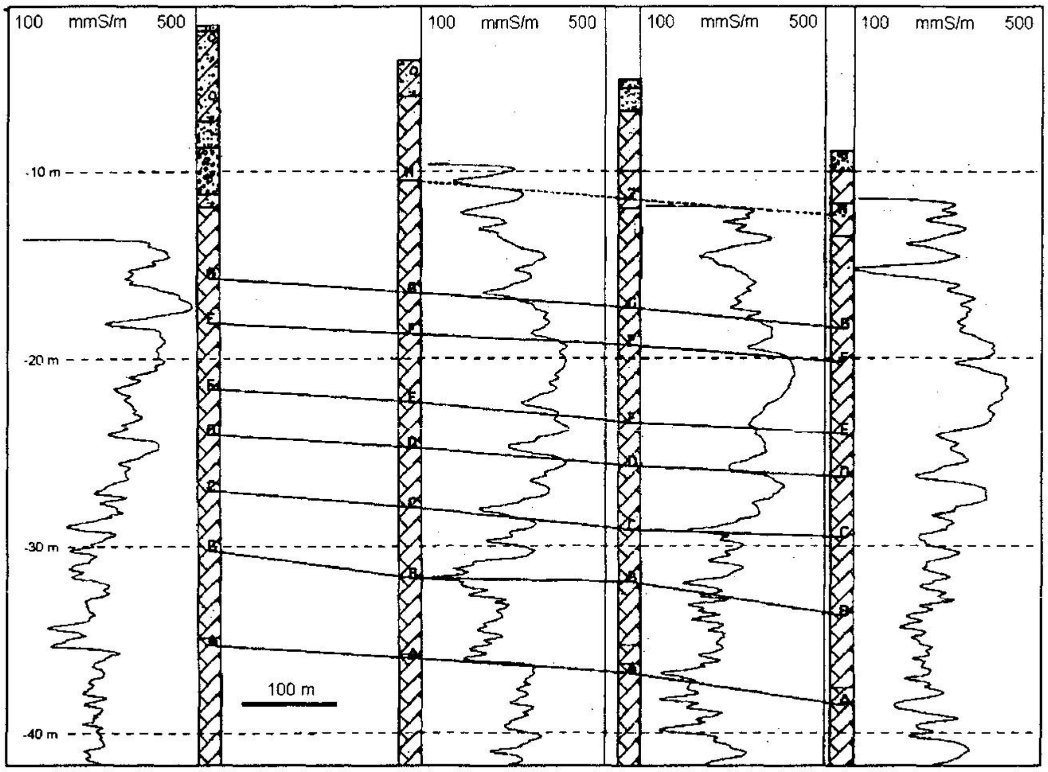

Fig. 2. Stratigraphic correlation of the limestone in four boreholes in the western part of Øresund using the information from induction conductivity logs. Modified from /7/.

3.3 The role of geophysics

The combination of a multiplum of geophysical investigation techniques proved very useful for the Øresund Link project. Although the importance of the different methods was variable the suite of methods provided complementary information, impossible to attain by means of only a single method.

3.3.1 Reflection seismic

Prior to the seismic investigations depth variations in marker horizons in the limestone were ascribed to faulting. The seismic data revealed, however, that these depth variations could be explained by folding.

The boomer data proved very useful. The water gun data were expected to provide data from significantly gtreater depths than the boomer data. The water gun data did not, however, provide significant improvement because only limited attenuation of the seabed multiple was possible /4/. The reason for this is believed to be inadequate move-out correction, mainly due to the limited number of hydrophone groups and limited length of hydrophone spread.

The collection of digital data also proved a success. In the construction phase reprocessing of the data could be performed providing additional details of great use for the project.

3.3.2 Refraction seismic

The refraction seismic data were used for defining the extent of glacially disturbed limestone. The results were not, however, equivocal. For definition of deeper geological successions the method was inadequate, because an internal low velocity layer prohibited precise information for deeper lying successions.

3.3.3 Vertical seismic profiling

The VSP data were very useful. The limestone succession was subdivided in successions with characteristic seismic velocities, indicating geotechnical bulk characteristics. The VSP data were also useful for depth conversion of the reflection seismic data so that correlation with borehole data was possible.

3.3.4 Borehole logging
A detailed metre scale stratigraphic subdivision of the limestone succession was defined on the basis of the borehole logs (Fig. 2). The log stratigraphy proved very useful in veryfying the structural geological model.

The logs also provided important information concerning the ground water flow characteristics. Distinct inflow horizons were observed in certain stratigraphic successions indicating that fractures act as hydraulic corridors.

Physical properties of the limestone rocks were accounted for on a decimetre scale, providing important information for the prediction of geotechnical properties. As a consequence, it was decided to use borehole logs also during the construction phase for detailed site investigations for all bridge piers.

4 THE FEHMARN BÆLT LINK

4.1 General
Feasibility studies are currently being carried out for the planned 17 km Fehmarn Bælt Link between Fehmarn in Germany and and Lolland in Denmark (Fig. 1). At present the link has not been politically decided. The general geological succession in the Fehmarn Bælt include (from the top): Quaternary Late/Postglacial fines, Quaternary glacial till, Tertiary clay and Cretaceous limestone.

4.2 Geophysical investigations
In 1995 geophysical investigations were carried out as part of a geological/geotechnical feasibility study /8/. The investigations included shallow reflection seismic surveys applying pinger and sparker, and deep reflection seismic applying air gun and a 24 channel hydrophone array. VSP was obtained from the boreholes drilled during the feasibility study. A suite of borehole logs comparable to the Øresund Link investigations was also applied. The shallow reflection seismic investigations were carried out by DGI whereas RAMBØLL performed all other geophysical investigations.

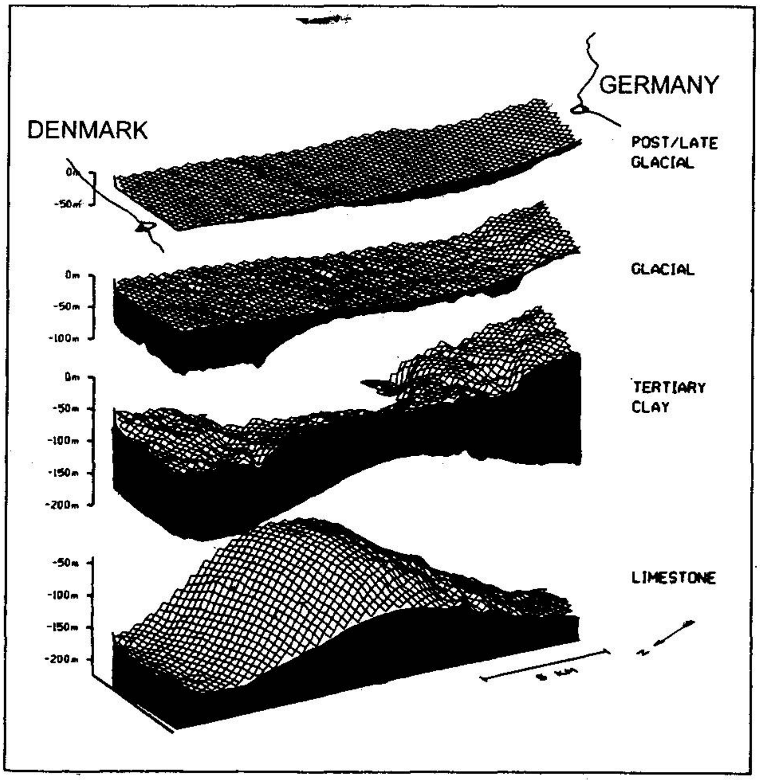

Fig. 3. Geological model from Fehmarn Bælt based on reflection seismic data. High resolution was obtained in the Quaternary Post/Late-glacial and glacial deposits by single channel shallow reflection seismic methods. The deeper lying Tertiary clay and Cretaceous limestone units were excellently outlined by the multichannel seismic method. Modified from /8/.

4.3 The role of geophysics

The shallow reflection seismic provided detailed information of the geological strata in the upper tens of metres. The experiences from the Øresund investigations were used to design a multichannel seismic setup capable of suppressing the multiple reflection and enhancing the geological stratification in deeper levels. Accordingly, the multichannel seismic data made it possible to improve the structural geological model significantly. Significantly the deeper seismic data showed the existance of a domal shaped limestone surface, interpreted to be a result of salt upheaval in deeper levels. In addition the presence of structural deformations could be outlined on the basis of the deeper seismic data. The VSP was primarily used for depth conversion of the reflection seismic data. The borehole logging formed an important supplement to the drill core data and an initial basis for log stratigraphic subdivision.

5 LESSONS LEARNED

The lessons learned during the three major construction projects can be summarized as follows:

1) *The right method at the right time.*
Geophysical investigations shall be performed at an early stage to form a guidance for drilling operations so that the drilling sites and numbers can be optimized. Moreover, careful evaluation of the techniques shall be made as to the effectiveness of the technique for the particular geological conditions. Courage and visions of the owners resulted in introduction of new methods hitherto not used for this kind of projects.

2) *Using the experiences from previous projects*
By evaluating the experiences from previous bridge construction projects it was possible to enhance the profits of the geophysical techniques.

3) *The advantage of integral geophysical investigations*
By combining different geophysical methods it has been possible to obtain complementary data which greatly enhance the value of the individual techniques.

6 ACKNOWLEDGEMENTS

The authors gratefully acknowledge the permissions from A/S Storebælt, Øresundkonsortiet and Trafikministeriet to publish this paper. The opinions expressed in the paper do not necessarily reflect the opinion of the three institutions.

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Rapid Investigation Methods of Soil Properties and Interpretation of their Results for Bridge Foundations Design

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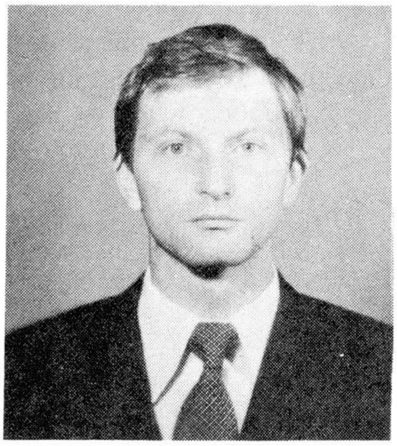

Yuriy Vynnykov, born 1963, got his civil engineering education in Poltava STU. Author of more than 100 scientific works, including books, normative documents and more than 30 inventions. Specialist in research, designing, construction and reconstruction of basements and foundations constructed without earth excavation, including piled foundations for bridges, mathematical simulation of behaviour of such foundations, anisotropic soil properties.

SUMMARY

For solution of geotechnical problems while designing and reconstruction bridge foundations it is reasonable to use rapid methods of field and laboratory investigations of soil properties: penetration, static probing, rotary shear, etc. Static soil penetration with widened tip comparing with traditional method of static penetration with the same rod and tip diameter that allows to use static effort more effectively due to elimination the friction from rod surface and the use of large size tips allows to increase accuracy of determination of poor bearing soils values. The depth of such penetration exceeds 20 m. For determination of soil characteristics in each point of soil massif according to the data of rapid methods of soil investigation the equations of interconnection between their physical and mechanical properties. It allows to set soil characteristics in each element of calculated field of bridge foundation basement with numerical simulation or analytical methods of their calculation quickly and objectively. Numerical simulation of stress-strain state of bridge foundation bedding is carried out with the use of programme of elastoplastic calculation of geotechnic structures based on the method of initial stress in combination with the method of ultimate elements.

1 INTRODUCTION

Main points in process of foundation designing are engineering geological survey, preparation of soil thickness parameters and evaluation of stressed-deformation condition (SDC) of basements and foundations. Speedy methods of soil properties investigation were tested for that purpose, results of which were interpreted for evaluation of SDC basements, foundations and pile bearing capacity with help of interconnection equations between physical and mechanical soil properties.

2 RAPID INVESTIGATION METHODS OF SOIL PROPERTIES

2.1 Static soil penetration with widened tip

Static soil penetration with widened tip (fig.1,a) with 30^0 angle at the top and the diameter exceeding rod diameter 1.6 times and more comparing with traditional method of static penetration with the same rod and tip diameter allows to use static effort more effectively due to elimination of friction from the rod surface (because conditions for pressing soil into formed cavities between the well and stem walls appear), and the use of large size tips allows to increase accuracy of determination of poor bearing soils values for which this only method is very often available. The depth of such penetration exceeds 20 m. For creation of conditions for friction of soil layer by layer during penetration the authors have investigated and patented several modifications of widened tip, namely: tip like a cone with cylindrical steps{1} (fig.1,b); tip like a cone with cylindrical steps which are connected telescopically{2} (fig.1,c); tip with spiral groove beyond the cone surface {3} (fig.1,d). The size of the tip can be changed in the process of penetration {4} (fig.1,e). It makes it possible to determine precisely soil strength characteristics which lay along the whole depth of penetration despite the difference of this characteristics. For combination penetration processes and selection of soil samples the tip {5} (fig.1,f) that has telescopic cylinders with lugs has been worked out.

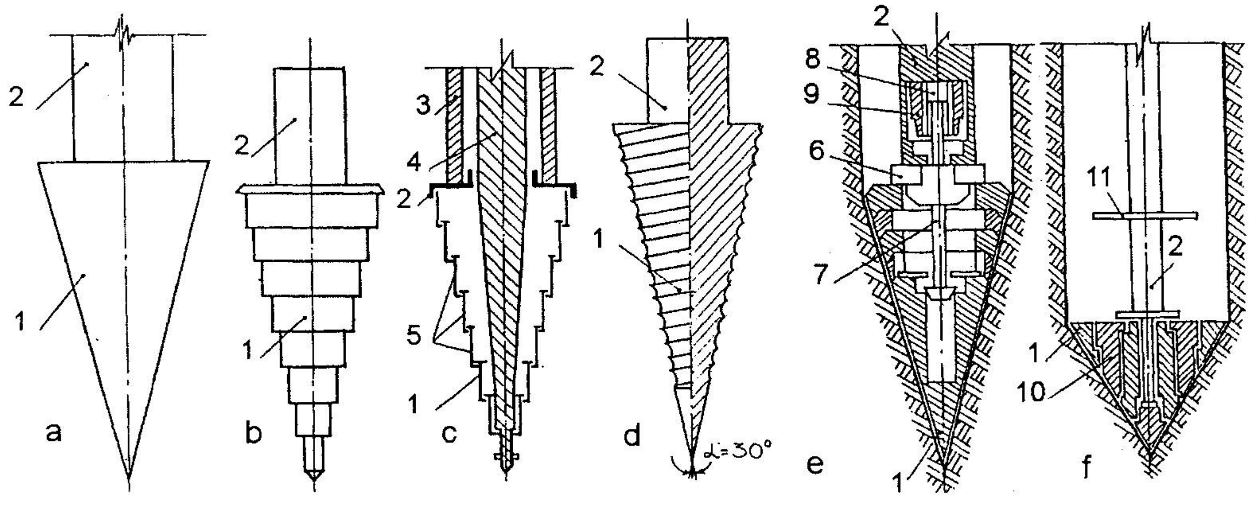

Fig. 1 Types of the widened tip: 1- cone; 2- tang; 3- bar; 4- stock; 5- cylindrical bucket; 6- figure lug; 7- rod; 8-screw; 9- nut; 10- cylinder; 11- banked-up ring

2.2 Method of penetration

Penetration method is based on slow submersion of tip cone into the depth h that mustn't exceed the cone height. For cohesive soils the investigation penetration characteristic is ratio of penetration effort P to square of cone submersion depth and it is called the unit penetration resistance R, Pa

$$R = P / h^2 . \quad (1)$$

For non-cohesive soils the so called penetration index U (H/sm^3) is used

$$U = P / h^3 . \quad (2)$$

Principle of the invariable of test penetration results provides possibility for objective control of precision and truth in definition penetration indices of mechanical soil properties. Results of penetration test present unique indices of resistance to soil shear. According to the results of penetration tests it is determined the angle of internal friction φ and unit cohesion c of non-cohesive soils. For investigation of soils with anisotropic properties it was also worked out a tip shaped like tetrahedron pyramid, opposite facets are symmetrical, side facets are concave and working ones are flat with 90° angles at the top and between working facets angle is 10° {6} (fig.2). Due to such form its interaction with soil occurs only beyond its working facets.

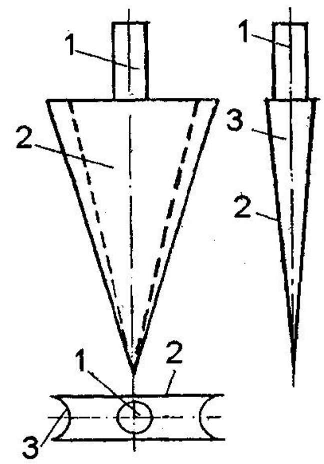

Fig. 2 Tip for penetration of soils with anisotropic properties: 1- tang; 2- working facets; 3- side facets

2.3 Method of rotary shear

Method of rotary shear consists of slow pressing the tip with two perpendicular vanes into soil and measuring the rotary moment at vane turn M_{max}. At laboratories, usually, combination of soil investigation by penetration and rotary shear are used. In a number of cases they help simply and exactly to determine the angle of internal friction φ and unit cohesion c of cohesive soils. The unit resistance to rotary shear τ is determined by ratio

$$\tau = M_{max} / K_\tau , \qquad (3)$$

where K_τ is a constant of vane-shear tip which is received from formula

$$K_\tau = 0.5\pi d_{cr}^2 (d_{cr} / 6 + h_{cr}) , \qquad (4)$$

where d_{cr} and h_{cr} - diameter and height of the vane. For soft clayey soils and silt where the angle of internal friction is small it is $\tau = c$.

3 INTERPRETATION OF RESULTS OF SOIL RAPID INVESTIGATION METHODS

3.1 Equations of interconnection between physical and mechanical soil properties

Objective characteristics of above given methods are received due to traditional three-term formula of limited basement condition. At the same time, the peculiarities of characteristics for cohesive soils and non-cohesive differ. Theoretical statement is well confirmed by experimental investigations. For determination of soil characteristics in each point of soil massif according to data of rapid methods of soil investigation the equations of interconnection between their physical and mechanical properties are used, for example: humidity W, void ratio e, unit cone resistance R, angle of internal friction φ, unit cohesion c, modulus of deformation E etc. Practically, while determining correlation between physical and mechanical properties in conditions of three - phase state of natural structure soil it is necessary to determine its three indications: free term and two angle coefficients of conditional linear equations. The general correlation equation in this case is:

$$\lg(R / R_0) = W_R(1/e_0) + (\rho_W / \rho_s)(1 - M_{kpf}) / (1/e_0) - WM_{kpf}(1/e_0) - (\rho_W / \rho_d)(1 - M_{kpf})(1/e_0), \qquad (5)$$

where R - unit cone resistance, MPa; R_0=1 MPa; $M_{kpf}=1-(1/e_0)/(1/e)$; $1/e_0$ and $1/e$ -angle coefficients of linear equations for the case of total soil saturation and in condition of constant humidity accordingly; W_R - moisture content in complete saturation with R_0= 1.0 MPa.

Multiple investigations showed that while determining correlation between the properties of clayey soils the indicative values of equation (5) are influenced by such properties as plasticity index, mineralogical composition of clayey ingredient, grain size distribution and mineralogy coarse-graded component of soil. The condition for determination of correlation between indices of physical state of soil (W, e) and indices of mechanical properties (R, φ, c, E) is the accumulation of test results for determination of these soil characteristics with relatively constant plasticity index and uniform genetically. Thus, for definite soils we can set a number of dependencies:

$$\lg(R/R_0)=A_R-B_Re-C_RW; \tag{6}$$

$$\lg(E/E_0)=A_E-B_Ee-C_EW; \tag{7}$$

$$\lg(c/c_0)=A_c-B_ce-C_cW; \tag{8}$$

$$\lg(tg\varphi/tg\varphi_0)=A_\varphi-B_\varphi e-C_\varphi W; \tag{9}$$

where $R_0; E_0; c_0; tg\varphi_0$ are values that equal the unity in the taken units; A, B, C - coefficients are functions of indicative soil features: $1/e_0$, $1/e$, W_R.

According to these equations we have the following:

$$\lg(E/E_o)=A_E-(B_E/B_R)A_R-W[C_E-(B_E/B_R)C_R]+(B_E/B_R)\lg(R/R_0); \tag{10}$$

$$\lg(c/c_o)=A_c-(B_c/B_R)A_R-W[C_c-(B_c/B_R)C_R]+(B_c/B_R)\lg(R/R_0); \tag{11}$$

$$\lg(tg\varphi/tg\varphi_o)=A_\varphi-(B_\varphi/B_R)A_R-W[C_\varphi-(B_\varphi/B_R)C_R]+(B_\varphi/B_R)\lg(R/R_0). \tag{12}$$

With the help of equations (10-12), having indices of soil natural humidity W and unit cone resistance R in any point of footing it is possible to determine mechanical soil characteristics. Equations (6-12) were determined for soils at several dozens of sites during field and laboratory works. They give opportunity to determine mechanical characteristics of natural structure soil and within the compaction zones while calculating and modeling work of bridge basements and foundations. It allows to set quickly and objectively soil characteristics in each element of calculated field of bridge foundation basement with numerical simulation or analytical methods of their calculation

3.2 Determination of pile bearing capacity due to results of soil static penetration

We should point out that technique of determination of bearing capacity of pile bridge foundations due to results of soil static penetration by widened tip was successfully conducted.

3.2.1 Bearing capacity of driven prismatic piles

Bearing capacity of driven prismatic piles is:

$$F_d=\theta q_S^0 A\lambda_0+0.0117u\sum_{i=1}^{n} q_S^i\lambda_i h_i+6u\sum_{i=1}^{n} h_i\ , \tag{13}$$

where θ - coefficient that takes into account deformation character of pile base; $q_S^o; q_S^i$ - resistance to soil cone in the plane of pile edge and of each layer along the whole length of pile, kPa; A - pile cross section area, m^2; $\lambda_0; \lambda_i$ - transition coefficients from q_S to calculated soil resistance; u - perimeter of pile, m; h_i - thickness of bearing layer within depth of pile submersion; n - number of bearing layers of soil along pile length.

In fig.3 there are diagrams of dependence coefficient λ on value q_s and soil type. Diagrams are based on results of parallel test of soil by penetration and static pile tests carried out in the central part of Ukraine.

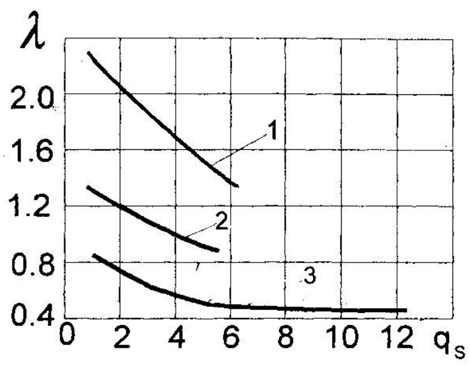

Fig.3 Dependence curves $\lambda = f(q_s)$:
1- loam and clays; 2- loamy sands; 3- sands

3.2.2 Bearing capacity of cast-in-situ piles in punched hole (CPPH)
For determination of bearing capacity of CPPH according to results of static soil penetration the calculation scheme was accepted according to which soil resistance along the side surface of the shaft is taken into account only at a point h from its top till the point of intersection of shaft with surface of conditional cone having the forming line tangential to the edge of widening at angle $\varphi/4$ to pile axis, where φ is average arithmetical value of internal soil friction angle laying within limits of the given cone. We offer to determine bearing capacity of CPPH on the base of static penetration by formula

$$F_d = \theta q_s^0 A_{cr} \lambda_0 + 0.0117u \sum_{i=1}^{n} \theta_{si} q_s^i \lambda_i h_i + 6u \sum_{i=1}^{n} \theta_{si} h_i \ , \qquad (14)$$

where

$$\theta = 1005/(4213 + q_s^o) \ , \qquad (15)$$

$$\theta_{si} = 4328/(3850 + q_s^i) \ , \qquad (16)$$

$q_s^o; q_s^i$ - resistance of soil to cone on the level of bottom of widening and for each layer within point h, kPa, correspondingly; $\lambda_0; \lambda_i$ - transition coefficients for each soil layer on level of widening and within limits of h; u is perimeter cross section of shaft, m; h_i - strength of each soil layer within point h, m.
3.2.3 Bearing capacity of tapered piles
Bearing capacity of tapered piles equals

$$F_d = kmq_s A \ , \qquad (17)$$

where k is coefficient of soil uniformity; m - scale coefficient m=0.5; q_s - average value of soil resistance under the tip; A - area of cross section of tapered pile on top section, m^2.

4 NUMERICAL SIMULATION OF STRESSED-DEFORMATION CONDITION OF BRIDGE FOUNDDATION BASEMENTS

Numerical simulation of stress-strain state of bridge foundation bedding is carried out with use of the programme of elastoplastic calculation of geotechnic structures based on method of initial stress in combination with method of ultimate elements and theoretic - mathematical description of soil as continuous isotropic media modeled in accordance with theory of plastic flow.

4.1 Hypotheses in statement of elastoplastic task

In statement of elastoplastic task such hypotheses are used: 1)evidence of non-linearness includes plastic deformation of form change in complicated stressed state and free deformation in tension; 2)in complicated stressed state (compression with shear) general deformations include linear and plastic parts, plastic components of deformation appear after reach of strength limit according Mises-Shleikher-Botkin:

$$\alpha I_1 + I_2^{0.5} - K = 0 \ , \qquad (18)$$

where $I_1 (I_2)$ is first (second) invariant of stress gauge (deviator); α and K - characteristics of soil strength in space stressed state; 3)vectors of main plastic deformations (and their velocities) and main stresses in complicated stressed state are taken as co-axial; 4)at the stage of plastic deformation was taken into account dilatancy was taken into account with correlation between speed of volume change and form change, which is expressed as the constant (non-associated law) in space stressed state:

$$\Lambda = \ I'_1 / 6I'_2 \ , \qquad (19)$$

where I'_1 ,I'_2 are speeds of main stresses.

4.2 Calculated sphere of foundations and piles

Calculated sphere of axis symmetric task is cylinder, it is based on revolution of rectangular calculated zone around symmetry axis (fig.4). Symmetry axis coincides with foundation axis. In calculations the continual space elements of triangular section which simulate foundation material and soil of bedding are used. Properties of bedding are defined by real soil characteristics: unit weight γ , modulus of deformation E , coefficient of side pressure ν, angle of internal friction φ, unit cohesion c, parameter dilatancy. Physical and mechanical soil characteristics within calculated zone are determined according to the results of penetration and revolving shear and with help of equation of interconnection (10-12).

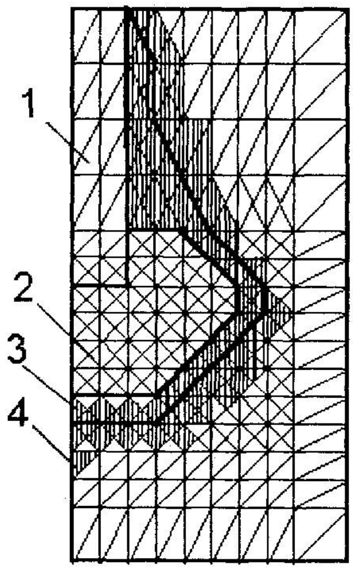

Fig.4 This is an example of the calculated sphere of cast-in-situ piles in punched hole during of numerical modeling its work: 1- shaft; 2- hard widening; 3- compaction zone; 4- zones of plastic deformations

5 SUMMARY

Investigations of soil properties by static probing with the widened tip, penetration, rotary shear with following interpretation of results for designing and bridge foundations considerably allow to speed up process of designing with great extent of reliability.

REFERENCE

1 AUTHOR CERT. №476356 USSR. IPC E 02d 1/00. Tip for soil penetration
2 AUTHOR CERT. №582478 USSR. IPC G 01N 3/42. Tip for soil penetration
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4 AUTHOR CERT. №476355 USSR. IPC E 02d 1/00. Tip for soil penetration
5 AUTHOR CERT. №1744194 USSR. IPC E 02D 1/00. Tip for soil penetration
6 PATENT of Ukraine №17737. IPC E 02D 1/00. Tip for penetration of soils with anisotropic properties.

Geotechnical design considerations for Storebælt East Bridge and Øresund Bridge

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Mr. Sørensen has been involved in research, teaching and consulting projects for more than 20 years. The work has involved field and laboratory testing of soft and stiff soils as well as design of large bridges and offshore foundations. Recently he has been in charge of the foundation of the Øresund Bridge. He is chairman of the committee responsible for the Danish Code of Practice for Foundation Engineering.

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Dr. Hededal has extensive experience with numerical analysis using the finite element method. In particular, Dr. Hededal has been involved in development of several linear and non-linear finite element codes for research and educational purposes. He has recently been involved in the design of the Øresund Bridge and in the design evaluation of the Metro in Copenhagen.

SUMMARY

The fixed links crossing the 18 km wide Storebælt and the 15 km wide Øresund are the first two of three major infrastructure projects – Storebælt Link, Øresund Link and Femern Bælt Link – that will eventually link Denmark internally and externally to its neighbours. By their extent and complexity, these projects have been a great challenge to all engineering disciplines. The paper describes the investigation strategies employed and the geotechnical design of the bridge foundations. In both projects it was necessary to combine several theoretical models in order to meet the design requirements. Some of the challenges offered by such large projects will be illustrated by two examples: 1) Investigations for and design of anchor blocks founded on gravel wedges and clay till for the Storebælt East Bridge and 2) Investigations for and design of heavily ship impact loaded piers founded on limestone for the Øresund Bridge.

1 INTRODUCTION

This paper presents the Storebælt East Bridge and the Øresund Bridge from a geotechnical point of view. For both projects comprehensive ground investigation programs were carried out in order to establish the geotechnical basis. The strategies for and analyses of the tests were different for the two projects even though very similar analysis methods were used. The aim of this paper is primarily to demonstrate the interaction between the ground investigation program and the development of the analysis models. Finally, the analysis models and the safety strategies for two of the critical foundation problems are considered.

1.1 The Storebælt East Bridge

The East Bridge (established 1986-1997) is one of the major components of the fixed link across Storebælt. The 6.8 km long motorway suspension bridge includes a main span of 1624 m with a navigational clearance of 65 m, see Fig. 1.

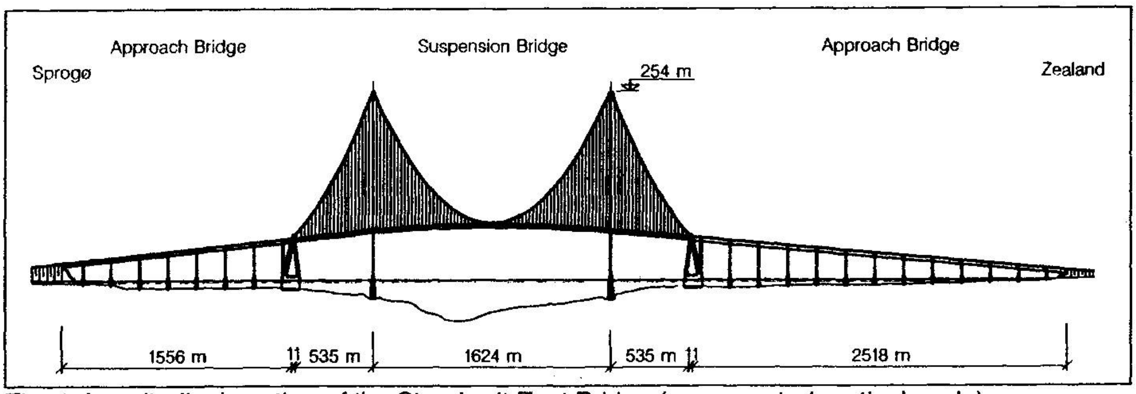

Fig. 1: Longitudinal section of the Storebælt East Bridge (exaggerated vertical scale)

The foundation conditions were good, consisting of firm clay till on top of Kerteminde Marl and Danian Limestone. A comprehensive geological description is given by e.g. Foged et al. [4].

Based on the West Bridge experiences, it was decided that all piers, pylons and anchor blocks should rest on caissons placed on compacted gravel pads of crushed rock. All caissons should be manufactured onshore with a skirt system designed for penetration into the gravel pad in order to establish an ambient space for grouting below the caisson base slab. Special attention was given to the design of the pads beneath the anchor blocks, cf. Section 2.1.

1.2 The Øresund Bridge

The Øresund Bridge is part of the Øresund Link (established 1995-2000) linking Denmark and Sweden. The fixed link will carry rail and road traffic. It comprises a 3510 m long immersed tunnel, a 4055 m long artificial island and a high bridge and approach bridges of a total length of 7810 m. The cable stayed high bridge has a free span of 490 m with a navigational clearance of 57 m. A section of the Øresund Bridge appears from Fig. 2.

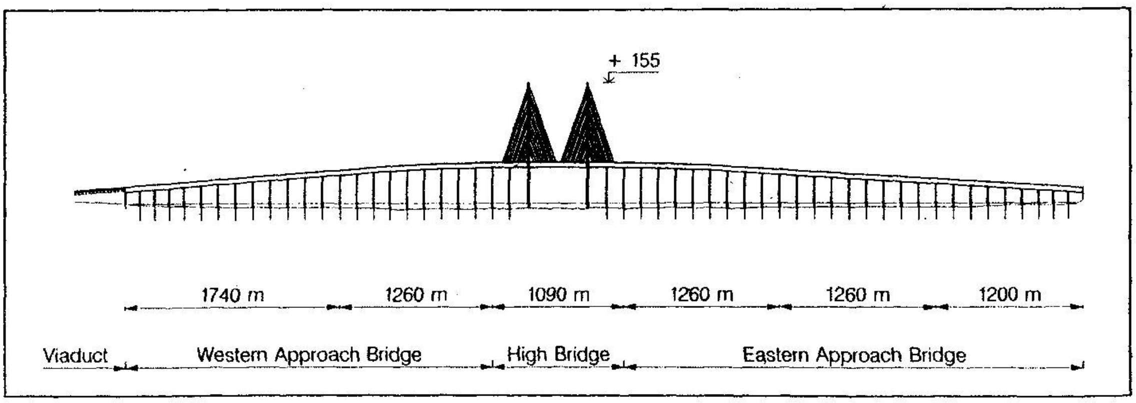

Fig. 2: Longitudinal section of the Øresund Bridge (exaggerated vertical scale)

The foundation conditions in Øresund are dominated by the Copenhagen Limestone, see e.g. [7]. The limestone is generally a competent foundation base, but regions with unlithified limestone or fissures could give problems. Furthermore, it is vital to consider the effects of a reduction of the limestone strength at large strains, [2]. The ground investigations therefore aimed at identifying potential problem zones.

Shallow direct foundation was found to be the most economical foundation due to the competent limestone in Øresund. It was decided that the Øresund Bridge piers should rest on a thin cement grout layer cast after the pier caisson foundations were installed. To secure proper grouting conditions the caissons were temporarily placed on three small footings and a temporary curtain was mounted on the sides of the footings.

2 GEOTECHNICAL INVESTIGATIONS

The ground investigation programs were for both projects extensive. With direct implication on the assessment of critical foundations such as the anchor blocks at Storebælt and the ship impact loaded piers and pylons at Øresund, the geotechnical investigation programs included large scale tests in test pits, [1], [2]. These tests were designed specifically for calibrating the strength models used in the assessment of the bearing capacities.

A significant difference in the two test strategies was that the East Bridge investigations were performed to obtain material parameters to already established calculation models. At Øresund a test program was carried out by the bridge owner prior to the detailed design of the bridge. The challenge for the geotechnical engineer was thus to define and calibrate a suitable constitutive model to be used in ship collision analyses.

2.1 The East Bridge investigations

During the ground investigations for the West Bridge a thorough knowledge of the geotechnical conditions was obtained. The ground investigation for the East Bridge, comprising 100 geotechnical borings and 400 CPTs, thus served to give site specific information and to enhance the geotechnical design basis. Laboratory tests for classification and determination of strength and deformation properties of the insitu soils were carried out. Furthermore, the properties of the crushed rock used in the gravel pads were determined using large scale triaxial and shearing tests. These tests were used to defined a strength model for the clay till and the crushed rock used for the gravel pads, [1], [9].

Of special interest was the foundation of the anchor blocks. Using several different models it was found that the critical section was the gravel pads on which the anchor blocks were founded. Hence a calculation model for the interfaces between concrete caissons, gravel pads and clay till was established. Three principally different types of failure modes are possible for each foundation pad depending on load inclination, see Fig. 3. The critical mode for a given case will depend upon geometry, soil strength, and the inclination of the resultant force. A more detailed discussion of the failure mode that involves sliding along the disturbed clay till surface was presented by Mortensen, [8].

The sliding issue were dealt with by performing large scale sliding plate tests onshore at Sjælland on intact as well as remoulded clay till. The results from these tests were used to calibrate the analytical and numerical calculation models, [1], [11].

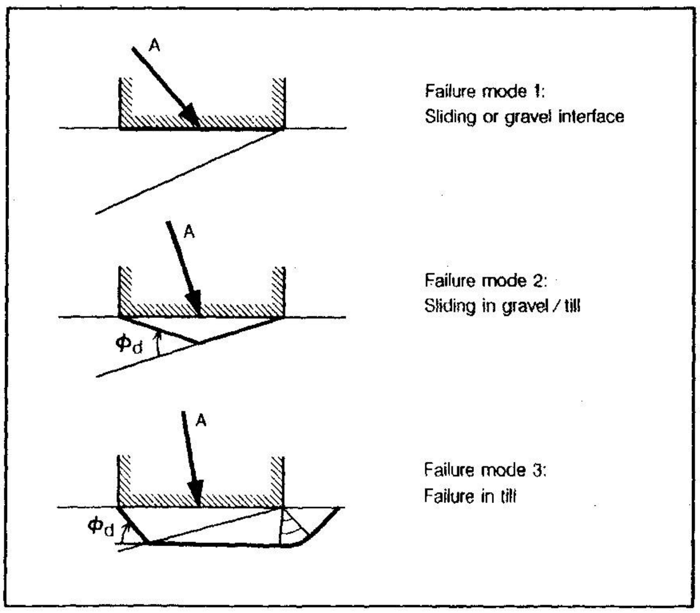

Fig. 3: Foundation pad failure modes

2.2 Øresund

The geotechnical investigations on Øresund was concentrated on the determination of the location and characteristics of the Copenhagen Limestone. The ground investigations for the bridge comprised 55 geotechnical borings and approximately 200 soundings. Particular emphasis was given to geological/geotechnical features that, if present, could present particular problems in relation to the foundation design:

- Highly crushed or fissured limestone.
- Extensive zones of unlithified limestone.
- Voids or cavities due to solution of limestone (karstic limestone).
- Soft sediments in depressions between mounds of bryozoas, especially in the transition between Copenhagen limestone and Bryozoan limestone.

The experience with foundations on the limestone was very sparse. Therefore extensive laboratory testing and medium scale shear plate tests were carried out, [2].

A vital issue in the design of the foundations was the accidental ship impact. As part of the tests carried out in advance of the design, several large scale shear plate tests were carried out on limestone onshore at Lernacken on the coast of Øresund. The vertical and horizontal plate loading tests were large enough to represent the rock mass properties of the limestone. Hence the results from the tests would be a indicator for the behaviour of the limestone when subjected to large horizontal forces, as for example during ship collision. The limestone was further investigated in triaxial and shear tests in the laboratory. The results of the investigations were synthesised into a constitutive model, see Fig. 4. This model served as basis for the definition and calibration of a computer model used to model ship collision in ABAQUS, [6]. Hence, in contrast to the Storebælt investigations no supplementary tests were necessary in order to calibrate the computer models used for assessment of the foundation design.

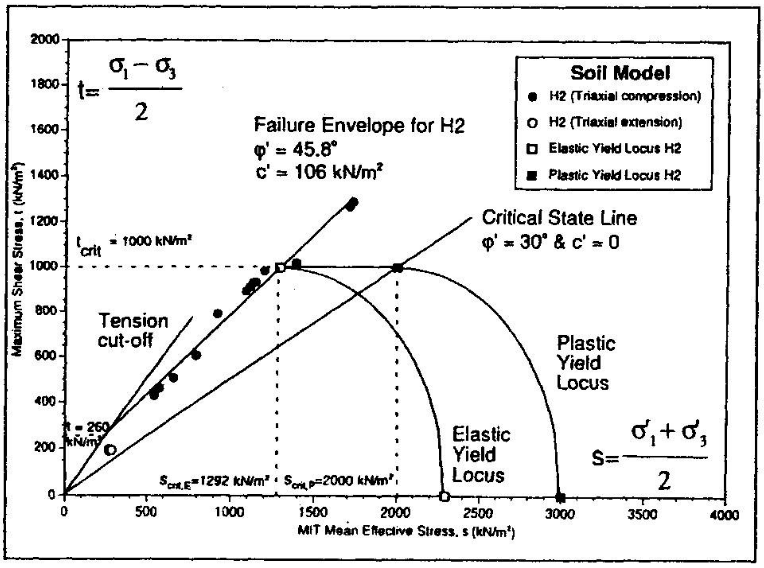

Fig. 4: Principle model for Copenhagen Limestone, [2]

3 GEOTECHNICAL DESIGN

The size and complexity of the projects were beyond normal experience and codes of practice. The safety strategies employed for the two projects were slightly different. At Storebælt the safety level was determined in terms of the safety factor β. The method giving the lowest possible safety factor was to be decisive for the design. The Øresund project was based on the Eurocode system (ENVs). Hence the design basis reflected the safety implied in this code system. For the ship collision case the verification of the material model used in the FE analyses demonstrated a significant conservatism in the design parameters when used with the chosen material model, [6].

A thorough evaluation of different calculation tools and their suitability in design was carried out prior to and during design. Three independent calculation methods were used:

- Upper Bound Theory
- Limit Equilibrium Analysis
- Finite Element Analysis

3.1 Storebælt anchor blocks

As part of the East Bridge Project all three methods were subjected to benchmark calculations. The governing load case for the anchor blocks was mainly due to the permanent load in suspension cable, see Fig. 5. One of the benchmark problems was the Anchor Block Case with a geometry and soil parameters close to those of the real anchor block.

The distance between and shape of the two foundation pads implied that their failure modes did not interfere. Hence the bearing capacity could for both the Upper Bound Theory and the Limit Equilibrium Method be determined by combination of two solutions to the basic failure modes shown in Fig. 3. A elasto-plastic 2D finite element analysis of the anchor block confirmed the assumptions of the two theoretical solutions. It gave almost the same bearing capacity and the plastic zones in the non-linear model coincided with the theoretical failure lines, [10].

From the benchmark tests it was concluded that all three methods could be applied to problems of combined sliding and vertical load. That is, of course, provided that the model parameters are carefully calibrated.

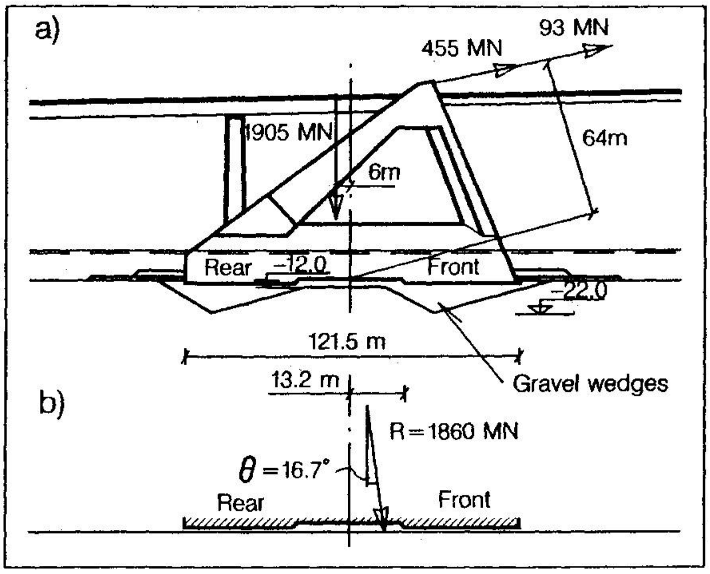

Fig. 5: Principle section and loading of the anchor block.

3.2 Ship collision at Øresund

Ship collision is the governing load case for the design of most of the piers, see Fig. 6. Due to the large horizontal load, the effective width of the footings will be only 3-4 m even with high mobilisation of the passive resistance on the sides of the caissons.

The three above-mentioned calculation methods were used for Øresund in spite of the conclusions drawn from the Storebælt investigations: "*.....as long as the bearing capacity is governed by the clay till, the differences between the selected analysis methods will be small. ... For cases where the strength of the frictional material dominates the bearing capacity, care will be needed when deciding upon the analysis method to be used*", Sørensen et al., [10]. This warning caused fundamental calculations as the bearing capacity for the footings was primarily dictated by the limestone's frictional strength parameters. The assessment of the bearing capacity therefore implied a reassessment of the applicability of the three methods in a case with friction material and strongly eccentric loading.

The Upper Bound Theory was used to calculate the bearing capacity of the rupture mechanism for the foundations. Further, the theory was used to evaluate whether it was the peak or the residual strength parameter that should be used to calculate the peak bearing capacity of the footing. It was concluded, [5], that the peak strength should be used and that the friction angles should be corrected for the influence of dilatancy not satisfying the associated flow role.

Also the Limit Equilibrium Analysis was used with the same purpose. Two limit equilibrium computer programs were used. The first program, BEAST, is based upon the method of slices. The second program, WEDGE3, was developed as a part of the studies carried out. The results for both programs were comparable with the other methods. See also [3].

Finite element analyses using ABAQUS were performed to verify the capacity against accidental ship impact. The foundations should, according to the design basis for accidental ship impact, not only be able to withstand the maximum ship collision force, but furthermore, the maximum permanent displacement was not to exceed a given limit value. FE analysis was a means to assess both requirements in a consistent way. The FE models comprised 2D plane strain elements with a material behaviour, that was calibrated to match several large scale plate tests carried out at Lernacken, [2]. The calibration and application of the model to ship impact is described in a fellow-paper, [6].

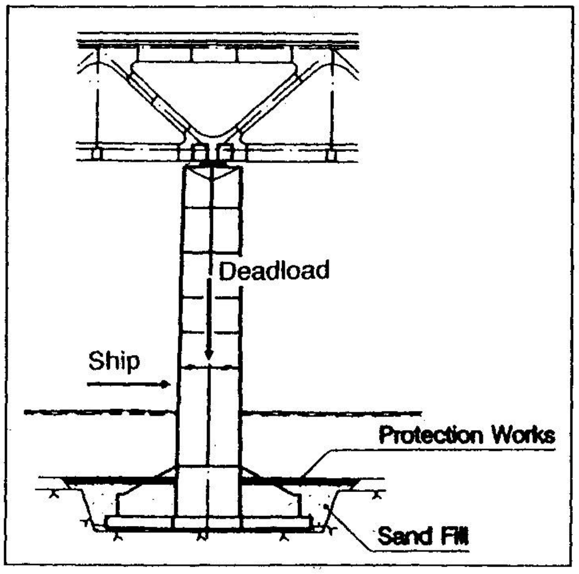

Fig. 6: Principle section in the pier foundation

The finite element calculations confirmed the results from the Upper Bound Theory and the Limit Equilibrium Analysis. Furthermore, the use of FEM made it possible to calculate the expected permanent displacements resulting from a ship collision.

4 CONCLUSIONS

Some of the most important considerations regarding the geotechnical design for the Storebælt East Bridge and the Øresund Bridge have been presented. The two projects clearly demonstrated two important issues in modern bridge design:

- Careful design of the geotechnical investigations, test pits and laboratory tests is necessary in order to calibrate the design models for critical elements. The tests are important in order to minimise the costs while preserving the proper conservatism of a highly critical infrastructure component, such as a bridge.
- The application of several independent tools in the assessment of the bearing capacities of anchor blocks and ship impact loaded piers has proven successful and necessary. The increasing focus on a combined design criterion consisting of both strength and displacement requirements makes the use of numerical methods such as finite element analysis an essential part of the designer's documentation. First of all the FE model can give estimates on bearing capacity and the associated deformations. Secondly, FE analysis may be used to assess the applicability of the simpler models, which are more suitable for design.

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Evaluation of Cracked Soft Rock on In-Situ Test Results

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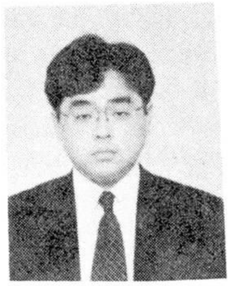

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Summary

The base rock expected to be used for the foundation of a railway arch bridge(180m span) currently in the planning stage is heavily-cracked auto-brecciated andesite (rock grade about C_M - C_L~Rock classification of dam foundation in Japan). There was a need to accurately evaluate the bearing capacity, since a load amounting to as much as about 2.1kN (normal condition) would be applied to the base rock at each bridge pier. Since being an arch bridge, there was a desire to restrain support point displacement of the foundation to the utmost, and since it was also necessary to take into consideration the reduction in bearing capacity due to the effect of slopes due to the fact that the foundation would be constructed on slopes facing the river. We thus conducted block shear tests as a way of improving the accuracy of the shear constant. We evaluated bearing capacity of the base rock and shear constants (c, ϕ), analyzed the test results as well as the results of prior tests on the same type of base rock and determined the material values to be used in design. And we explained various methods for evaluating the bearing capacity, shear constants of cracked soft rock.

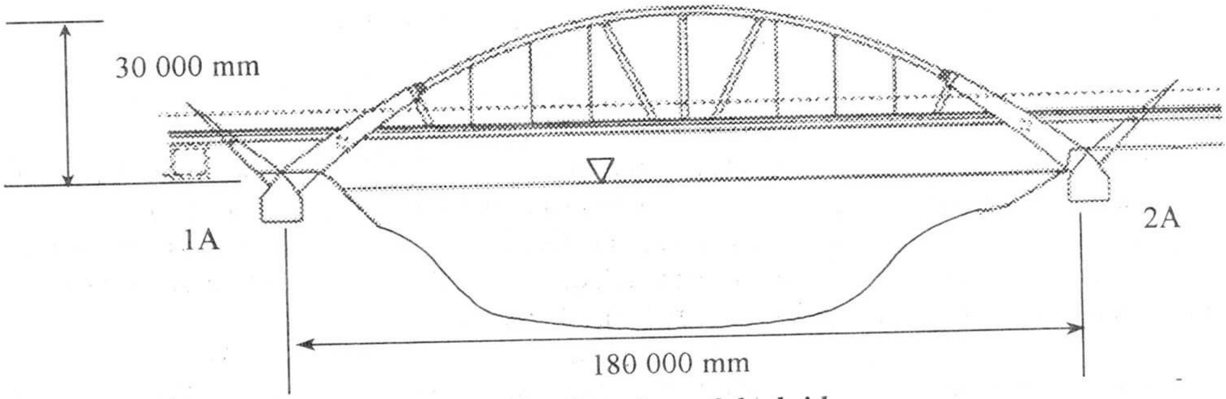


Fig.1 Outline view of the bridge

1. Estimations of the bedrock conditions based on geological surveys

We will next give an explanation of the strata composition in the vicinity of the foundation. Side 2A is characterized by surface deposits (fill, old surface soil) with an N-value of about 5 to a depth of GL-4m, and terraced deposits (sand and gravel mixed with cobblestone) at a depth of GL-7-9m with soft stone tuff-breccia with heavily-cracked auto-brecciated andesite intrusions) at depths greater than GL-9m (refer to Fig.1 and Table 1).

Table 1 Results of geological surveys of the bedrock (2A)

Type of rock		Andesite
Wet density γ_t (g/cm^3)		2.20
Water absorption Q (%)		12.5
Triaxial compression test (CU)	cohesion strength c' (N/cm^2)	1.00
	Angle of internal friction(°)	43.3
Unconfined compression strength (N/cm^2)		5.62
Ultrasonic velocity V_{p0} (km/s)		2.54
Elastic wave velocity V_p (km/s)		1.80

The bedrock of side 2A is rock grade C_M-C_L and has R.Q.D (Rock Quality Designation; percentage of the total hole length made up of cores with a length of 10cm or more acquired by boring. And we must estimate immediately after retrieval) of 10-25.

It is moreover classified as "extremely bad" on Deere's bedrock quality scale (refer to Table 2). In addition, the number of cracks per unit volume 'Jv' estimated from the RQD is 27-32. The rock is therefore fragile in quality and is conjectured to be composed of small blocks.

Table 2 Bedrock quality table according to RQD (Deere)

RQD indication	explanatory expression
0 - 25	extremely bad
25 – 50	bad
50 – 75	generally good
75 – 90	good
90 - 100	extremely good

2. Evaluation of various evaluation methods for bedrock bearing capacity and shear constant

2. 1 . Method using past examples of design based on rock type

According to data from nationwide surveys of material values used in the design of long bridges with soft rock as the bedrock, it is conjectured that cohesion strength c and angle of internal friction ϕ of the andesite at side 2A, which would be indicated with in the range in Table 3. These should probably be used strictly as reference values, however, since there are no classifications of the conditions of cracking, rock grade or other elements.

Table 3 Examples of c and ϕ employed in design by rock quality

Classification	Type of rock	c (N/cm^2)	ϕ(°)
Pyroclastic material	andesite	0.5	40

2.2. Method according to the Japan Roadway Public Corporation ; Method using the cracking index

There is a method that uses the cracking index that quantitatively indicates the effect of bedrock cracking on strength constants. The cracking index is shown in formula (1).

$$Cr = 1 - [V_p/V_{p0}]^2 \quad \text{.........................(1)}$$

Where Cr: Cracking index
V_p: Natural ground horizontal elastic wave velocity
V_{p0}: Specimen ultrasonic wave velocity

In addition, Figs. 2 & 3 indicate data compiled by the Japan Roadway Public Corporation relating to the relationship between the cracking index and the reduction index of cohesion strength and the angle of internal friction. This data included the test data of the Railway Technical Research Institute of the National Railway. It is possible to estimate the values of c and ϕ by means of a curve that takes the lower limits of variation indicated in Fig. 2 & 3. In other words:

$$c = k_c \times c_0 \text{..................(2)} \qquad \phi = k_\phi \times \phi_0 \text{....................(3)}$$

where k_c: reduction index of cohesion strength; k_ϕ: reduction index of the angle of internal friction; c_0: specimen cohesion strength; ϕ_0: specimen angle of internal friction

Material values for the bedrock of side 2A determined from the above are shown in Table 4. According to this method, it is necessary to carry out a reduction of about 80% for cohesion strength and about 40% for the angle of internal friction relative to the results of triaxial compression tests. However, it is possible that shear constants c and ϕ are underestimated with this method, since there are various reasons, such as the fact that basically the minimum values are used for the reduction index and the impossibility of differentiating quantitatively between open and closed cracks because of the use of elastic waves.

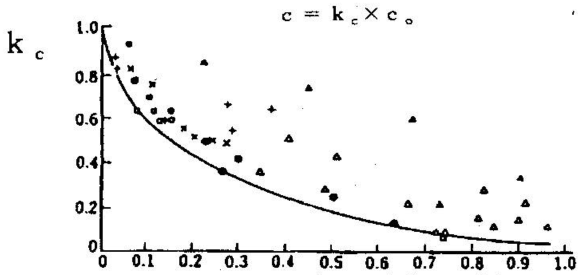


Fig.2 Relationship of Cr and k_c

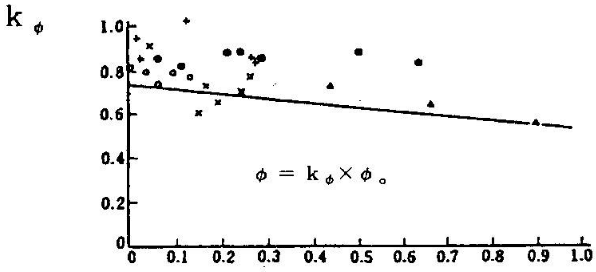


Fig. 3 Relationship of Cr and k_ϕ

Table 4 Shear Constant estimated by using the cracking index

Cohesive strength c			
Triaxial compression test c_0	Cracking coefficient c_r	Correction factor k_c	Corrected value c
1.0 N/cm^2	0.50	0.2	0.2 N/cm^2
Angle of internal friction ϕ			
Triaxial compression test ϕ_0	Cracking coefficient c_r	Correction factor k_ϕ	Corrected value ϕ
43.3 °	0.50	0.6	26 °

2.3. Method of the Honshu-Shikoku Connecting Bridge Public Corporation

In addition to rock classification and triaxial compression tests, the Honshu-Shikoku Connecting Bridge Public Corporation also conducted numerous other tests. These were including RQD, unconfined compression strength and longitudinal elastic wave velocity tests as well as in-situ plate bearing tests, boring hole horizontal loading tests and block (or rock) shear tests and determined the various correlations between them. As a method for estimating shear constants c and ϕ, Guidelines for the Determination of Weathered Granite Bearing Characteristics (draft) were set up. Thereby, we were able to estimation at three levels of precision using (1) actual measurements of c and ϕ as the result of block (or rock) shear tests in in-situ or similar base rock as the main estimates, (2) estimates consisting of measurements in boring holes and also making use of correlations to prior tests and (3) estimations carrying out rock classification by macroscopic observation and using the results of correlations to prior tests.

It was thought that using a method such as this would make it possible to grasp the material values of the base rock with a considerable degree of precision if various surveys and tests of similar base rock were conducted and correlations were determined. From the standpoint of economy, however, it would be difficult to conduct the same surveys and tests on ordinary bridges as on a massive structure such as the Honshu-Shikoku Connecting Bridge. In addition, since the guidelines are limited to weathered granite, they could not be applied intact as a method for the evaluation of the base rock in the present case.

3. Evaluation of shear constant by block shear tests

The bedrock of side 2A is heavily-cracked intrusive rock and, in terms of quality, was judged to be "extremely bad." We therefore decided to focus first on the bedrock and conduct block shear tests in order to promote an improvement in the precision of the shear constant.

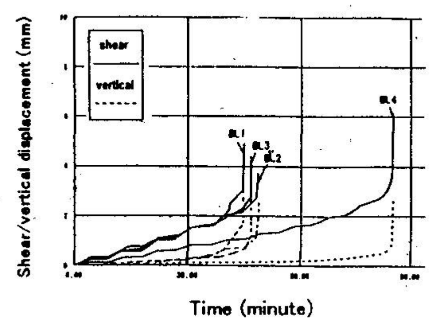


Fig. 7 Relationship between time and shear/vertical displacement (average)

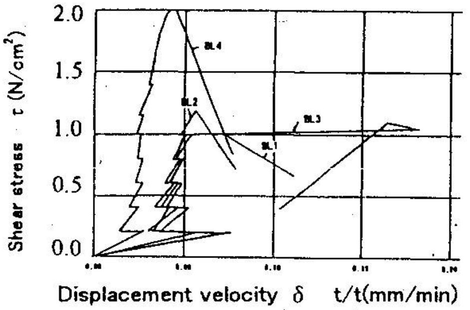


Fig. 8 Relationship between shear stress and displacement velocity

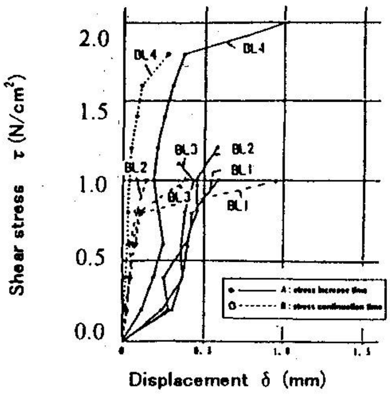


Fig. 9 Relationship between shear stress and the amount of shear displacement

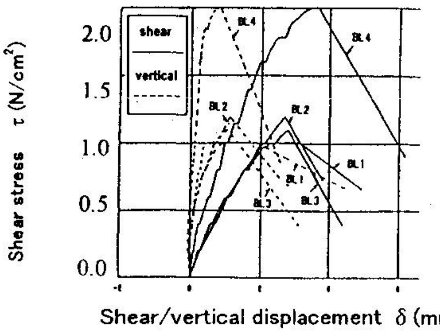


Fig. 10 Relationship between shear stress and shear/vertical displacement (average)

Table 5 Shear strength by base rock origin

(τ_o : N/cm², ϕ : ° f : tan ϕ)

Rock type by origin		Rock classification								
		C_H			C_M			C_L		
		τ_o	ϕ	f	τ_o	ϕ	f	τ_o	ϕ	f
Paleozoic and Mesozoic deposits	Average	2.0	50	1.2	1.9	45	1.0	0.7	45	1.0
	ceiling	3.1	57	1.5	2.9	53	1.3	1.3	51	1.2
	lower limit	1.2	44	1.0	1.0	39	0.8	0.5	33	0.7
Regional metamorphic rock	Average	2.4	50	1.2	1.4	45	1.0	0.6	45	1.0
	ceiling	3.9	55	1.4	2.5	53	1.3	0.8	49	1.2
	lower limit	1.1	50	1.2	0.8	41	0.9	0.4	40	0.8
Igneous rock, abyssal rock	Average	4.4	51	1.2	3.0	47	1.1	2.2	45	1.0
	Ceiling	8.2	52	1.3	6.8	50	1.2	4.0	45	1.0
	lower limit	2.0	50	1.2	1.4	45	1.0	0.8	45	1,0
igneous rock volcanic rock	Average	2.7	47	1.1	1.9	45	1.0	0.7	40	0.8
	ceiling	3.5	53	1.3	2.6	50	1.2	1.1	44	1.0
	lower limit	1.5	45	1.0	1.3	44	1.0	0.3	35	0.7
volcanic eposits (Tertiary and later)	Average	3.5	51	1.2	2.9	51	1.2	—	—	—
	ceiling	4.8	55	1.5	2.8	55	1.4	—	—	—
	lower limit	2.2	50	1.2	1.1	50	1.2			

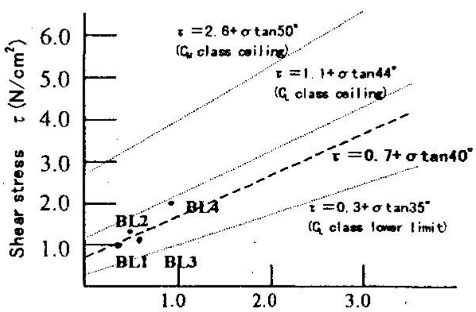


Fig. 11 Test results and volcanic rock shear strength properties

The following is a summary of the results of the tests that we conducted.

(1) Base rock conditions of the test surface and failure surface

Observation of the conditions of the shear plane indicated that the failure of blocks BL1-3 was deeper than in block BL4 and there was also a tendency in all of the blocks to form a slope inclining generally toward the front. In regard to the shear plane conditions, blocks BL1-3 sheared in the direction of the joint of the breccia contained in the auto-brecciated andesite. However, block BL4 sheared with a number of cracks appearing in the base rock itself in the direction of the shear and in the perpendicular direction as shear loading was applied.

(2) Shear loading strength properties

* Relationship between time and shear/vertical displacement (average)

Failure took 2-3 times longer with BL4 than with BL1-3.

* Relationship between shear stress and displacement velocity

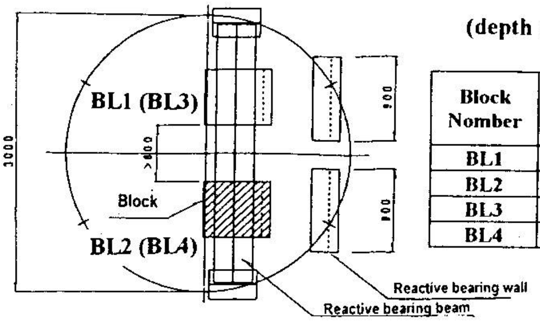
The displacement velocity of BL4 was about 1/2 that of BL1-3 and the progress of the displacement was slow.

* Relationship between shear stress and amount of shear displacement (average.)

3.1. Test apparatus and method

Deep foundation holes with a diameter of 3m were excavated in the vicinity of the foundation on side 2A and the tests were conducted using 2 blocks each at a depth of 8.5m (block numbers BL 1 & 2) and a depth of 10.0m (block numbers BL 3 & 4) from G.L. A plane view of the test conditions is shown in Fig.4 and an inclined view is shown in Fig.5.

The shear plane was 60cm × 60cm in size and, since load strength was conjectured to be in the range of 0.4N/cm^2 under normal loading conditions at the bottom of the foundation, vertical loading was carried out in the stages of 0.1, 0.2, 0.3 and 0.5N/cm^2. The loading velocity was set at 2.5×10^{-2} N/cm^2/min with a stationary period of 5 minutes at each stage. In addition, horizontal shear loading was carried out at a velocity of 5.0×10^{-2} N/cm^2/min with a stationary period of 5 minutes after each 0.2N/cm^2 loading and the load was increased until shear failure occurred. The blocks were set up so that they would shear in the direction of the river.



(depth of 10.0m)

Block Nomber	Vertical loading (N/cm^2)
BL1	0.1
BL2	0.2
BL3	0.3
BL4	0.5

Fig.4 Plane view of the conditions of block shear tests [depth of 8.5m]

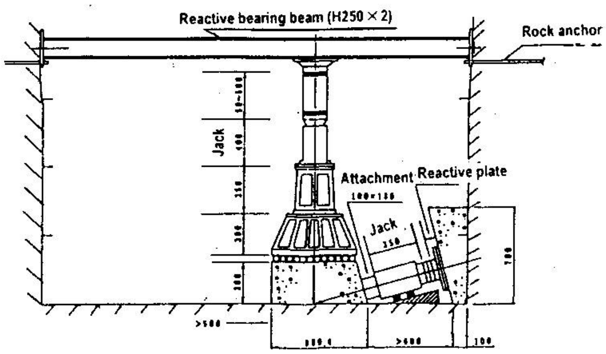


Fig.5 Side view of the conditions of bock shear tests

3.2. Results

The base rock at the bottom of the deep foundation holes was not uniform and there were portions of hard material here and there. It was necessary to carry out the shear plane of each block with the same quality and grade of rock and conduct the tests in base rock that was typical of the rock quality in the deep foundation holes. Therefore, we decided to conduct the tests with C_L grade base rock.

The conditions of failure of the shear plane of the blocks are indicated in Fig.6. The relationship between time and shear/vertical displacement (average) is indicated in Fig.7. The relationship between shear stress and displacement velocity is indicated in Fig.8. The relationship between shear stress and the amount of shear displacement is indicated in Fig.9. And the relationship between shear stress and shear/vertical displacement (average) is indicated in Fig.10.

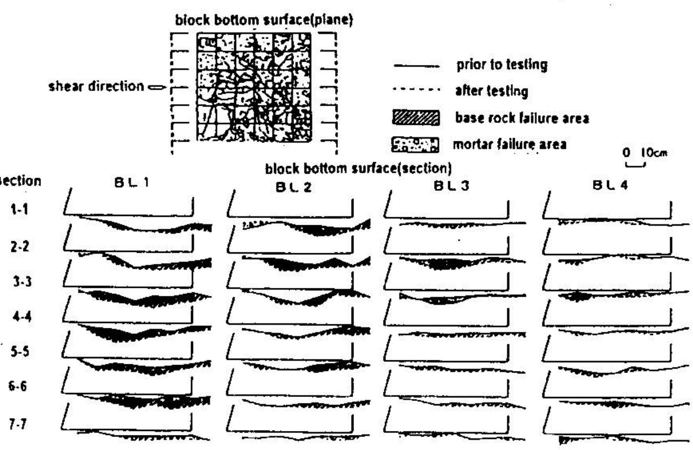


Fig.6 Conditions of failure of the shear plane of the block

The amount of shear displacement was about 1/2 that of BL1-3 for both stress increase time and continuation time. This would indicate that the base rock of BL4 is hard and that fractures are closed.

The above points out differences between BL4 and BL1-3 in dynamic properties and we feel that it is necessary to treat them differently in the compilation of test results. n addition, based on our observations of the base rock, we conjectured that cracks in the base rock were closed and that there was also little polarity in the cracking.

3.3. Material evaluation

Table 5 shows values for general shear strength by type of rock obtained from in-situ shear tests conducted at a dam site. The andesite in these tests is classified as volcanic or igneous rock and we propose that the shear strength t_0(= c) of rock grade C_L is 0.3-1.1N/cm^2 and that the angle of internal friction is 35-45°.

In addition, if we plot the values obtained in the tests in the shear strength properties of the volcanic rock shown in Fig.11, we could see that the test values for BL1-3 would be average values for C_L grade rock and that the test values of BL4 would be at the upper limit for C_L grade rock. Based on this, we can state that appropriate values were obtained in the tests corresponding to the rock type and grade.

If we therefore consider the data for BL1-3 to be typical of the shear strength of the andesite in the tests and use the average angle of internal friction (ϕ = 40°) for the igneous and volcanic rock of Table 5, it would be possible to formulate the following relational expression:

$\tau = 0.7 + \sigma \tan 40°$ Where, τ: shear strength (N/cm^2), σ: normal stress (N/cm^2)

However, taking into consideration the risk of creep deformation due to sustained loading, residual displacement in the wake of massive earthquakes and other deterioration as well as the scale effect, cohesive strength c was reduced to about 1/3 as an engineering judgment.

Based on these, we decided to use the following as the shear constant for the bedrock on side 2A in the design of this bridge:

Cohesive strength: c = 0.2N/cm^2 Angle of internal friction: ϕ = 40°

4. Conclusion

It is thought necessary to take the following into account when applying these to base rock material evaluation and design.

(1) Although there is a variety of methods for evaluating base rock bearing capacity and shear constant, there are no established methods. Therefore, instead of adhering to one single method, it is necessary to conduct wide-ranging examinations of rock type, rock grade, cracking direction, converted N-value, design examples and other elements.

(2) When using the method for evaluating the shear constant of base rock by means of the cracking index, shear constants c and ϕ may possibly be underestimated.

(3) In-situ tests are effective in the evaluation of base rock shear constant and, especially in the case of base rock with little prior experience, block (or rock) shear tests, which can grasp the shear constant to a certain degree by mass, are effective. However, since costs will increase as more tests are conducted, it is desirable to conduct evaluations while referring to shear strength by base rock origin for which data from prior in-situ shear tests is available.

Postscript

The above is an account of our implementation of block shear tests to an unusual base rock as a bridge foundation and the application of the results to design. There are no established methods for the evaluation of base rock bearing capacity and we feel that it will be necessary hereafter to accumulate in-situ test data and conduct research into bearing capacity theory with the addition of the influence of cracking.

COMPUTERISED RESISTIVITY METER FOR SUBSURFACE INVESTIGATIONS

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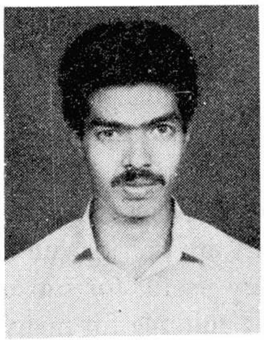

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ABSTRACT:

Geophysical investigations are useful for the selection of suitable sites for foundation of bridges, dams, tunnels etc. There are four types of geophysical methods employed for the solution of various civil engineering problems. They are electrical methods, seismic methods, gravity methods and magnetic methods. But electrical methods are widely used because of their easy operation, less cost and the principle suitable for many problems. Electrical resistivity methods are being successfully used to findout soil thickness, rock layers, depth to basement, subsurface structures and groundwater.

When current is passed through the two current electrodes, it is conducted through the soil, rock and mineralised water and the potential difference is received through the two potential electrodes. This is the resistance and apparent resistivity is obtained by multiplying the resistance with the geometric factor.

The commonly used electrode arrangements are 1. Wenner and 2. Schlumberger. Two types of field procedures are in common use, resistivity sounding and resistivity profiling.

Many instruments are available in India and abroad for geoelectrical survey. Aquameter-CRM is a modern version of earth resistivity meter, which makes use of advanced technology of microprocessors. The operation procedures, available facility in the instrument and its uses in various field conditions are explained in this paper.

The data obtained from the field can be processed by using either curve matching techniques or computer inversion techniques. Applications and results of geoelectrical survey in various lithological conditions are also presented. The use of geoelectrical survey for the selection of suitable sites for bridge are presented.

Results obtained from electrical survey should be correlated with the results of direct methods of exploration, as there are some limitations in this method.

1. INTRODUCTION:

Before undertaking any design work for a project, a civil engineer must have full information about the earth material on which the structure is to be founded, constructed with or in which the construction is to be carried out. This will necessiate a preliminary examination of the work site before detailed designs are prepared. Otherwise disastrous may result and almost always with consequent monetary loss. Many such losses for bridges, dams and tunnels have been reported in the past due to improper investigations {4}.For subsurface exploration , exploratory drillings and borings are carried out normally. This involves high cost and labour. Geophysical surveys are very useful for subsurface exploration because of its lesser cost, easy operation and the principle suitable for many problems. The suitability of geoelectrical surveys for various geologic problems have been reported by many earlier workers {1}.

Geophysical methods are of four types viz., 1. Electrical methods, 2. Seismic methods, 3. Magnetic methods and 4. Gravity methods. But electrical methods are widely used for many civil engineering and geologic problems, because of high contrast of electrical resistivity exists between the subsurface formations. Many electrical resistivity instruments are available in the market. But Aquameter CRM 20 is the only available computerised resistivity meter in India for subsurface investigation.

2. INSTRUMENTATION

Aquameter CRM 20 (Computerised Resistivity Meter) is a computerised earth resistivity meter, which make use of advanced technology of microprocessors. In conventional resistivity meters, the transmitter and amplifier are housed in seperate enclosures because of interference and electromagnetic coupling. The CRM is housed in a single casing-possible because of incorporation of modern electronic principle. Aquameter CRM 20 is shown in plate 1.

The generator circuit in Aquameter CRM 20 is so designed to maintain the current flowing through the ground at preset level, irrespective of variations in the soil and contact condition. The current passes through the C_1 C_2 electrodes in the ground and the potential generated across V_1 V_2 electrode is sensed by the instrument. The instrument is capable of detecting even a very low signal riding on a noisy background. The noise elimination takes place at various stages of amplification and the filtered signal is then processed to derive the resistance information. For excessively noisy condition the averaging mode is provided externally in addition to the internal built-in averaging. This facilitates geophysical prospecting even in difficult terrain {3}.

The microprocessor based circuit always ensure the proper functioning of the instrument. Any malfunction such as low battery voltage, a poor contact at the current setting etc, is immediately detected and conveyed to the operator through LCD display. This eliminates the possibility of any erraneous readings.

A seven position switch is provided to select the appropriate current. The current ranges are 0.2, 0.5, 1.0, 2.0, 5.0, 10 and 20 mA. Accuracy of the result will be higher, the higher the value of current. The Aquameter is provided by a rechargeable high power battery of 12V. The generator circuit inside the Aquameter coverts the 12V DC into a high voltage supply of 150 V DC which is made to alternate at regular intervals. This constant current genetator circuits sends out the current through the C_1C_2 electrodes and maintain it at the preset value throughout the measurement.

By selecting the number of cycles depending on the ground noise, one can get noise free readings. The available cycles are 1, 4, 16 and 64. Using the range switch it is possible to select the resistance range 10 Ohm, 100 Ohm, 10 K Ohm or 1 M Ohm depending on soil resistivity. Normally the lowest range is very suitable for accurate readings. Self potential(SP) cancellation is done automatically by the instrument itself. There is no need of using non-polarising electrodes. But when SP values are required they can be measured in the voltage mode through the use of non polarising electrodes. There are only 5 controls of the instrument. They are (i) On/Off switch, (ii) 7 position generator current selection switch, (iii) 6 position range selector switch, (iv) 4 position cycle switch and (v) measure start/push button.

3. PRINCIPLES OF REISTIVITY

The pattern of electric current flows in the ground is dependent on the sub-surface variation in conductivity. The electric potential distribution in the surface is thus related to the subsurface geology. The electrical resistivity of different materials beng different from each other, it is possible to distinguish them by their resistivity.

The matrix minerals in rocks are insulators. In rocks containing fluids, current is conducted electrolytically by the interstitial fluids and resistivity is controlled by porosity, water content as well as quantity of dissolved salts. Clay minerals are capable of storing electrical charges and current conduction in clay minerals is electronic and electrolytic {10}.

Resistivity is a physical property of a substance. It is defined as the resistance offered for the flow of electric current by unit cube of the substance when voltage is applied across the opposite faces. The resistivity is expressed in Ohm-meters. The inverse of resistivty is termed as the conductivity.The resistance for a substance is a function of the resistivity, size and shape. The resistance (R) offered by the substance with regular shapes such as cylinders, cubes etc can be determined with the formula

$$R = \frac{\rho L}{A}$$

Where

ρ is the resistivity of the substance,
L is the length of the substance and
A is the area of cross section of the substance

4. RESISTIVITY SOUNDING VERSUS RESITIVITY PROFILING

Two basic types of field procedures are in common use, resistivity sounding and resistivity profiling. In resistivity sounding, the electrode spacing interval is changed while maintaining a final location for the center of the electrode spread. Since depth of investigation increases in a general way with increasing electrode spacing, resistivity sounding will be preferred if the aim is to determine the vertical variation of formations (like thickness of rock layers and depth to basement). In resistivity profiling, the location of the spread is changed while maintaning a fixed electrode spacing interval, for example to locate the boundary of a gravel deposit.

Measurement of resistivity

Four electrodes are required for measuring the resistivities of the subsurface formations. A current of electrical intensity " I " is introduced between one pair of electrodes, called current electrodes C_1C_2. The potential difference produced as a result of current flow is measured with the help of another pair of electrodes called potential electrodes P_1P_2. Fig 1 shows the electrical resistivity method of subsurface investigation. Let " ΔV "represents the potential difference, the apparent resistivity measure (ρa) is

$$K \times \frac{\Delta V}{I}$$

A comprehensive of 25 electrode arrangements are available for various geological investigations {8}, of which Wenner and Schlumberger arrangements are being widely adopted for civil engeineering and groundwater problems.

Wenner array has four collinear equi-spaced point electrodes as shown in Fig 2a. The four electrodes C_1,C_2,P_1 and P_2 are placed at the surface of the ground along a straight line symmetrically about a central point, so that $C_1P_1 = P_1P_2 = P_2C_2 = a$, where " a " can be called as interelectrode separation. The configuration factor 'K'(also called as geometric factor) for this array is $2\pi a$ and therefore the apparent resistivty (ρa) for this array becomes

$$\rho a = 2 \pi a \left(\frac{\Delta V}{I}\right)$$

Schlumberger Array is shown in Fig 2b. This array also uses four collinear point electrodes, but measures the potential gradient at the mid-point by keeping the measuring electrodes close to each other. The configuration factor for Schlumberger array is

$$K = \pi MN \left(\frac{L^2}{MN} \quad \frac{1}{4}\right)$$

5. RESISTIVITY SOUNDING INTERPRETATION

The reading obtained from Aquameter is the resistance and apparent resistivity is obtained by multiplying resistance with geometric factor. After the apparent resistivity is calculated , it is important to findout the true resistivity. Many theoretical curves {5,9} and computer inversion programs {2,6} are available to calculate the true resistivity with layer details.

Types of sounding curves:

The form of the curves obtained by sounding over a horizontally stratified medium is a function of resistivities and thickness of the layers as well as of the electrode configuration. If the ground is composed of a single homogeneous and isotropic layer of infinite thickness and finite resistivity then, irrespective of the electrode array used, the apparent resistivity curve will be a horizontal line.

If the ground is composed of three layers of resistivities $_1$, ρ_2 and ρ_3 and thickness h_1, h_2 and $h_3 = \infty$, the geoelectric section is described according to the relation between the values of ρ_1, ρ_2 and ρ_3. There are four possible combination between the values of ρ_1, ρ_2 and ρ_3. They are

$\rho_1 > \rho_2 < \rho_3$	H type sections
$\rho_1 < \rho_2 < \rho_3$	A type sections
$\rho_1 < \rho_2 > \rho_3$	K type sections
$\rho_1 > \rho_2 > \rho_3$	Q type sections.

Fig 3 shows the model master curves of H,A,K and Q type sections for Schlumberger sounding data. Electrode spacings (AB/2) are plotted in abcissa and apparent resistivity (ρa) is plotted in ordinate. True resistivities for first, second and third layer are given in the figure for the H,A,Q and K type curves as $\rho 1$, $\rho 2$ and $\rho 3$ respectively. "H" type curves are found in place where top soil of high resistivity with water saturated or clay layer as the second layer and underlain by con act hard rock. "A" type curve results if the formations are resistive continuously towards the th. "K" type occur if the cental layer has low resistivity then the overlying and underlying trata. " Q " type curve results if the resistivity is decreasing towards depth.

If the ground is composed of more than three horizontal layers to resistivities ρ_1, ρ_2, $\rho_3 .... \rho_n$ and thicknesses h_1, h_2, h_3....hn = ∞, the geoelectric section is described in terms of relationship between the resistivities of the layer, and the letters H,A,K and Q are used in combination, to indicate the variation of resistivity with depth. In four layer geoelectric sections, there are eight possible relation between ρ_1, ρ_2, ρ_3 and ρ_4.

$\rho_1 > \rho_2 < \rho_3 < \rho_4$	HA type sections
$\rho_1 > \rho_2 < \rho_3 > \rho_4$	HK type sections
$\rho_1 < \rho_2 < \rho_3 < \rho_4$	AA type sections
$\rho_1 < \rho_2 < \rho_3 > \rho_4$	AK type sections
$\rho_1 < \rho_2 > \rho_3 < \rho_4$	KH type sections
$\rho_1 < \rho_2 > \rho_3 > \rho_4$	KQ type sections
$\rho_1 > \rho_2 > \rho_3 < \rho_4$	QH type sections
$\rho_1 > \rho_2 > \rho_3 > \rho_4$	QQ type sections

For five layer sections there are 16 possible relationships between $\rho_1, \rho_2, \rho_3, \rho_4$ and ρ

Interpretation by curve matching:

Curve matching technique is widely used for the assessment of layers thickness and layers resistivity. Apparent resistivity values are plotted in ordinate and electrode spacings are plotted in abcissa in a tracing paper superimposed on double log sheet. A smooth curve should be drawn to connect all the points. This is the field curve which may be two, three or four layer case. If the field curve matches a three layer theoretical master curve, then that three layer curves should be used for interpretation.

Three layer master curve should be selected and the field curve should be superimposed on the master curve. Move the transparent sheet (field curve) horizontally or vertically over the master curves keeping the corresponding axes parallel, till the field curve matches with one of the master curves. Trace the master curves origin on transparent sheet and note down the h2/h1, $\rho 2/\rho 1$ and $\rho 3/\rho 1$ values corresponding to the master curve which matched with the field curve. Origin traced on the field curve will yield $\rho 1$ and h1. Then the other parameters $\rho 2$, $\rho 3$ and h2 can be deduced automatically.

Computer Inversion Techniques:

Some errors may occur in manual curve matching methods. Computer inversion programme RESIST87 is available for accurate interpretation {6}. The results obtained from curve matching method can be given in this programme as input for getting effective result. Otherwise some arbitrary values for the field curve can be given by experience and by iteration method the true resistivity and layer thickness can be obtained form this program.

Limitation of this method:

From these methods only we can get a geoelectric section. This is because the resistivity method is governed only by the resistivity contrast between various subsurface layers which need not always correspond to the geological boundaries. Geoelectric and geological boundaries coincide only when there is a pronounced contrast between geologic layers. But if the resistivity values and layer thicknesses are correlated with the existing neigbhour borewell or electric logs or with the old data, one can get effective result and this will save much cost.

6. CONCLUSION

Subsurface prediction from geophysical survey is indirect and such prediction must be checked by actual penetration of subsurface, either by exploratory boring and sampling of the soil or by excavation of small shaft or adits, which will also yield samples of materials for later testing in laboratories. Geologic structures like fold and fault may be undetected by exploratory boring and other direct methods because of the random sampling ,but they can be easily detected by geoelectrical survey. Caerful examination of geoelectrical sounding curves with actual exploratory boring will be very useful for larger areas where it is not possible to go for exploratory boring densely. The exploratory boring will be carried out frequently for shallow depths. But geoelectrical survey can be carried out for 500 m depth with high accuracy. This survey can also be carried out rapidly for many locations and if anamolies in subsurface is found then one can go for exploratory boring. This reduces the cost and time of exploratory boring.

7. ACKNOWLEDGEMENTS:

First author thank Dr.P.Radhakrishnan, Principal and Prof.S.Rajasekaran, Head of the Civil engineering department, PSG College of Technology , Coimbatore for their constant encouragement.

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SHORT-FALLS IN SUB-SURFACE INVESTIGATIONS FOR BRIDGE FOUNDATIONS - A FEW CASE STUDIES ALONGWITH REVIEW OF CODAL PROVISIONS.

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1. SUMMARY

Sub-surface exploration for all Engineering structures ingeneral and bridge foundations in particular is the most important item in the planning of these structures. Investigations by core drilling are discussed in this paper. Importance of core drilling is said to have been realised by the engineers, however, in practice it does not seem to be true. Even in recent past there are a number of projects where there was time and cost over-run basically due to inadequate or improper investigations. At the same time, there are a few projects where considerable economy is achieved with proper investigations. Interpretation is another aspect which is usually neglected. In the investigations what is not obtained from the cores is more important than what is obtained. What is not obtained can be analysed only if very careful observations are made and also recorded during drilling and then the total information collected is interpreted, either by an Engineering Geologist or by a specially trained Engineer, who is well acquainted with the geology of the area as well as the subject proper.

Guidelines are available in various I.S./ I.R.C. codes and P.W.D. Handbook of Maharashtra State. However, experience is that these guidelines are not strictly followed. This is true whether the investgations are carried out by a renowned agency or a local one. Provisions in these specifications which are important but are usually neglected are discussed in this paper. Although very important, core drilling, particularly for bridge or tunnel projects, is done more as a formality and interpretation is either totally neglected or where attempted, give an entirely false picture as they are done by people not qualified for the purpose.

In view of authors' experience in that area, the paper deals with provisions more applicable to Deccan trap area which covers Maharashtra, Madhya Pradesh, Gujarath, Saurashtra, Karnataka and Andhra Pradesh. The paper also suggests some modifications to the codal provisions. Case studies are given which include some success stories of proper investigations and some failure stories.

2. DECCAN TRAP BASALTS - A Few Special Features

In the Deccan Trap two main types of basalts occur : the compact or non-vesicular basalts and the amygdaloidal basalts. The common cavity fillings are zeolites producing white spots. The important difference between them is that the compact basalts are well jointed while the amygdaloidal basalts are unjointed. Jointing in rocks plays a very important role in foundations. Chlorophaeitic basalts (amygaloidal or compact), are also common. Many chlorophaeitic basalts are prone to rapid deterioration on exposure to atmosphere. Hydrothermally altered basalts are often in poor condition. The black and red tachylytic basalts, on exposure to atmosphere, disintegrate into powdery material. Volcanic breccias is another common variety and needs careful consideration. Dykes are also very common in Deccan Traps. These dykes in Deccan Traps are usually harder than surrounding but are very closely jointed. Resting foundations partly on dykes and partly outside is risky (central pier of CBD arm of Konkan Bhavan flyover). Vertical or steeply inclined fractures along which no movement has taken place are common in some parts of the Deccan Trap area. Many of them are quite tight. Water seeping along the crack brings about decomposition of the basalts producing a zone of vertical sheet jointing varying in width from a few centimeters to a meter or more. A number of fractures were met with on Kharpada bridge and were required to be dealt with indivisually.

3. OBSERVATION AND PRECAUTIONS DURING DRILLING

Core drilling is done by two methods : diamond drilling and calyx drilling. Of these diamond drilling is the most efficient process.

3.1 Minimum depth of drilling

There is no uniform practice about depth upto which core drilling is carried out. Practice followed is to drill about 3m in hard rock and 5m in soft rocks. Specifying depth of drilling based on hard rock and soft rock alone is unscientific.

In amygaloidal baslats three types of weaknesses may occur. One is cavities which occur due to escape of gases. These may be varying in size from minute to huge in which even a person can stand (Pune). In second, weathering takes place along various sets of joints. This weathering results in insitu boulders. The thickness of clayey weathered zone in between two insitu boulders may be a couple of meters also (Nagni bridge on Godavari). Third is pot holes which are formed in the Deccan Trap rivers, due to rejuvenation. These pot holes may be couple of meters in diameter (Mula at Mandave). It is necessary that the drill hole as well as the foundation must go beyond these weaknesses.

3.2 Location of Bore Holes

Bore holes as far as practicable, shall be located at the exact foundation location of every foundation. It is Authors' experience that variation of even two meters changes the subsurface geology. At Kharpada bridge on NH-17, drill hole for Pier No. 12 was 4.5 m away from the actual foundation of the Pier. The rock met within the pit was totally different than that met with in the bore. In case of Konkan Bhavan flyover, drill hole for one pier happened to be at correct position where dyke existed. If it was taken 2 - 3 m towards south, picture would have been totally misleading.

3.3 Care During Drilling

To ensure that drilling data are not misinterpreted and also that valuable data are not lost, certain precautions have to be taken during drilling and observations carefully recorded as described in PWD Handbook of Maharashtra. All the water that is fed into the drill comes back to the surface if, the rocks being drilled through, are water tight. If, however, the drill is passing through pervious rocks the water will leak into them and will not return to the surface. This drill water loss may be complete or partial depending on the nature of the rocks. As drill water loss indicates a leaky zone all drill water losses must be carefully recorded during drilling. Observing carefully the colour of drill water is important. Rate of drilling of each run gives in-valuable information. Experience shows that these important requirement are usually neglected. It is always important to know exactly where weak zones occur and what their nature is. But, routine drilling procedures will not provide adequate information on this vital point. In such cases another hole close to previous one is to be drilled in short runs in weak zone. Another alternative is to carry out nearly dry drilling at a very slow rate. Both these methods were adopted on Kharpada bridge on NH-17.

3.4 Length & Number of Pieces of Core

In hard but jointed rock the core recovery may be very good, and consideration of the core recovery alone will lead to the conclusion that the rock is good. This, however, may be wrong, as because of its fragmented condition, the rock will not be good from the engineering point of view.

3.5 Preservation of Core Pieces

The cores of some rocks such as tachylytic basalts (GERU), Volcanic breccias with tachylytic basaltic lava matrix, chlorophaeitic basalts, shales will disintegrate. Therefore, the cores of such rocks must immediately be coated with wax. This was done and was found very useful on Konkan Bhavan flyover and Kharpada bridge.

3.6 Mechanical Fractures

Core would normally break along preexisting divisional planes only. However, due to vibrations during drilling, particularly with a defective machine or defective operation, core may also break even at places where joints do not exists. It is necessary to distinguish between fractures due to jointing and mechanical fractures, which can be done by examining the fracture surfaces.

3.7 Corelogs & Lithologs

All the information gathered during drilling is to be recorded in corelog form. The core log serves as the basic record. A litholog is prepared from the core log to present the information contained in the core log, in a readily intelligible form. Core logging and preparation of lithologs and graphic logs require not only geological expertise of a high order, but also skill in interpreting geological data for engineering purposes and hence these should not be attempted by anyone except an experienced engineering geologist or an Engineer trained in this respect.

3.8 Interpretation

Usually probable founding stratum and its level is not known before drilling. Decision about founding level, SBC and buyoncy can be taken only after proper interpretation of the core log which is usually done later when drilling is terminated. At such time number of alternative types of foundations or alternative levels with different SBCs can be specified and most suited one can be chosen. This was done for 6 to 7 foundation of each of Konkan Bhavan and Chhedanagar flyovers and Kharpada bridge on NH-17. Suitability or otherwise of a particular type of foundation such as open, well or pile can be decided after interpretation of core log. In case of two foundations of Kharpada bridge open foundation was recommanded instead of piles and for three, piles instead of open. Type of foundation need not depend on rock levels only. More precautions are needed for well foundations. Sinking wells through rock is a very costly and time consuming process. It is also risky. Sinking wells through any rock is, therefore, resorted to only when it is unavoidable. In case of pile foundations, further more precautions are necessary. In closely jointed rocks conclusions from normal drilling may, indicate need to go down in rock. Very heavy chiselling will be required for reaching such levels which is costly, time consuming and risky. Where permenent liners are provided, they refuse to go below a certain level and while doing chiselling below the bottom of liner, collapses may occur making piling more and more difficult and complicated and more vulnerable for failure (Kharpada).

Codes need to give better and scientific guide lines upto where drill hole should go. It is felt that the drilling shall be continued at least six meters in rocks giving consistant recovery. Little extra expenditure on extra depth of drilling may result in ease and economy during actual execution.

4. COMMENTS ON SOME CODAL PROVISIONS

4.1 Type of Drilling Equipment

IRC-78/1983 or any other code does not specify type of equipment to be adopted. Three types of equipment are currently available; single tube, double tube and triple tube. Single tube unit is mostly used which needs to be prohibited. Most of the NITS, these days, provide for double tube boring but in practice it is not implemented. Even for very large projects costing crores of Rupees single tube drilling is being adopted. In some regions, triple tube drilling may be needed where weak rocks such as tuff, shale etc. occur. Code needs to specify the type of equipment to be used.

4.2 Depth of Drilling in Rocks

IRC 78/1983 (Cl.704.5.3) provides that "exploratory drill holes may be drilled into the rock to a depth of about 3 meters to distinguish a boulder from a continuous rock formation. A minimum depth of 3 meters in sound rock is recommended. Normally the drill hole shall pass through the upper weathered or otherwise weak zone, well into the rock." Firstly transported boulders need to be distinguished from insitu boulders. As discussed hereinabove, in case of latter, location and thickness of weathered zone will be more important. Secondly, for identifying a boulder, depth need not be a criteria. While saying that a certain depth in sound rock shall be drilled, question is what is a "Sound rock" and who shall decide. At present such decisions are usually left to the driller. Decision where to stop needs to be taken by a trained Engineer or an Engineering Geologist.

4.3 Characteristic Strength of Rock Mass

Cl.704.5.1 of IRC-78 recommends that for arriving at the characteristic strength of the rock mass, reliance be placed more on insitu tests in comparison to laboratory tests. Actually, no insitu test which can give real picture of stratum below is available. Permeability tests will indicate to some extent, nature and material available in joints. Compressive strength, specific gravity and water absorption are the laboratory tests which can be well relied upon if done and interpreted properly.

The clause further states that an Engineering Geologist be associated in the exploration program. In practice it is observed that this is very rarely done. One reason could be ignorance of importance of this requirement. Secondly, probably, such services are not available. It is therefore necessary that all engineers concerned need to be specifically trained in this respect. Such courses need to be designed and implemented on war footing.

4.4 RQD (Rock Quality Designations) Table-5 of Appendix-1 of IRC-78/ 1983.

Firstly the definition needs correction. The correct definition shall be RQD in % = length of the core between joints which are 100 mm and longer devided by length of run. Mechanical fracture of core needs to be properly differentiated from joints. Secondly the concept needs modification. Taking an extreme example of 100% recovery, RQD will be zero if there are 11 No. of pieces between joints of 9.1 cm length whereas it will be 100% if there are 10 number of pieces between joints of 10 cm length. This is ridiculous. It is felt that concept of modified RQD needs to be introduced which can be defined as percentage recovery divided by number of joints rounded to nearest integer and shall be designated as a number. Thus in the former case as above RQD will be 9 and in the latter it will be 10 and will give a more realistic picture.

4.5 Weathering and hardness

Tables 2 and 3 of appendix 1 of IRC 78 give guidelines to decide extent of weathering and hardness. Although extent of weathering and hardness can be decided from these tables, it is difficult for the field Engineer to decide what to do with it. Recommended range of SBCs need to be given in such cases as has been done in table-1.

4.6 Pressures on foundations

Clause 706.2.1 Recommends FOS for rocks : It is felt that the FOS (Factor of Safety) recommended for rocks, particularly for Deccan Traps, can be reduced when investigations and interpretations are properly carried out. While specifying FOS for pressures on foundations, differentiation needs to be made for short and long spans and also for simply supported or continuos spans.

4.7 Scour

No specific provision is made in codes (nor such is possible) for scour in rocks. This is true for both the conditions either when rock is exposed or it is above the calculated scour level considering actual bed material. A provision is always made in NITS that scour shall be considered upto rock. This is very dangerous. The nature and structure of rock so met with needs to be carefully studied and possibility of scour in it is estimated. Foundations levels will then have to be decided based on such estimation. Some guidelines need to be given by codes.

4.8 Depth of Embedment in open foundation

Provision made in cl.705.3.1 of IRC-78/ 1983 is more logical then before since now it defines Hard rock and Soft rock. Still, some more clarification is needed. If foundation has to rest on hard rock and if there is soft rock over it question is whether equivalence of soft rock can be taken while deciding embedment and if so how much. Secondly, minimum depth of rock of required capacity needs to be specified. There are cases where hard rock is overlain by soft rock and in an attempt to provide required embedment, cover of good rock available over soft rock is reduced and the foundation becomes unsafe (Bridges on down stream of Ghod and Chaskaman Dams in Maharashtra).

5. CONCLUSIONS

1) Importance of proper subsurface investigations need to be repeatedly explained and insisted upon all Engineers; from clients, from Consultants or Project Management Consultants and Contractors.

2) Codal provisions need a review and while doing so a proper Engineering Geologist need to be associated with.

3) A special training needs to be imparted to all Engineers concerned so that interpretation of investigations is properly done.

6. ACKNOWLEDGMENT

The authors are thankful to all the Chief Engineers of PWD and Irregation department for allowing reference to various cases referred herein.

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Establishment of Foundation Design Parameters for Limestone

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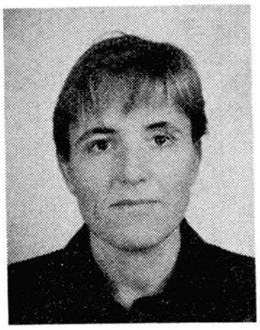

Ms. Feld, born 1964, graduated from University of Florida with a Master in Geotechnical Engineering. Since 1991, she has been involved with marine structures and has a profound knowledge of dynamic soil-structure interaction. She has specialized in soil mechanics and foundation engineering with special emphasis on finite element analysis.

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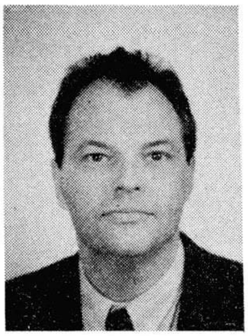

Mr. Petersen, born 1958, graduated from the Danish Technical University in Copenhagen with a Bachelor in Geotechnical Engineering in 1982. Following research at the Technical University and a research-fellowship at the Norwegian Geotechnical Institute, Mr. Petersen joined the private industry in 1985 for participation in, a. o., major bridge projects.

SUMMARY

For the establishment of the design parameters for the foundation design for the stay bridge and the immersed concrete tunnel for the 15 km long Øresund Link Project between Sweden and Denmark, a geological model was first established. The geological model included the method of deposition of the Copenhagen Limestone, and was developed assisted by seismic profiling, boreholes, geophysical borehole logging and visits to quarries.

Based on the understanding of the geological and the depositional models and the assumed behavior of the foundations, series of large scale plate bearing tests and laboratory tests on 0.5 m samples were performed. Further, thin sections of the undisturbed limestone and limestone samples from the plate bearing tests were microscopically investigated after impregnation with fluorescent epoxy.

The test results from the active, passive and shear failure tests showed anisotropic strength conditions for the limestone.

The investigations resulted in a physical understanding of the failure mechanisms and a possible way of deriving the design parameters considering the anisotropic behavior of the Copenhagen Limestone, and distinguishing between active, shear and passive failure conditions.

Establishment of Foundation Design Parameters for Limestone

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Ulf Bergdahl

Swedish Geotechnical Institute. Sweden

1 THE ØRESUND LINK PROJECT

The design and construction of the fixed link across the Øresund between Denmark and Sweden was initiated in 1993. A Joint venture between RAMBØLL, Scandiaconsult, Tunnel Engineering Consultants and Sir William Halcrow named the Øresund Link Consultants were nominated house consultant for the client, Øresundskonsortiet, a joint Danish-Swedish company.

The fixed link comprises a 3510 m long immersed tunnel, a 4055 m long artificial island and a high bridge and approach bridges of a total length of 7845 m. The combined road and rail link is scheduled to open for traffic in the year 2000.

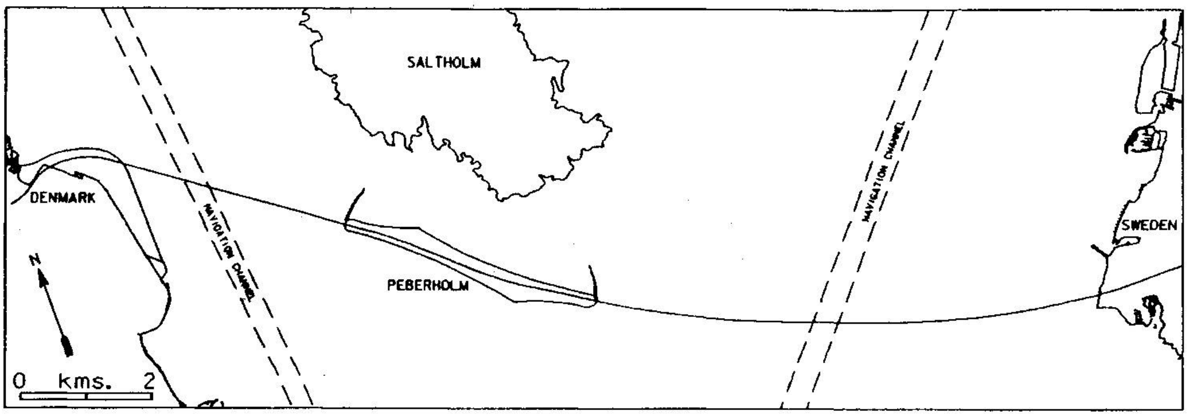


Fig.1 Alignment for the Øresund Link between Denmark and Sweden

2 METHODOLOGY FOR ESTABLISHMENT OF GEOLOGY AND PARAMETERS

Based upon existing knowledge about the geology and the stratigraphy of the Link Area, an Existing Model, ref. Fig.2, for the geology was established. Via this Existing Model additional geological and stratigraphical investigations were planned and executed. These investigations constituted reflection seismic and refraction seismic surveys and borehole logging for assessment of the stratigraphy, performance of core drillings for collection of samples for geological and rock mechanical description and performance of coccolith analysis, and performance of test pits inland on the Danish side, in the Sound and inland on the Swedish side, and investigations of a limestone quarry on the Swedish side. The test pits were performed to extrapolate the detailed investigations in the Sound. These investigations resulted in an Updated Model, ref. Fig.2, for the geology and the stratigraphy. To further assess the rock mechanical parameters and the foundation design parameters for the Copenhagen Limestone large size plate load tests, 0.5 m size laboratory tests and thin sections analysis of the limestone were performed. This finally led to the Updated Model and the Design Parameters for the Copenhagen Limestone. The process and the various activities within the process is presented schematically in Fig. 2., and described in further details below.

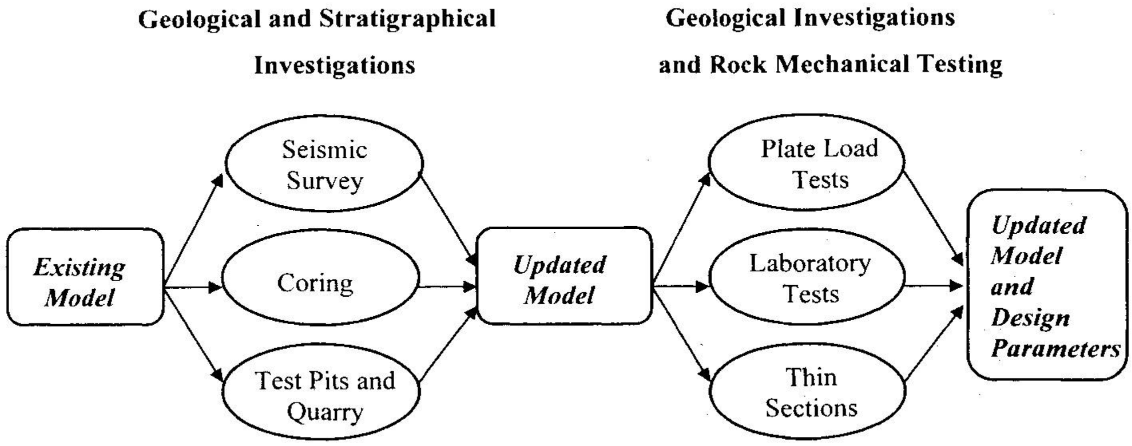


Fig. 2 Methodology for Establishment of Geology and Parameters

3 UPDATED MODEL

3.1 Seismic Survey

In 1993 high resolution seismic surveys were performed in the Link area. The interpretation of the surveys led to mapping of three seismic units separated by two boundaries. The units were interpreted to represent Quaternary glacial deposits, prequaternary, Danian Copenhagen Limestone and Bryozoan limestone. The interpretation changed the structural model in the area from dominantly faulted to dominantly folded, and contributed hereby significantly to a consistent geological model for the Link area.

3.2 Coring

The boring campaign for the Øresund Link comprises an extensive number of borings (core drilling, vibrocores etc.) performed prior to the first tenders for major contracts, forming part of the background information for the geotechnical basis.

Geophysical borehole logging was implemented to the engineering geological and hydrogeological investigations of the Danian limestone. Since the commencement of the offshore investigations in 1992, a large number of geophysical logging types were applied to the exploration of the boreholes. The logging methods respond to the variation in conditions and parameters of the limestone, such as: sediment composition, bulk density, porosity, permeability, seismic velocity and salinity. Based on the characteristic frequency of the strongly indurated high density beds, the observations made it possible to create a stratigraphical sequence for the Copenhagen Limestone [2].

3.3 Test Pits and Quarry

Test pits were performed inland on either sides of the Sound and in the west part of the Sound. The test pit on the Swedish side was located at the site for the performance of the large scale plate bear-ing tests. The test pits enabled a thorough geological, stratigraphical and rock mechanical de-scrip-tion of the limestone, which, in conjunction with the 60 m deep pit for the quarry at Limhamn in Sweden, gave the opportunity of a correlation across the Sound, of the encountered limestone strata. Later this correlation was extended and refined by the results from the offshore core drill-ings and the geophysical investigations.

4 UPDATED MODEL AND DESIGN PARAMETERS

4.1 Plate Load Test

As part of the site investigations an extensive series of plate load tests were performed at Lernacken close to the landing area on the Swedish side of the Sound. A total of 17 plate load tests were conducted with plate sizes varying from 1.0 to 3.0 m^2. The tests were performed in three different ways, namely as vertical active tests, horizontal passive tests and horizontal shear tests.

The tests, including both static, dynamic and cyclic loading giving both drained and undrained behavior and strength-deformation relations, are described in detail in [2] and [4]. The principle of the different bearing capacity tests are illustrated in Fig. 3.

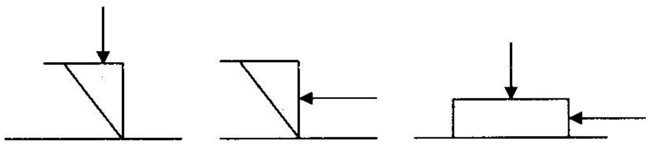


Fig. 3 Definition of active, passive and sliding cases

The different load situations were covered by different test types and different ways of applying the load, elucidating the influence of the anistropic behavior of the horizontally layered limestone, the effect of fissures and other anomalies in the limestone, the effect of cyclic loading and unloading, the effect of strain and strain rate, the effect of unloading and the degradation of the limestone. The tests were performed on slightly indurated and harder limestone

4.2 Laboratory Test

To study the behavior of the limestone at variable stress conditions and to supplement the plate load tests a series of triaxial tests on 0.5 m diameter and 0.5 m high samples were performed as well as direct simple shear tests on 0.5 m diameter samples, ref.[4]. The triaxial testing program included both active and passive shear tests performed on anisotropically consolidated samples. The shearing was performed under both drained and undrained conditions, and both with - and without an initial cyclic loading phase. The direct simple shear testing program included both undrained and drained shearing tests and dynamic failure tests. As it was assumed that the slightly indurated limestone would have a major impact on the bearing capacity of the limestone, all tests were performed on this type of material. The results from the plate load tests and the laboratory tests showed, in spite of the influence from the differences in induration of the limestone, that the strength parameters obtained in the laboratory tests are lower than the parameters obtained in the plate load tests. It was, however, assessed that the parameters from the laboratory tests would be representative for the limestone.

4.3 Thin Sections

To study the physics of the limestone at failure, both undisturbed reference samples and samples of material subjected to shearing at the plate bearing tests were extracted, ref [3] and [5]. Both the reference samples and the sheared samples were impregnated under vacuum by epoxy, to stabilize the samples and to enable a microscopic study of thin sections under UV-light. In the microscopic study the following subjects were analyzed: changes of the porosity, fissures and patterns of fissures, fissuring and movements of

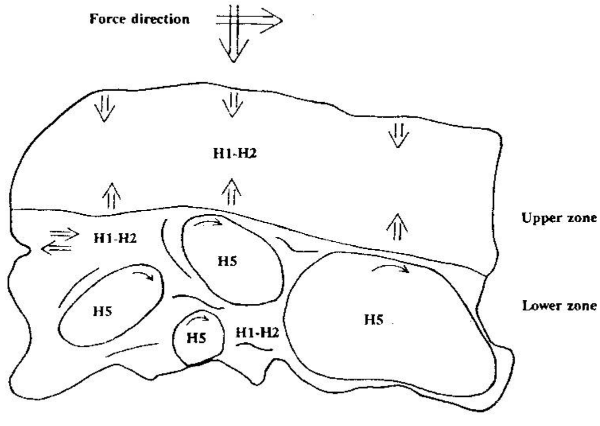


Fig. 4 Principal Sketch of Limestone Sample.

shells and more indurated limestone parts, ductile flow and the directions of deformations of the material within the samples, as depicted in Fig. 4. In the analysis comparisons were made, both between the reference samples and the sheared samples, and within each individual sheared sample. This study provided the information that for slightly indurated limestone without any content of hard nodules, the limestone will experience a reduction of the porosity leading to a stronger material which eventually will endure a higher failure load than limestone with some content of hard nodules. The explanation is that the nodules will prevent the compaction of the limestone during shearing, leading to unchanged strength parameters for the slightly indurated material, and to movement of the nodules which will further cause fissures to open and to destabilize the material. This leads to a comparatively lesser bearing capacity of the material with the content of nodules

5 DETAILED GEOLOGY AND FAILURE MECHANISMS

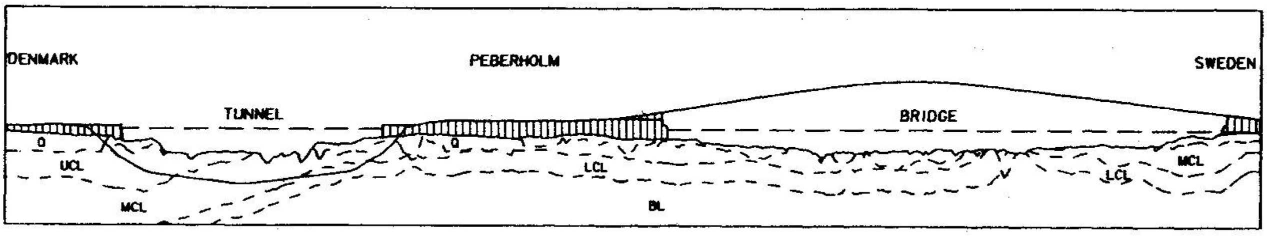
5.1 Stratigraphy

In the Øresund area, the Danian limestone series consist of an upper unit of Copenhagen Limestone up to 40 m thick, overlying Bryozoan limestone.

The Copenhagen Limestone can be subdivided into three stratigraphical subunits, named Upper, Middle and Lower Copenhagen Limestone, the variation and location of the subunits are shown in Fig. 5.

The subunits were identified and mapped through stratigraphical studies based on the geological description of core samples and samples from test pits, supplemented by geophysical logs and reflection seismic [2].

The degrees of induration varies from unlithified to very strongly indurated, due to varying degree of calcite cementation and silicification. The strongly indurated limestone typically occurs as 0.2-0.4 m thick layers intersected by less indurated rock, however benches up to 1.5 m have been found, as have flint layers typically of a 0.5 m thickness. At some locations, the upper part of the limestone series, has been disturbed by glacial processes.



Q = Quaternary/UCL = Upper Copenhagen Limestone/MCL= Middle Copenhagen Limestone/LCL=Lower Copenhagen Limestone/BL=Bryozoan

Fig. 5 Longitudial Profile of the Geological Section of the Øresund Link Alignment.

5.2 Anisotropic Behavior

The Copenhagen Limestone has a cemented structure, but is at the same time anisotropic due to layering and fissuring. The behavior of the limestone can be compared to "an old brick wall". Vertically the brick wall has a high bearing capacity, but against horizontal loading or sliding, the mortar will be the weak element. Correspondingly the unlithified and weaker layer in the Copenhagen Limestone is the fragile area. High stresses or large strains will lead to degradation of the material and the limestone will change into a silty cohesionless material.

The test results from the active, passive and shear failure tests confirmed the anisotropic strength conditions for the limestone and the structures. In any major occurrence of unlithified limestone, excess pore pressures may develop during shearing as a consequence of contractancy.

5.3 Failure Mode

The investigations resulted in a physical understanding of the failure mechanisms and a possible way of deriving the design parameters considering the anisotropic behavior of the Copenhagen Limestone, by distinguishing between active, shear and passive failure conditions.

From the insitu plate load tests performed at Lernacken [4] it became evident that the failure mechanism of the limestone could be divided into active, shear and passive failure conditions, as illustrated in Fig. 3.

In Fig. 6 the principal failure conditions in the mechanism are depicted. This failure surface has previously been introduced [1].

The shear strength at any part of the failure surface can be given by : $\tau_F' = c' + \sigma' \cdot \tan\phi'$.

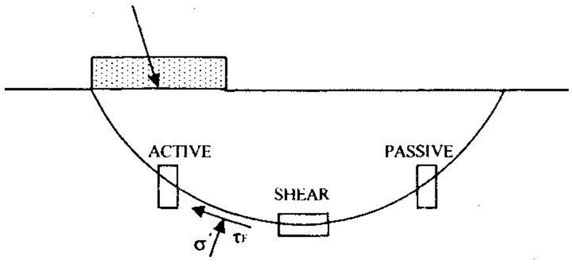


Fig. 6 Principal Failure Conditions

6 CONCLUSION

In conclusion, the applied methodology, by firstly establishing a geological model, secondly obtaining a physical understanding of the behavior and failure mechanisms of the limestone, and finally assessing the strength and deformation parameters through an interpretation of the performed tests based on this physical understanding has succeeded. The use of the methodology has led to design parameters which, to a far extent, represents the true nature of the Copenhagen Limestone.

7 ACKNOWLEDGEMENT

The authors are grateful to the Client, Øresundskonsordiet, for permission to publish this paper. Furthermore we wish to thank our joint venture partners. The options expressed in this paper are exclusively the Authors.

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