# Transcendent foundation solutions -

**the cross pollination of scientific and industrial disciplines to overcome economic and technical challenges in foundation of large bridge structures**

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**Abstract:** Where the medical profession buries its failures, the geotechnical profession buries its successes. Hence, a lot of the fruitful interplay between the structural and geotechnical engineers is not immediately apparent from the many imposing land mark structures seen all over the world. The success often relies on the ability of the designers to combine existing scientific and industrial disciplines in a new fashion or adapt to new technologies emerging. This cross pollination of scientific and industrial disciplines to achieve feasible and economically viable truly transcendent foundation solutions really requires lateral thinking. The paper describes a number of foundation solutions where part or all of the solution could be considered transcendent and where the authors have been involved. The authors believe that the realisation of large scale structures with due consideration of soil-structure interaction is the most appropriate way to pay tribute to Terzaghis' contributions to the evolvement of soil mechanics and foundation engineering as a modern science.

## 1. Introduction

The basics of engineering geology and geotechnical engineering have changed surprisingly little since the earth-shattering appearance of Terzaghi's "Erdbaumechanik" in 1936.
However, the environment in which the geotechnical and structural engineers operate has changed very considerably. Every new decade has seen dramatic changes in:

- field testing techniques
- laboratory testing techniques
- numerical and physical modelling techniques and capabilities
- constitutive modelling
- monitoring techniques
- construction techniques
- information and communication technologies(ITC)

Moreover, the work of the "engineer" has changed with the changes in society. The work environment is becoming increasingly more competitive and complex as evidenced by "fast tracking", "turn key projects", "design build and operate", "public-private-partnering" etc.
The role of the geotechnical engineer merges and separates with that of the structural engineer and calls for cross-border knowledge and understanding. The survival of consulting companies in the competitive environment depends heavily on lateral thinking and creative utilization of soil-structure interaction.
Cutting edge technologies by themselves are not enough. The cross pollination of bordering disciplines and technologies are necessary to produce feasible and cost effective solutions.
The paper illustrates the importance of lateral thinking in addressing the foundation of major fixed links and bridge structures world-wide.

## 2. Fixed link concepts

### 2.1 Feasibility studies

Many of the fixed links established are literally cross-border projects. Thus, the challenges are often non-technical. As pointed out by Steenfelt (2000) this places heavy demands on the capabilities of the engineers. The ability to understand and communicate non-technical issues and to translate "soft demands" into sustainable hard-core bridge solutions is a must for the successful engineers of the Millennium.

Many of the most challenging foundation solutions are associated with the construction of fixed links, where "landmark" structures play an important role. To make ends meet "feasibility studies for fixed links" is the most vital stage where cross pollination of extremely varying disciplines and professions is necessary at the very start of the project.
Geo-engineers, architects, structural engineers, biologists, economists, traffic planners, lawyers, 3-D animators, financial experts and many more have to work together to achieve the common goal of uniting land masses and populations.
This philosophy has successfully been developed and honed in Denmark, the homeland of the authors, where internal links

- the New Lillebælts Bridge
- the Farø Bridges
- the Fixed Link across Storebælt,

and transnational links

- the Øresund Link
- the Femer Belt Link

have provided the backbone for addressing the challenges on a truly international scale, as described in the subsequent sections.
Presently the fixed link concept is applied to two prestigious and very challenging projects in the Middle East:

- the Qatar-Bahrain Causeway - a 45 km world record length for a fixed link
- the Kuwait City Subiyah Causeway (shown in Fig. 41)

## 2.2 Incommensurable quantities?

In deciding the feasibility of fixed links and their components - notably major bridge structures - a large number of "issues" must be quantified for comparison and risk assessment. This calls for interaction of numerous disciplines, united by the mathematics of reliability theory.
One major difficulty is that this inherently requires comparison of incommensurable quantities.
One example is the environmental issues in relation to the establishment and operation of a fixed link.
From near non-existence in the economically roaring 1960'ties the environmental issues play an ever increasing role in the decision process, prompted by movements such as Greenpeace, "Green parties" and a growing public awareness of the need to protect Mother Earth.
This has played a very major role in the handling and decision processes for the fixed link projects based in or extending from Denmark.
Environmental issues were raised during the construction of the Fixed Link across Storebælt with hardy protests from fishermen, marine biologists and others. This led to the initiation of a thorough investigation and monitoring programme.
Fortunately the dire forecasts regarding the marine wild life were proven wrong. Fishes, plants clams etc are now plentiful in the waters which seem to provide a better habitat now than before the construction of the link started.
When the Øresund Link was decided politically in 1993, the environmental studies were initiated concurrently with the other investigations to avoid public outcry and to gain the benefits from environmental management. Furthermore, the Swedish law called for a positive ruling by the "Water Tribunal" to approve the project.
The message was completely understood when the investigations for the future fixed link - The Femer Belt Link between Denmark and Germany - were initiated in 1995.
Hence, a three year environmental study was carried out concurrently with geotechnical and geological investigations.
In this way the political system ensured a very low-voice public protest and established a far better basis for the decision process.
When the two governments decide to go ahead with the project, this is believed to ensure a much better informed and smooth process with a positive public awareness and attitude towards the Link.
Strong opinion groups very loudly voiced their negative concern over the two completed fixed links, the Storebælt Link and the Øresund Link, before and during construction. Even vandalism was suffered by the contractors, particularly for the Øresund Link.
Today some seven years after the inauguration of the Storebælt Link the project is a huge success with re-payment well ahead of schedule due to much higher train passenger and car crossing numbers than forecasted. The protests have completely faded (except complaints over the toll fare). It seems likely that the same will happen with the Øresund Link (inaugurated in 2000) where the train traffic has met expectations, but where car traffic presently is lower than the forecast.

# 3. The Fixed Link across Storebælt

## 3.1 Components of the Link

The Storebælt Link is 18 km long and consists of three major components: A bored railway tunnel and a high level motorway bridge across the eastern channel, and a low level combined railway and motorway bridge across the western channel, cf. Fig. 1.


Fig. 1: The Storebælt Link (The Great Belt Fixed Link)

The task of developing cost effective foundation solutions in deep waters suited for fast track construction and the clay till soils in the area called for new concepts. In the past, massive foundation techniques were based on extensive and risky offshore operations (open dredge or pneumatic caissons). This had to be replaced by methods where the massive foundations could be pre-fabricated onshore and laid directly onto the sea bed in order to minimise offshore works and interference with the extensive ship traffic in Storebælt. The development of foundation concepts based on this philosophy was to a great extent inspired by technologies and designs from the offshore oil industry.

## 3.2 The West Bridge

The West Bridge is a 6.6 km long multi-span concrete bridge with 51 spans of 110.4 m and 12 spans of 81.75 m, cf. Fig. 2.
The superstructures consist of two haunched concrete box girders each supported on separate pier shafts and sharing a common gravity based caisson foundation, cf. Fig. 3.

Fig. 2: The West Bridge with the railway to the left

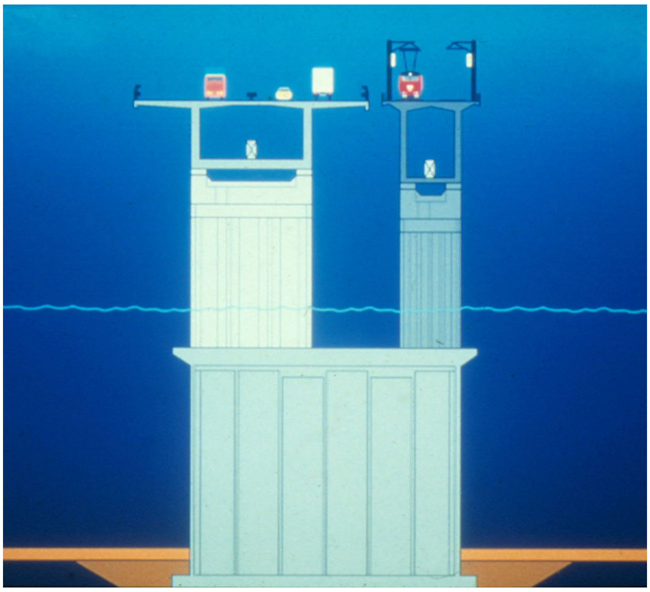

Fig. 3: West Bridge cross section

The overall idea behind the design and construction of the West Bridge was pre-fabrication of mega size elements erected by a huge purpose built crane vessel. This heavy lift technique offshore were well known from the erection of steel topsides for oil platforms, but on this project it was adapted to handling concrete elements in a scale never seen before.
Altogether 324 pre-fabricated units, comprising 62 caissons, 124 pier shafts and 138 bridge girders were cast in a yard close to the bridge site, cf. Fig. 4 and erected by the purpose built heavy lift (capacity 6,400 tonnes) catamaran crane vessel Svanen (The Swan) with overall dimensions 94 x 65 m and 65 m height, cf. Fig. 5.

Fig. 4: Pre-fabrication Yard


Fig. 5: Placing a caisson by Svanen

Fig. 6: Jack-up platform

The caissons placed in water depths up to 30 m have a 1.2 m thick bottom slab and simple vertical walls in a cellular pattern suitable for slip-forming.
In order to place the caissons directly on the sea bed with great accuracy it was necessary to construct carefully compacted and levelled stone beds varying from 1.5 m to 4.0 m in thickness.
This was done by means of a multi-purpose jack-up platform, cf. Fig. 6, designed to handle all operations such as foundation inspection, placing and compaction of stones and surface levelling to very strict tolerances allowing the caissons to rest directly on the stone bed surface without subsequent under base grouting.
Lowering of the caissons onto the stone beds required close surveillance by the survey system giving on-line information to the crane operator about horizontal position, differences relative to exact position, and suggested remedial actions. During lowering the caissons were filled with water pumped in either through holes in the top or entering through flood valves in the bottom slab designed to ensure that the stone beds were kept intact during this operation.

### 3.3 The East Bridge

The East Bridge is 6.8 km long comprising a suspension bridge with 1624 m main span and 535 m side spans and approach bridges on both sides with spans of 193 m. The navigational clearance of the main span is 65 m, cf. Fig. 7.
The superstructures for both the main suspension bridge and the approach bridges consist of closed steel box girders. All other bridge elements are in concrete including the towers of the suspension bridge rising to 254 m above sea level.
The overall idea behind the design and construction of the foundations for the East Bridge is basically the same as used for the West Bridge,

Fig. 7: The East Bridge

pre-fabrication of mega size elements onshore and subsequent transport and placing these elements directly on the seabed. However, due to the extraordinary large size of the foundation caissons for the towers, anchor blocks and the seven largest approach span piers a different scheme as compared to the West Bridge had to be used for these special elements.

The scheme selected was based on constructing the caissons in a dry dock, towing them to the bridge location where they would finally be sunk by controlled water ballasting to stone beds prepared in advance. It was obvious to choose this technology as it had been used successfully for construction of more than 20 major concrete oil production platforms in the North Sea at the time of planning the East Bridge.

Two separate dry docks, cf. Fig. 8, were purpose made for the project in a harbour 30 nautical miles from the bridge site. Casting of the cellular 78 m x 35 m and 20 m high caissons for the pylons, and the two anchor block caissons and the seven largest approach span piers, respectively, took place in the two docks.

The anchor block caissons have a rectangular base 121.5 m long and 54.5 m wide and 16 m high divided into approximately 100 cells.

The caissons weighing about 50,000 tonnes were towed by tug boats to the bridge alignment and sunk unto the stone beds with great accuracy, cf. Fig. 9. Finally, under base grouting was performed followed by ballasting with sand.

It was essential to confine the grout and ensure that the void between the stone bed and the bottom slab was completely filled.

Therefore, the underside of the base was divided into smaller compartments with roof shaped surface by a system of small skirts designed to penetrate into the upper 300 mm non compacted part of the stone bed.

The main geotechnical challenge in connection with the anchor block design was the mere size of the horizontal force (~550 MN) to be transferred to the ground being beyond normal experience and Codes of Practise. Therefore, very thorough investigations and careful assessment of the safety by applying several independent analysis models were made.

As a result of the anticipated construction method requiring underwater excavation to remove unsuitable postglacial deposits it was expected that the underlying stiff pre-consolidated clay till being the primary foundation strata would be disturbed and have reduced sliding resistance.

Fig. 8: Anchor block caissons in dry dock

Fig. 9: Placing of anchor block caissons

This question was thoroughly studied by conducting several series of large sliding box and direct shear box tests in the laboratory as well as 28 field sliding tests using 1.2 $m^2$ concrete blocks. A detailed description of these tests and the results for the sliding resistance achieved considering a variety of parameters is given in Steenfelt (1992).

To avoid sliding failure along a thin weakened zone of the excavated clay till surface several solutions for improvement of the soil-structure interaction were investigated such as short steel skirts penetrating the weakened zone and large diameter steel pile dowels. However, the preferred solution turned out to be two separate stone wedges with an inclination of $16^0$ to ensure that the combined vertical and horizontal load would be almost perpendicular to the stone/clay interface thus effectively reducing the shear stresses, cf. Fig. 10.

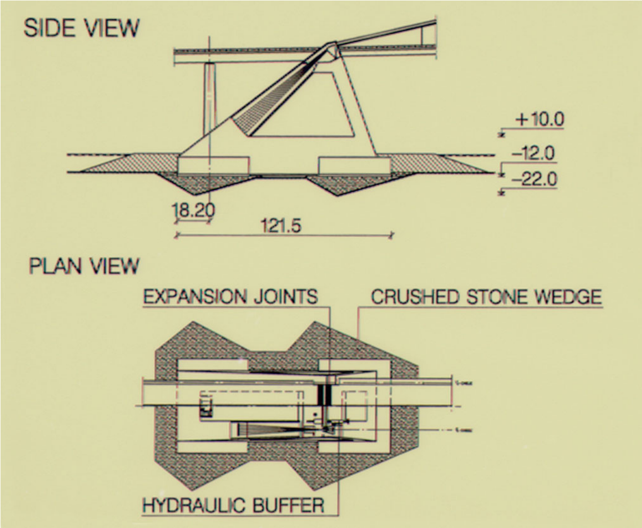


Fig. 10: Wedge shaped stone beds

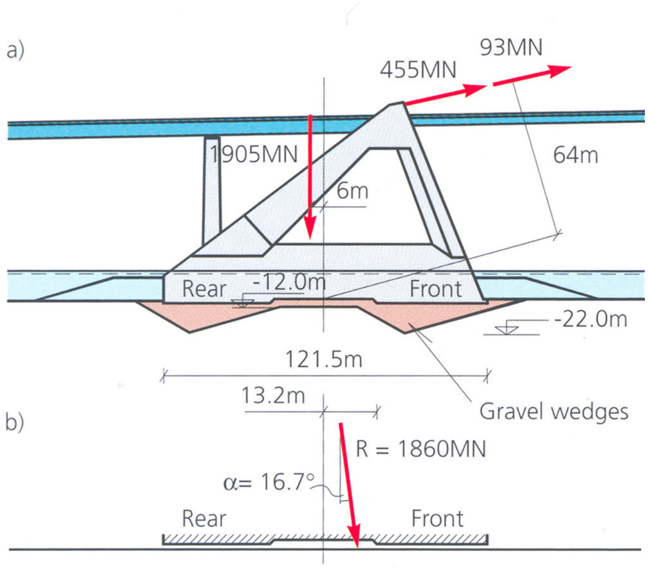


Fig. 12: Forces on anchor blocks

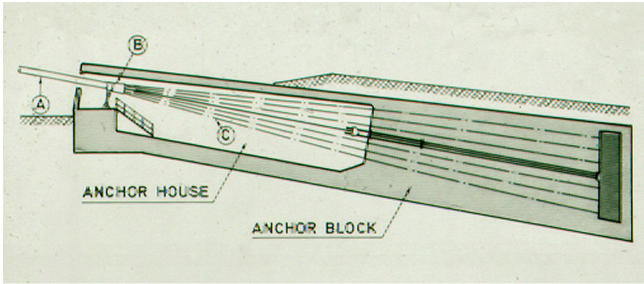


Fig. 11: Lillebælt anchor blocks

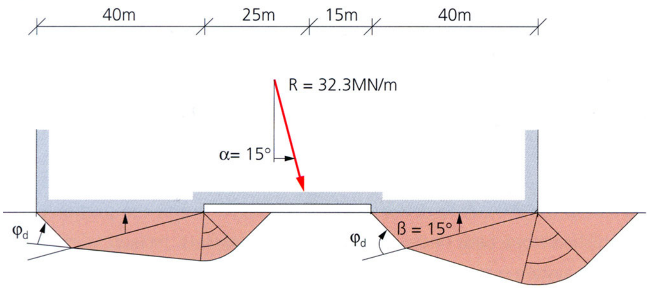


Fig. 13: Rupture figure in upper bound analysis

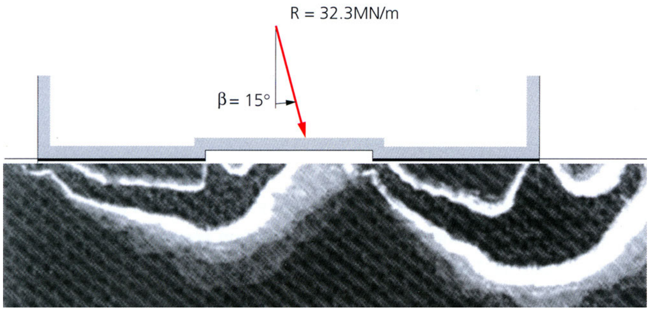


Fig. 14: Rupture figure from FE Analysis

The idea described above to reduce shear stresses in the weak interface between structure and soil was inspired by the solution developed earlier for the other large suspension bridge in Denmark, the Lillebælt Bridge. For this bridge the anchor blocks were designed as thick slabs each covering a base area of 3,300 m$^2$, the base sloping 10.4$^0$ with the horizontal, cf. Fig. 11.
The combined action of the horizontal cable pull and the self weight of the concrete slab and the soil ballast on top would then be almost perpendicular to the interface between the slab and the soil eliminating any risk of sliding failure.
Comprehensive soil-structure interaction analyses were made to determine the overall safety for transfer of the large resulting forces on the anchor blocks, cf. Fig. 12.
Due to the importance and complexity of the design three independent methods were used to determine the capacity of the anchor blocks. These comprised: upper bound theory, limit equilibrium analyses and finite element analyses.

The results of five benchmark cases tested were in the same range for all three methods (Sørensen et al, 1993).
The critical rupture figures determined for the upper bound method and for the finite element model are shown in Fig. 13 and Fig. 14.
It is noted that the critical rupture figures goes through the intact clay till, i.e. sliding failures through the stone beds or at the stone bed/clay till interface are not critical.

### 3.4 Project MOSES

On the morning of October 14, 1991 a sorry sight met the eyes of spectators at the tunnel ramp area on Sprogø, as seen in Fig. 15. The ramp area and the two eastbound tunnel tubes were inundated in what is referred to as the *Jutlandia incident*.

During a period of maintenance stoppage of the TBM Jutlandia a chimney-like connection to Storebælt formed allowing ingress of sea water into the tunnel. Both TBMs and tunnel tubes were successfully evacuated without loss of lives, but both ongoing mining operations had to be stopped for a long time (restart of first eastbound TBM in August, 1992) to allow remediation and to ensure that a similar incident would not occur again at any of the four TBM-positions.

In order to control the mining operations project MOSES *Method of Obtaining Safety by Emptying Storebælt,* was set in action. No doubt the acronym was coined with some knowledge of Eckersberg's wonderful painting seen in Fig. 16.

Project MOSES entailed large scale dewatering below the seabed with the objective to improve the safety of mining operations.

The immediate aim was to reduce the pore water pressure at the TBM axis by 3-4 bar in order to arrive at a nominal pore pressure of 3 bar at the tunnel axis, compatible with the use of compressed air for manned intervention in the TBM head. Furthermore it was important to improve soil stability at the cutter head and as a side effect to reduce the need for local dewatering at cross passages (more information may be found in Biggart et al, 1993; Odgård et al, 1993).

The first hint in the direction of the MOSES project was found when an onshore dewatering scheme was started in 1989 for the tunnel ramp area on New Sprogø. The horizontal extent of the dewatering was considerable as first experienced by the caretaker at Sprogø, as the sweet water well here dried out in October/November of 1989.

During the 1990-91 boring campaign for the East Bridge, it was confirmed that the extent of the Sprogø dewatering was large enough to produce down drags of the order 2-10 m in boreholes for anchor block west and approach piers some 1-3 km away from the ramp area.

In effect the Sprogø dewatering with a yield of 1100-1300 $m^3/h$ produced down drags of 25-34 m in the ramp area with a radius of influence of 3-4 km. This constituted the factual background for initiation of project MOSES.

Fig. 15: Inundated tunnels and ramp on Sprogø

Fig. 16: Moses at the Red Sea by Eckersberg

During July to October 1992 a conceptual scheme for sub-sea dewatering was developed and in August '92 the Danish Geotechnical Institute conducted preliminary investigations sinking a number of boreholes and conducting trial pumping for the MOSES project on Halsskov and Sprogø sides. This provided a platform for the initial test programme which allowed fundamental insight into the geologic-geotechnical-hydrologic-al conditions pertinent to the success of the scheme.

The well heads were prototyped and manufactured by the Danish Geotechnical Institute. After completion of a well, installation of well head, submersible pumps, pressure transducers, flow meter and discharge pipe the well was connected by seabed cables to the contractor's set-up. The MOSES set-up is shown in Fig. 17, with the marine spread and the wells.

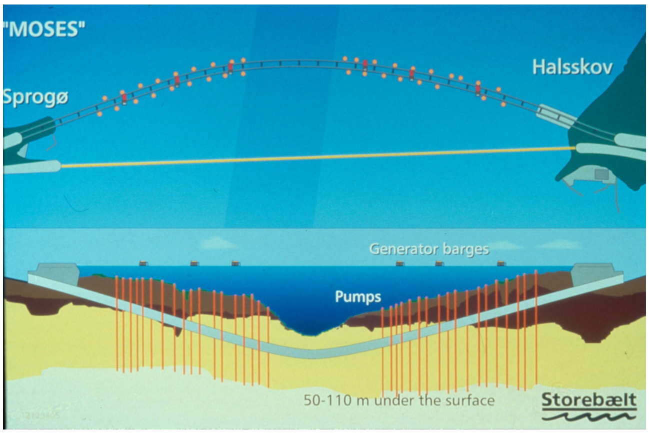


Fig. 17: Extent of MOSES operation for Storebælt

The original design objective, to reduce the water pressure to a nominal 3 bar at the tunnel axis, was by and large achieved by the MOSES project together with the secondary objective, to reduce the water inflow indicated by the site investigations and trials.
The lowering of the groundwater pressure furthermore improved the conditions for site investigations and reduced the extent of ground treatment required for the construction of the cross passages. Even under the Nadir a significant reduction in pore pressure was achieved.
With project MOSES tunnelling was resumed after the 1991 incident with rates of up to 134 m/week.
During the 30 months of operation of MOSES some 45 million $m^3$ of water was extracted from the Selandian marl and the glacial tills.
The costs of 27 M Euro may seem high. But the added safety and overall feasibility of actually completing the tunnels by reducing delays which might have occurred without MOSES in all likelihood recovered the costs many times over.
Project MOSES demonstrated that when client, contractor and designer meet in a co-operative approach then new borderline techniques can be successfully adopted for special applications

### 3.5 Storebælt Link Spin-off and lessons

I many ways the completion of the Storebælt Link was an eye-opener for the geotechnical engineers.
The challenges of the project necessitated a serious review of well established practices and techniques. But the well-winnowed Danish tradition of interplay between geotechnical engineering and neighbouring disciplines proved its value once again.
Part of the success of the project was ascribed to the establishment of a 3D geomodel, a transparent system for storing, evaluation and retrieval of the diverse data from geotechnical, geological, hydrological and geophysical sources.
The pit falls in mixing old and new data from different sources and cultures were apparent, as was the challenge of mixing local and international experience for the many nationalities involved in the project.
For the Danish geotechnical community at large the Storebælt Link project was the stepping stone to a number of initiatives. Apart from involvement in the offshore industry, this was the first project where in depth quality insurance - for better and for worse - was enforced by the client.
In general improved procedures and advanced testing set-ups were developed in the light of the international scope of the work, for understanding and handling soil types otherwise considered well known.
A more detailed description of the Storebælt Link may be found in Steenfelt (1993), Steenfelt & Foged (1994), Steenfelt & Hansen (1995), Steenfelt & Sørensen (1995), Sørensen et al, (1995).

## 4. The Øresund Link

The fixed motorway and railway link across Øresund extends just under 16 km between Kastrup on the Danish coast and Lernacken on the Swedish coast. The link's key elements are:

- an artificial peninsula extending 430 m from the Danish coast at Kastrup
- a 3510 m long immersed tunnel under the Drogden navigational channel
- a 4055 m long artificial island, "Peberholm", south of the island of Saltholm
- a 7845 m long bridge between the artificial island and the Swedish coast at Lernacken.

The location of the Øresund link appears from Fig. 18 and Fig. 19.

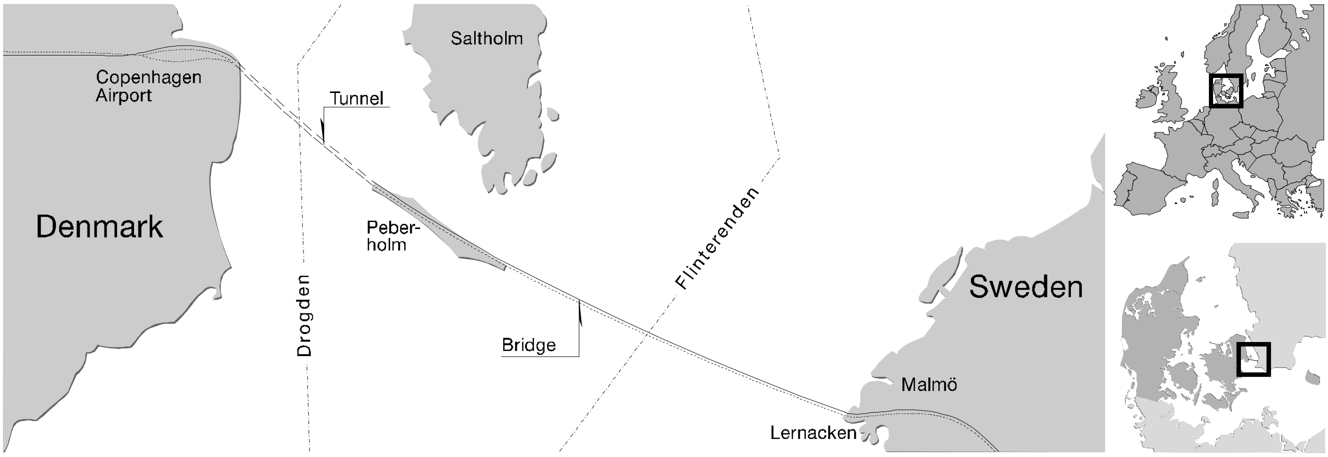


Fig. 18: Location plan of Øresund Link

Fig. 19: Aerial view of Øresund Link with Copenhagen airport to the left

### 4.1 The Øresund Bridge

The Øresund Bridge (Fig. 20) is divided into three main components:

- a western approach bridge 3014 m long leading up to the high bridge from the artificial island "Peberholm"
- a cable stayed high bridge 1092 m long, with a main span of 490 m crossing the Flinterenden navigational channel. The navigational clearance of the high bridge is 57 m.
- an eastern approach bridge 3739 m long leading down from the high bridge to the Swedish coast at Lernacken

The bridge superstructure is a composite steel-concrete structure with truss girders. The upper deck contains a four-lane motorway while a two-track railway is on the lower deck. The cable stayed bridge is the largest of its kind in the world carrying train and motorway traffic, cf Fig. 20.

The foundation design is based on large scale pre-fabrication of mega size elements according to the same principles used for the Storebælt Link (cf. Sec. 3). However, the completely different ground conditions in the Øresund area called for modifications of the concept. The principle of a typical pier foundation is seen in Fig. 21.

In contrast to the Storebælt Link stonebeds are not used. Instead, the caissons are initially supported by temporary "footprint foundations" in the dredged pits. Subsequently the voids between the base slabs and the limestone surface are filled with cement grout.

Like Storebælt, the Øresund Strait is an extremely busy, international shipping route. Hence, the key challenge for this project was to optimise the foundations to resist accidental loading by ship impact on the large number of piers.

Fig. 20: The Øresund Bridge

The design principles and geotechnical soil-structure interaction models developed to meet this challenge is considered a transcendent step forward in the optimised design against ship impact from large scale vessels.
The Øresund is shallow in the bridge alignment, with water depths of generally less than 8 m. The soil strata forming the sea bed consists of post-glacial sands, generally less than one m thick, and at a few locations of thin layers of peat or organic mud.

These deposits are underlain by 0 to 5 m of glacial clay till and glacial meltwater sands.
Prequaternary deposits, consisting of limestone of Danian age, form the bedrock in the bridge alignment. As seen in Fig. 22 all bridge pier and pylon caissons are founded in the Danian limestone, but predominantly in the Copenhagen limestone
A summary of the geology in the Øresund area is given by Knudsen et al (1995). A geological profile in the bridge alignment, with indication of pier and pylon caisson foundation levels, is shown in Fig. 22.

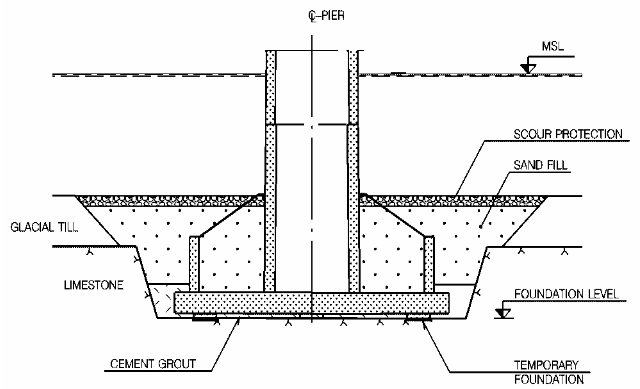


Fig. 21 Foundation of bridge pier

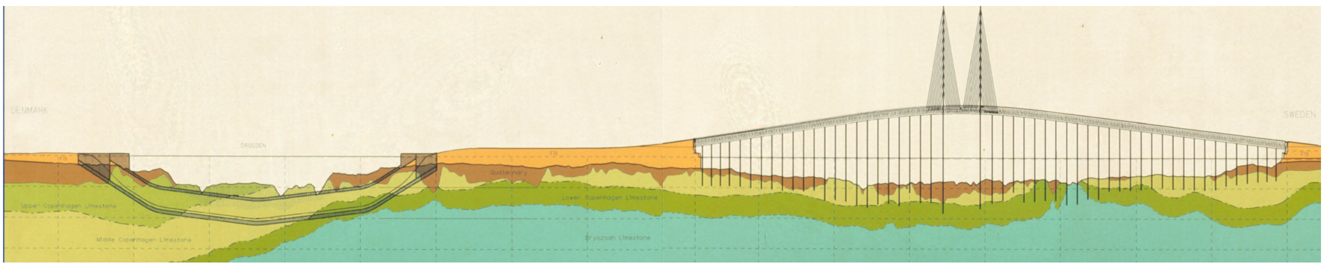

Fig. 22 Longitudinal profile from Copenhagen to Malmø in alignment (Light brown = fill, brown = glacial deposits; green, yellow, green = Upper, Middle and Lower Copenhagen limestone Units; blue = Bryozoan limestone)

### 4.1.1 Ground investigations

Prior to tender the Owner carried out geotechnical boreholes in the bridge alignment, supplemented by large scale site trials in the Danian limestone formation at Limhamn in Sweden. On the basis of these investigations the Owner defined the geotechnical design basis to be adopted for the tender, i.e. soil stratification and soil properties. The results of the pre-tender were entered into a geotechnical database (Geomodel) by the Owner.

For the detailed design the geotechnical design basis from the pre-tender ground investigations was verified by one geotechnical borehole and four DTH soundings at each pylon and pier caisson position. In these ground investigations particular emphasis was taken to verify that none of the following features existed, which, if present, could have presented problems in relation to the bridge foundation design:

– Highly crushed or fissured limestone.
– Extensive zones of unlithified limestone.

– Cavities due to solution of limestone (karstic limestone).
– Soft sediments deposited in depressions between mounds of bryozoas, especially in the transition between Copenhagen limestone and Bryozoan limestone.

A suite of geophysical logs were run in all boreholes and sounding holes. These logs comprised caliper, natural gamma radiation, neutron porosity, gamma density, and deep and shallow guard resistivity. In addition a flow log was carried out in all core drilled boreholes. The geophysical marker horizons defined in the pre-tender investigations, Klitten et al (1995), could generally be traced and correlated in all the boreholes and soundings, thereby facilitating the interpretation of the ground stratification.

No significant occurrences of any detrimental features were found during the detailed ground investigations.

### 4.1.2 Design for Ship Impact

The main geotechnical challenge was the size of the project and the design of the bridge piers and pylons for large ship impact loads (up to 620 MN). This fact led to very thorough evaluations of the feasibility of different calculation tools for the design. The main issues to be considered were the effect of weak horizontal layers embedded in the limestone and the effect of the limestone's peak/residual strength on the bearing capacity of the pier and pylon foundations.

The accidental limit state load situation (ship impact) was decisive for the design of the piers, except near the coast where the serviceability limit state was governing. Special measures were initiated during the construction period where numerous ships had to pass the construction area daily as seen on Fig. 23.

The large ship impact corresponds to strongly eccentric loading. Thus, the effective foundation width is reduced to some 3-4 m, even with high mobilisation of the passive resistance.

Therefore it was decided that the design should be based on more than one calculation tool. The following tools were examined to make a reliable basis for determination of the bearing capacity of the footings:

– Theory of plasticity, kinematically (upper bound) admissible solutions.
– Limit Equilibrium Analysis
– Finite Element Analysis

Fig. 23: Ship passing the bridge during construction

These methods were also used for the design of the Storebælt Anchor Blocks. The results of a comparative study were presented in Sørensen et al (1993). Here it was concluded that: *"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."* This warning caused further fundamental calculations as the bearing capacity for the actual footings primarily was dictated by the frictional strength parameters of the limestone. The assessment of the bearing capacity therefore implied a reassessment of the applicability of the three methods for a case with frictional material and strongly eccentric loading.

The theory of plasticity was used to calculate the bearing capacity of the failure mechanism for the foundations. Further, this theory was used to evaluate whether the peak or the residual strength parameter should be used to calculate the bearing capacity of the footing. It was concluded that the peak strength should be used and that the friction angles should be corrected for the influence of dilatancy, Hansen (1996).

The Limit Equilibrium Analysis was used with the same purpose. Two limit equilibrium computer programs were used. The first program, BEAST, was 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, Clausen (1997).

The following calculation procedure was used for the Finite Element Analysis using the ABAQUS package:

– Establish a conceptual model for the limestone based on triaxial laboratory test results.
– Calibrate and validate the constitutive model using large scale shear tests.
– Apply the constitutive model to the ship collision problem.

A large number of test results (Bergdahl & Steenfelt, 1994) were used to calibrate a constitutive model for the limestone. Even though the limestone was essentially anisotropic due to horizontal layering and fissuring it was an acknowledged fact that the available numerical models were isotropic. Therefore an isotropic conceptual model was proposed, see Fig. 24.

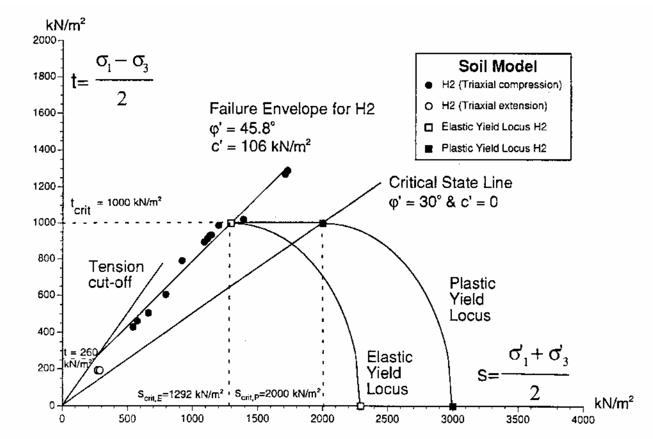


Fig. 24 Conceptual model for Copenhagen Limestone (after Bergdahl & Steenfelt, 1994)

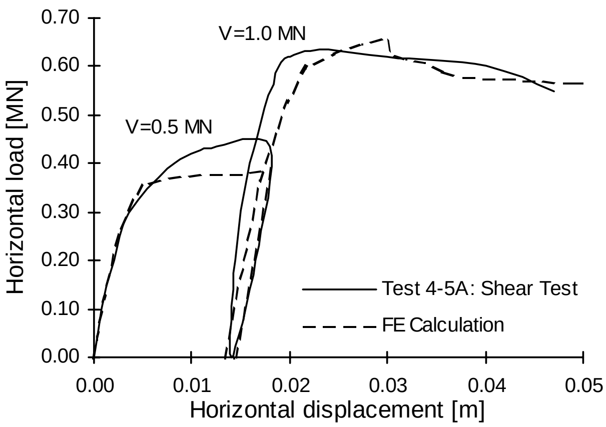


Fig. 25 Calibration result for direct shear

The most important features of the limestone behavior can be captured by a Drucker-Prager model with cap in ABAQUS, Hibbitt et al. (1996). The Drucker-Prager surface controls the frictional behavior and the cap ensures that the shear stresses do not exceed the maximum shear strength.

The material parameters were calibrated to match large scale shear tests performed in test pits at Lernacken. The calibration was carried out on several large scale shear tests, representing direct shear, passive shear and active shear. The emphasis was on the horizontal shear failure mode as the resistance towards ship impact was assumed to be governed by this mode, see Fig. 25.

The bearing and displacement capacities were verified by quasi-static push-over analysis using a 2-dimensional elasto-plastic finite element model, Hauge et al (1998) and Hededal & Sørensen (1999). The results from the model were used to define line springs and dashpot dampers that represented soil-structure interaction in a dynamic analysis with a global model containing the entire high bridge. An iterative process was carried out in order to obtain consistency between the local and the global model. The failure pattern for the west pylon is shown in Fig. 26.

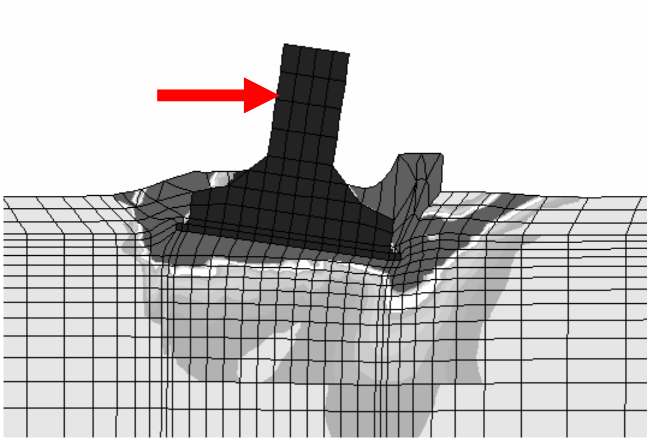

Fig. 26 Plastic zones developed due to ship impact

The three methods confirmed that it was possible to utilize very high foundation pressures (3-5 MPa) in the Accidental Limit State.

A simple calculation model for the bearing capacity was established based on the results from the three methods. The starting point for this model was the peak capacities of the passive earth pressure (rough wall) and the reaction from the foundation base. The passive pressure was then reduced by 60%. The reason for this large reduction was partly the large difference between the peak and the residual strength of the limestone and that the passive earth pressure was developed later than the maximum reaction force between the base and the limestone.

### 4.1.3 Serviceability Limit State

To limit the risk of yielding and collapse behaviour it was a Serviceability Limit State requirement that the stresses remained within the stress space defined in Fig. 27.

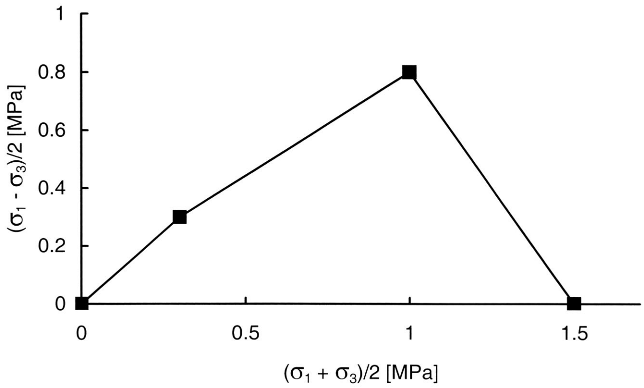


Fig. 27 Elastic stress space

The Elastic Stress Space defines the stress combinations for which no excessive plastic deformation will occur. All serviceability loads had to fulfil this condition.
The stresses in the limestone immediately below the caisson were calculated using a linear elastic 3D finite element model of the caisson and the subsoil. The model comprised a detailed shell model for the caisson cells in order to obtain the correct stiffness distribution over the bottom slab. The shell model was connected to 3D solid elements, which modelled the foundation soil to a depth of about 100 m below the bottom slab.
The condition was satisfied for all interior points under the bottom slab for all load combinations. However, the condition was violated along the edges where the elastic finite element model had singularities. The extent of the zone affected by the singularities was negligible and the violations could therefore be accepted.

### 4.1.4 Lessons learned from the Øresund Link

The experience from the interplay between widely varying disciplines from the Storebælt Link project was brought to bear in the completion of the Øresund Link project. Part of the success in transfer of knowledge was due to the fact that a number of key persons were involved in both projects.

By integration of extensive ground investigations - drawing heavily on geological and geophysical methods, and dedicated physical modelling and testing at different scales - it was possible to develop numerical tools for optimisation and verification of the governing load case, ship impact on pylons and bridge piers.
The Øresund Link was truly remarkable in three respects: The project was on time and on budget and the claims were few and far between! This was achieved by a system close to partnering with a reward system encouraging the different contractors and consultants to work in a cooperative fashion for the good of the project. The Owner, furthermore, took real ownership of the ground conditions (baseline conditions) which was instrumental in reducing the claims possibilities.

## 5. Bangabandhu Bridge

### 5.1 The Jamuna River

One of the poorest countries in the world, Bangladesh, is divided by one of the biggest rivers in the world, Jamuna River, cf. Fig. 28.


Fig. 28: Jamuna River and bridge location

The river is characterised by being unpredictable in its flow, inundating every year during the monsoon season enormous areas of land with its masses of water, essentially the run-off of the Himalaya Mountains.
Many attempts have been made over the years to build fixed links across this river to replace unpredictable and dangerous ferry traffic, which over the years has caused tremendous losses of life due to overloading of the ferries by the local population.
Because of big differences of water level between monsoon season and dry season it is almost impossible to build permanent berths and harbour facilities for ferries. Access roads to such ferry terminals are likewise very difficult to build because of the changing river bed.
As if this would not be an adequate technical challenge in itself, foundation conditions for the bridge on fixed foundations present enormous technical challenges for the following reasons:

- the river bed is highly unstable due to the changing river bed with huge material transport with variations between sand dunes at level above water to local scour up to 50 m in depth.
- the sand is transported by the river flow and displaced downstream forming sand dunes, and further,
- liquefaction is imminent during possible earthquakes, making the sand-water suspension behaving like heavy fluid.

The reason for this foundation condition is the fact that the river bed consists of incompactable uniform grain-size fine sand, washed down from the Himalayas and deposited as loose sedimentary deposits through millions of years. No bedrock is existent at reasonable depth for foundation purposes.

## 5.2 Background for Development of the Bridge Project

In the interest of helping Bangladesh develop among others through improved east-west communication between the two halves of the Bangladeshi population, the World Bank looked into alternatives for linking the country by a bridge solution, either a floating bridge or a bridge on fixed foundations. The Japanese consultant JICA had in the 1970's concluded that such a bridge could be built as a series of long-span suspension spans founded on traditional huge 20-40 m diameter open dredge concrete caissons down to 80 m foundation depth. The cost of each of these foundations would be so high that only very long spans of superstructure would be feasible, thereby leading to either cable-stayed or suspension spans. Thus the cost of the bridge would exceed in terms of 1970's price level more than 1 billion USD, which in consideration of Bangladesh economy could not be justified, even macro-economically, by the benefits achieved.
The project was therefore not considered to provide benefit-cost ratio to warrant further consideration.
However, in the early 1980's the World Bank reopened the issue and appointed a World Bank panel of experts to review the matter and investigate whether technology would be available for alternative solutions to the previous attempts and projects.
In the late 1970's and 1980's offshore development of hydro-carbons, oil and gas had necessitated development of new technologies for deep sea foundations of steel jacket platforms. This was applicable for the North Sea, the Gulf of Mexico, Australia and many other places based on use of large diameter steel piles driven to great depths, often through soft soils.
Technologies for driving of piles with diameter up to 3 m and 70mm wall thickness having load carrying capability more than 10,000 t were developed to a high degree of sophistication and reliability.
These technologies opened up new avenues for bridge foundations. As it appeared that total capacity within the industry was not fully utilised in the 1980's, it became obvious for the panel of experts to look into whether such technologies could be applied for a fixed bridge across Jamuna River.
It was found that simple tripod structures consisting of large diameter steel piles driven to 80 m of depth could lead to considerably lower foundation costs on a per unit basis, which in turn would reduce the optimum span considerably to the order of magnitude 100 m per span. In the subsequent studies it was found that these new technologies applied to bridge building in large scale for the first time would lead to overall costs of the fixed link which would be more than 50% lower than the solutions considered previously had indicated. A study of the world market for the required pile driving equipment indicated that this was readily available because of a recession in the offshore/oil exploration market in the early 1980's.

## 5.3 The Realised Project and Foundation Solution

The Banabandhu Jamuna Multipurpose Bridge is 4.8 km long with 49 spans of approximately 100 m designed to carry a 4-lane roadway, a metre gauge railway, a 230 kV electrical power transmission line and a 760 mm diameter gas pipeline.
The overall construction concept of the bridge was based on repetitive utilisation of large identical elements for piles, pile caps and superstructures leading to efficient mass production and erection on site.
As such, the haunched concrete box girders of the superstructure were constructed by the balanced cantilever method using pre-cast segments supported on piers with groups of a few raking tubular steel piles, approximately 80 metres in length, driven into the river bed.
Out of 50 piers, 21 piers have groups of 3 piles, each 2.5m diameter, and 29 piers have groups of 2 piles only, each 3.15 m diameter, cf. Fig. 29 and Fig. 30. Pile wall thicknesses vary between 40mm and 60mm. The pile caps in the form of truncated cones about 5 metres high were cast in-situ within pre-cast concrete shells (see Fig. 31).

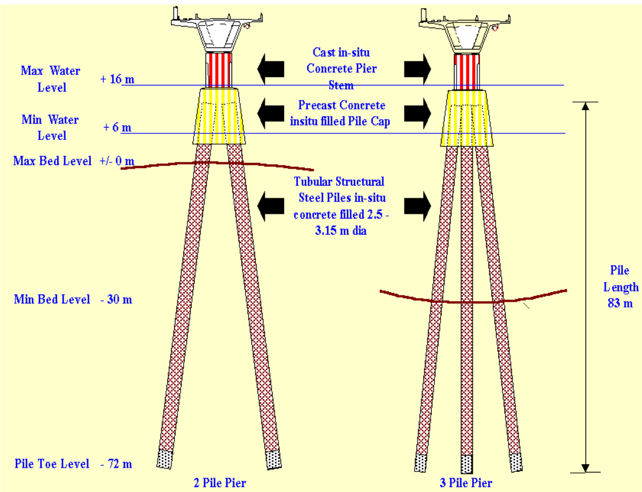


Fig. 29: Layout of pile foundations

After driving to final level the piles were cleaned out down to two diameters above toe level using an airlift followed by pouring infill tremie concrete. In order to reinstate the stiffness of the soil/concrete interface grout was pumped to the pile toe under 40 bar pressure after the tremie concrete had hardened sufficiently.

Fig. 30: Driving a 3.15 m diameter pile

This grouting was seen as a way to keep absolute and differential settlements to a minimum.
The casting of the pile caps and the completed bridge is seen in Fig. 31.
For a more comprehensive description of the construction methodology see Barr et al (1999).
The main advantages of the large diameter raking tubular steel pile foundations can be summarised as follows:

- low cost taking advantage that the offshore construction industry was somewhat depressed at the time of the bridge construction.
- improved lateral stiffness of the raking pile groups as compared to vertical foundation elements like open dredge wells
- very transparent design advantageous in relation to violent river flow and liquefaction under seismic activity.
- reliable and fast construction. All 121 permanent piles were successfully driven between October 1995 and June 1996 with two months to spare before shallow water would have stranded the barge until the following flood season

Fig. 31: Casting of pile caps and completed bridge

## 6. Strait of Gibraltar Crossing

Construction of the Ekofisk oil storage caisson in the North Sea in 1973 initiated a new era within the development of deep sea concrete foundations. Since the Ekofisk caisson standing in 70m water depth more than 20 major concrete platforms with a total of approximately 2.3 million $m^3$ have been installed in the North Sea. The era of the huge fixed concrete platforms ended in 1995 with the largest of them all, the Troll East platform, which was placed in no less than 303m water depth, cf. Fig. 32.

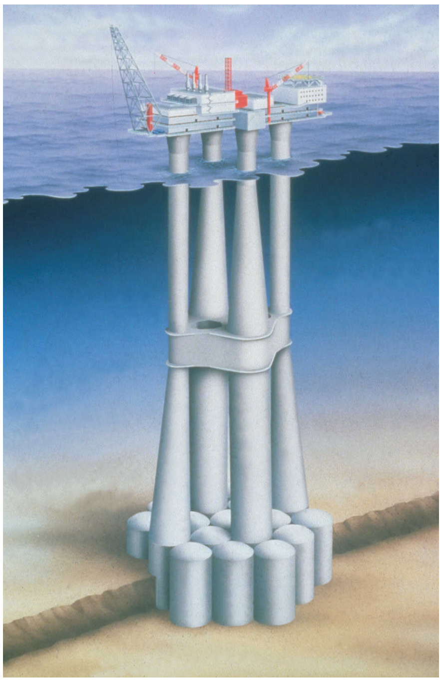

Fig. 32: Troll East platform (after www.ngi.com)

Almost all these concrete platforms rest directly on the seabed without any previous preparation or levelling. The contact is secured only by means of small skirts penetrating the upper soil layer and subsequent under-base grouting within the base area confined by these skirts.
However, for the Troll East platform a new technology was introduced to overcome soft soil conditions. This technology, the so-called skirt piling concept is based on penetrating up to 30m long skirts into the soft soils to ensure the foundation stability instead of traditional piling which would be almost unconceivable for the actual environmental conditions.

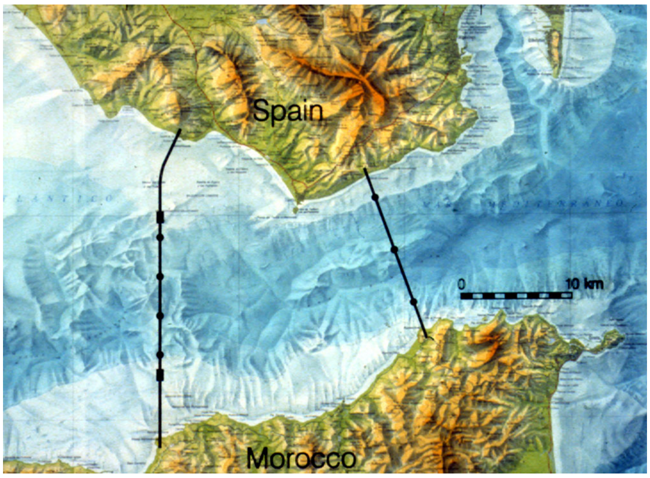


Fig. 33: Gibraltar Strait bridge alignments

The skirt penetration is based on a suction technique where a mechanical system is used to achieve a controlled under pressure below the platform base.
The technology developed within the offshore oil industry allowing concrete platforms to be installed at water depths of more than 300m has made it possible realistically to build a bridge on fixed foundations across the Strait of Gibraltar. The Strait is characterised by large water depths, at the narrowest crossing between 500-1000m, and 250.300m at a longer alignment along a subsea sill, cf. Fig. 33.

Fig. 34: Artist's view of a Gibraltar Bridge

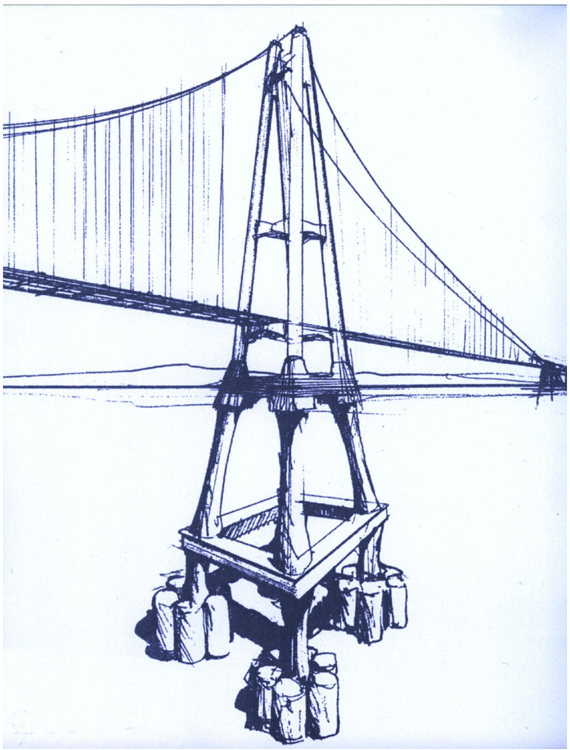

Fig. 35: Artist's view of Gibraltar pier concept

Costs of fixed foundations at such water depths would, of course, be very high and the optimum spans therefore very long. 10 years of studies of alternative foundation and superstructures have resulted in the development of a technically feasible concept of a series of 3,500m suspension spans with 450m tall A-frame towers founded on a gravity based structure (GBS) built and installed according to the technologies proven for construction of the concrete platforms in the North Sea, cf. Fig. 34 and Fig. 35.
The proposed pier concept is characterised by being physically transparent and light in its nature with minimum material consumption. This allows relatively free water flow minimising wave and current forces as well as inertia forces from vibrating masses generated by strong earthquakes.
The foundation pads, cf. Fig. 35, are cellular structures composed entirely of either thin walled cylindrical or spherical shells required to resist the very large differential water pressures occurring during the installation of the piers.
The construction sequence of the piers is as follows:

- casting of each of the four foundation pads individually in a dry dock
- towing of the pads to a temporary, shallow water site for adding the lower frame
- towing of the assembly to a deep water construction site where the four conical legs will be slip formed, and finally connected by the upper frame
- towing of the completed pier to its permanent location in the bridge alignment, sinking to the seabed and under base grouting

For a more detailed description of the proposed deep water foundations for the Gibraltar Fixed Link see Ostenfeld and Pedersen (1993).

## 7. The Chacao Channel Bridge

Investigations and planning for a new major bridge across the Chacao Channel (see Fig. 36) connecting the island Chiloé to the mainland in the southern part of Chile is going on at present.


Fig. 36: Location map of the Chacao Bridge

The bridge will be among the largest in the world comprising two suspension bridge spans of about 1,100m and 1,300m joining at a central A-framed tower on a tiny shoal, Roca Remolinos, in the middle of the channel. This would be the first suspension bridge in the world with continuous cables over two main spans, cf. Fig. 37.

Fig. 37: Artist's view of Chacao Channel Bridge

Fig. 38: Geotechnical drilling barge moored over Roca Remolinos in 5 m/s tidal current

The two main challenges in connection with the development of foundation solutions for this bridge are extreme seismic activity and very strong tidal currents in the channel up to 9 knots (5 m/s) as evidenced by Fig. 38.
In fact Roca Remolinos was originally a small island, but became submerged as the landscape sank several meters during the biggest earthquake ever recorded, the Valdivia earthquake of May 22. 1960, which surpassed 8.5 on the Richter scale.
Due to the high seismicity and the soft rock foundation strata it was decided to use 3.0 m diameter steel tubular piles with reinforced concrete infill in the entire pile length as seen in Fig. 39.
A major advantage of this solution is that the mass of the footing block is significantly reduced from that of a conventional pier. The vertical large diameter piles respond well to seismic excitation from all directions. And they still exhibit sufficient lateral stiffness under serviceability environmental loadings. However, the design of the footing block requires great care in order to transfer the high shears and moments.
The top part of the steel pile needs proper embedment in the footing block, increased wall thickness and interlocking to the reinforced infill concrete by shear rings for enhanced stiffness and ductility. A special feature, as seen in Fig. 40, is provision of slots in the upper part of the piles allowing horizontal reinforcement layers in the pile cap to pass through the piles.
The design of the footing blocks also requires particular consideration to the practicality of construction due to the special circumstances with very high tidal currents.

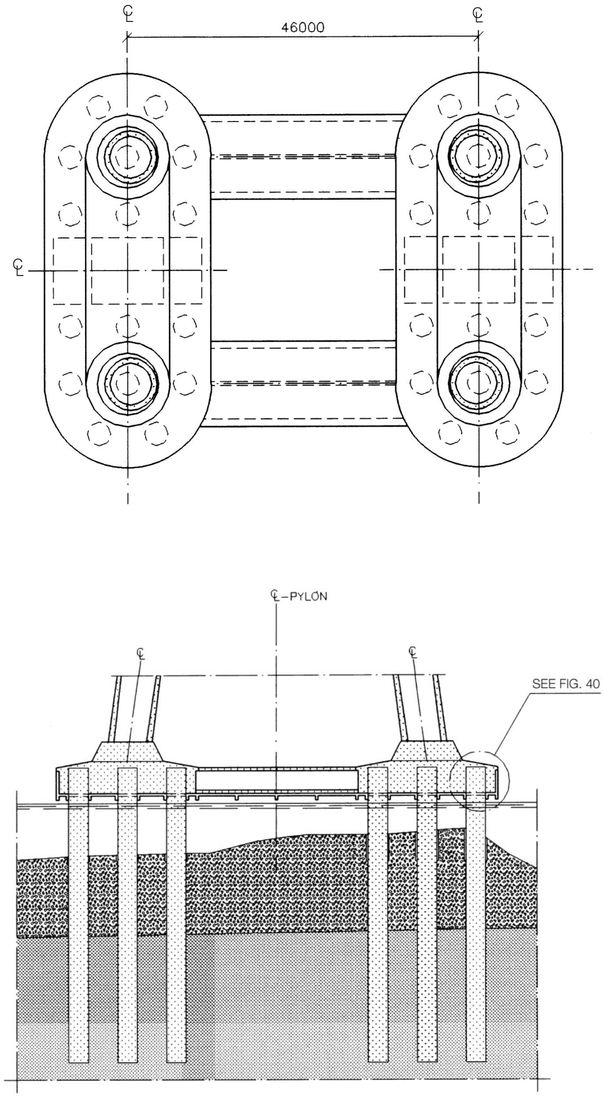


Fig. 39: Large diameter steel pile foundation

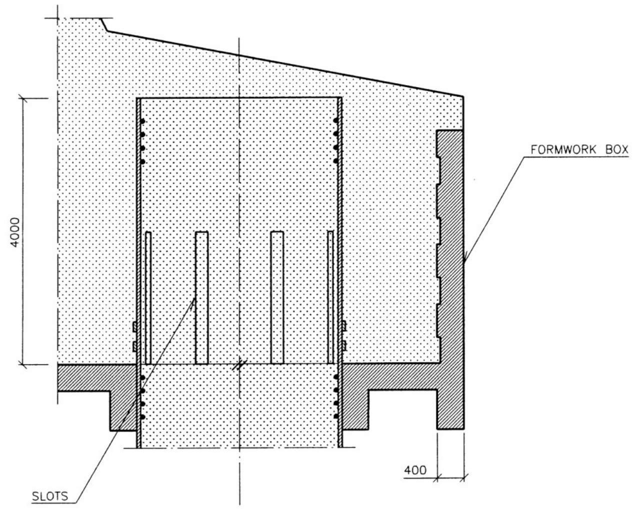


Fig. 40: Detail of upper part of steel pile with pre-cast formwork box

A pre-cast thin walled concrete formwork box, Fig. 40, is lowered over the piles through oversized holes in the bottom slab of the box to allow a positioning tolerance of the piles of ±150 mm. After dewatering of the box the structural concrete of the footing block can be cast in dry conditions. It is necessary to establish watertight seals between the steel piles and the oversized holes, e.g. by means of inflatable rubber tubes.
The formwork box left in place has the additional advantage of providing a perfect protection of the main load carrying footing block from ingress of chlorides etc. in the otherwise very exposed splash zone.

## 8. Conclusion

Society in general and the geotechnical profession in particular owe a great dept of gratitude to Terzaghi for his transcendent ideas which were instrumental in transforming soil mechanics and geotechnical engineering from a handicraft to a discipline of science and technology.
Today we all take the effective stress principle for granted and forget that this was a "leap" in thought for mankind.
Terzaghi was very aware of the role and importance of neighbouring disciplines, not least engineering geology, and it is difficult to imagine how large projects, as exemplified by the Kuwait City Subiyah Causeway project in Fig 41, can be feasibly accomplished without taking heed of this interrelationship.
The latter has been an integrated part of the Danish geotechnical tradition for decades and with successful cross pollination of geotechnical and structural engineering this has enabled involvement in, and successful completion of a number of challenging major bridges and fixed link project world wide.

Fig. 41: Artist's rendering of a possible solution for the Kuwait City Subiyah Causeway

In the paper some of these projects have been described and hopefully the necessity of and the rewards by cross pollination of science and industry and the positive involvement of incommensurable factors as environment, political climate etc. have become apparent.
New technologies appear all the time and nanotechnology, chaos theory etc. combined with ever increasing demands for global sustainability will challenge the engineers of the future immensely.
We should consider ourselves lucky to be able to contribute to this process. If we have an open mind our engineering disciplines may gain the respect they deserve in the public eye and we may become the servants of Society in pursuing this purpose in the great tradition of Terzaghi.

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