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FIP State of Art Report:

Foundations of concrete gravity structures in the North Sea

Fédération Internationale de la Précontrainte

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FIP STATE OF ART REPORT: FOUNDATIONS OF CONCRETE GRAVITY STRUCTURES IN THE NORTH SEA.

FOREWORD

This State of the Art report details with the experience from the foundations of the concrete gravity oil or gas platforms which have recently been installed in the North Sea. The report was prepared by a Working Group formed in November 1976 by the FIP Commission on Concrete Sea Structures. Members of the group represent a cross-section of the foundation engineering interests which have been involved in this recent experience.

By a number of working sessions and the preparation of original papers the Working Group has distilled the state of the art into four main themes:

Geotechnical investigations

Design procedures

Installation methods

Operational experience.

The benefit of this work has been considerable for those involved in its preparation and FIP has published the report for general information to other engineers who may be interested in this experience.

Since the installation of the Ekofisk concrete oil storage tank by Phillips Petroleum in 1973, there has been a rapid development of design and construction technology in applying concrete structures to other locations in the North Sea. In dealing with these offshore locations there have been many severe problems to resolve, chiefly arising from the severity of the wind and waves in this area. For the foundation engineers this has particular problems due to the innovative concepts of design and their location on deep water unprepared seabed sites for which there was limited experience of soils response to this type of structure.

The soil conditions at the sites, consisting of stiff clays and dense sands, are being observed to be capable of supporting the loads introduced by concrete gravity platforms. It is satisfactory to be able to report that the foundations of all the concrete structures are performing without difficulty.

The highlights of the conclusions contained in this report relate to the following points:

The quality of information obtained from geotechnical investigation techniques under severe offshore conditions has improved with the development of new equipment.

Many of the theoretical analyses of the geotechnical design problems related to the foundation behaviour have been confirmed by observed experience.

The reliability of installation procedures has been demonstrated, and careful monitoring has obtained further data which has improved production methods.

The interaction between the foundations and the installation of operational platform equipment for conductor and well drilling has been observed, with methods devised to attempt to minimise the disturbance to seabed foundation soils.

The report has been divided into sections and each section has been written by individual members together with the collaboration of colleagues in their organizations. The complete report has been reviewed by all members of the Working Group who have endorsed it in the form to be published. The Chairman wishes to acknowledge the considerable voluntary effort made by all concerned. Their co-operation in dealing with the preparation of original work for this report has led to a significant advance in published information being made available to others from the unique experience they have gained in the North Sea.

Tom Ridley
Chairman, Working Group on Foundations

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1 FOUNDATIONS OF CONCRETE GRAVITY STRUCTURES IN THE NORTH SEA

1.1 Introduction

Thirteen large concrete structures have been installed since 1973 on the seabed in the North Sea. The experience gained from the site investigations and design of the foundations for these structures can now be reviewed, together with the installation and initial performance over the last four years. These structures have the function of providing offshore platforms from which the various operations of drilling, production, storing and transportation of oil or gas can safely take place.

All of the structures being used have adopted the same type of foundation concept. This consists of the elimination of uplift forces by the provision of structural selfweight and ballast using large caisson-type foundations. This weight provides continuous contact with the seabed and resists the very large forces generated by the wind and waves in offshore water depths varying from 70 to 153 metres.

The foundation loading is transferred from the structure at the seabed interface, and this takes the form of vertical load, bending moments and shear forces. High local forces can also arise during installation, due to local highspots on the seabed. These can combine with the temporary differential water pressures on the platform walls and base which exist during immersion of the structures to cause critical soil and structure foundation loading. Projecting skirts have usually been provided below the foundation structure to increase the efficiency of transfer of forces to the underlying seabed soils. These also prevent undermining of the foundation due to scour action and are convenient compartments for sub-base grouting.

The very large forces thus generated in the seabed soils have posed formidable problems for the foundation engineer, requiring the rapid development of new knowledge and techniques to provide judgement of safe foundation conditions. This development has been associated with the design and installation of each of the different structures. The operating companies are also now beginning to judge the performance characteristics from the structures in service in the North Sea.

1.2 Concrete gravity structures in the North Sea

The thirteen concrete structures which are being used in the North Sea at this time are identified in Table 1.1, and are given in historic sequence of their installation. A brief description of each structure will assist an understanding of the general points which are made in the rest of the State-of-the-Art report on their foundation design principles. Their locations are indicated on the North Sea map as shown in Figure 1.1.

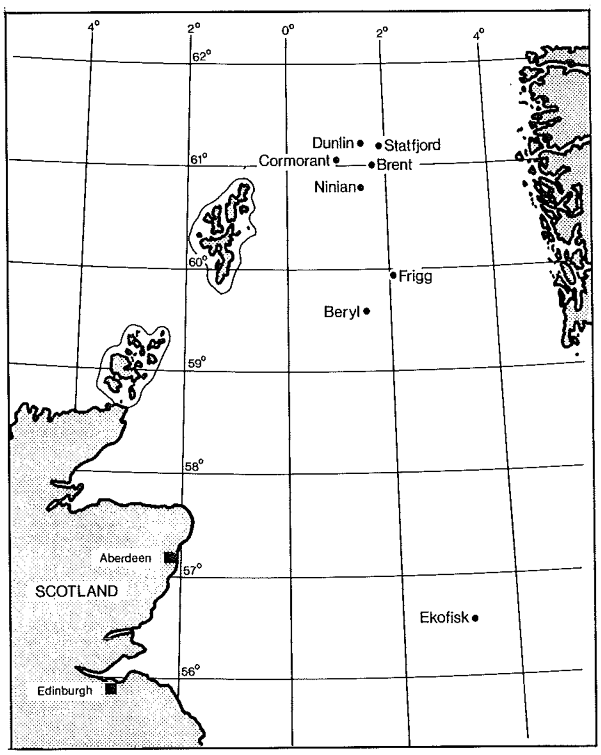


Figure 1.1: Location of concrete structures in the North Sea (1978).

Table 1.1: Concrete structures in the North Sea.

				Site data		Base		
Type	**Name**	**Operator**	**Year**	**Water depth m**	**Foundation soil**	**Slab**	**Skirts**	**Dowels**
Doris	Ekofisk	Phillips	1973	70	Dense fine sand	Flat A = 7400 m^2	0.4 m concrete ribs	None
Condeep	Beryl A	Mobil	1975	120	Dense fine sand over clay	Conical domes A = 6200 m^2	3.0 m steel 0.5 m concrete	3
Condeep	Brent B	Shell	1975	140	Stiff clay with sand layers	Conical domes A = 6200 m^2	4.0 m steel 0.5 m concrete	3
Doris	Frigg CDP1	Elf	1976	98	Dense fine sand	Flat, ring-shaped A = 5600 m^2	None	None
Sea Tank	Frigg TP1	Elf	1976	104	Dense fine sand over clay	Flat A = 5600 m^2	2.0 m concrete	None
Doris	Frigg MCPO1	Total	1976	94	Dense fine sand	Flat ring-shaped A = 5600 m^2	None	None
Condeep	Brent D	Shell	1986	140	Stiff clay with sand layers	Conical domes A = 6300 m^2	4.5 m steel 0.5 m concrete	3
Condeep	Statfjord A	Mobil	1977	145	Stiff clay	Conical domes A = 7800 m^2	3.0 m steel 0.5 m concrete	3
Andoc	Dunlin	Shell	1977	153	Stiff clay with sand layers	Flat A = 10 600 m^2	4.0 steel	4
Condeep	Frigg TCP2	Elf	1977	102	Dense fine sand over clay	Conical domes A = 9300 m^2	1.2 m steel 0.5 m concrete	3
Doris	Ninian	Chevron	1978	136	Stiff clay with sand layers	Flat A = 15 400 m^2	3.5 m steel	None
Sea Tank	Brent C	Shell	1978	140	Stiff clay with sand layers	Flat A = 10 300 m^2	3.0 concrete	None
Sea Tank	Cormorant	Shell	1978	150	Stiff clay with sand layers	Flat A = 9700 m^2	3.0 m concrete	None

1.2.1 **Ekofisk Tank (Figure 1.2)** [1-3]

This 1 million barrel oil storage tank was designed by C.G. Doris and constructed in Stavanger, Norway, for the Phillips Petroleum Company's Ekofisk field. The tank was towed into position in June 1973 and located in 70 m water depth. The foundation base is nearly circular in plan with a mean diameter of 95 m, and when completed this concrete structure had a total weight of 215 000 t.

1.2.2 **Beryl 'A' Platform (Figure 1.3)**[4-10]

This was the first concrete platform structure to be constructed to the Condeep design, and is used for the oil production facility on Mobil's Beryl field. It was built in Stavanger, Norway, and towed into position in July 1975 to be installed in 120 m water depth. The foundation base consists of 19 cylindrical tanks with steel skirts giving a mean overall diameter of approximately 90 m and a total platform weight of 330 000 t.

1.2.3 **Brent 'B' Platform (Figure 1.4)**[11]

This Condeep structure was also built in Stavanger, Norway, and installed in 140 m water depth during August 1975 for use by the Shell/Esso group on their Brent oilfield as a production platform. The foundation base was similar to the Beryl 'A' platform with 19 cylindrical tanks using steel skirts giving a mean overall diameter of approximately 90 m and a total platform weight of 330 000 t.

1.2.4 **Frigg CDP1 Platform (Figure 1.5)**[12, 13]

Designed by Howard-Doris and constructed in Norway by the Norwegian Contractors Group, this platform was installed in 98 m water depth in September 1975 and is used for the drilling, production and processing of gas. The platform is a prestressed concrete structure, and closely follows the design principles established with the Ekofisk tank. The foundation base is circular with a diameter of 102 m and the total platform weight (including some ballasting) is 183 000 t.

1.2.5 **Frigg TP1 Platform (Figure 1.5)**[14, 15]

Designed and built by the McAlpine–Sea Tank group at Ardyne Point in Scotland and installed in 104 m water depth in June 1976. This platform is used for the retreatment of the gas from the CDP1 platform. It consists of a caisson with 25 cells giving a foundation base of 72 m square in plan using concrete skirts and with a total platform weight of 176 000 t.

1.2.6 **Frigg Platform MCPO1 (Figure 1.6)**[16]

Designed by C.G. Doris and constructed first in a drydock and later in a fjord at Kalvic, Sweden, this barrel structure is located some 174 kilometres from the Scottish mainland in 94 m water depth. The platform was installed during June 1976 and its principal function is to boost the flow of gas down the pipeline from Frigg to the St. Fergus terminal. The circular base is 102 m in diameter and the total platform weight (including sand ballasting) is 183 000 t.

1.2.7 **Brent 'D' Platform (Figure 1.4)**[17]

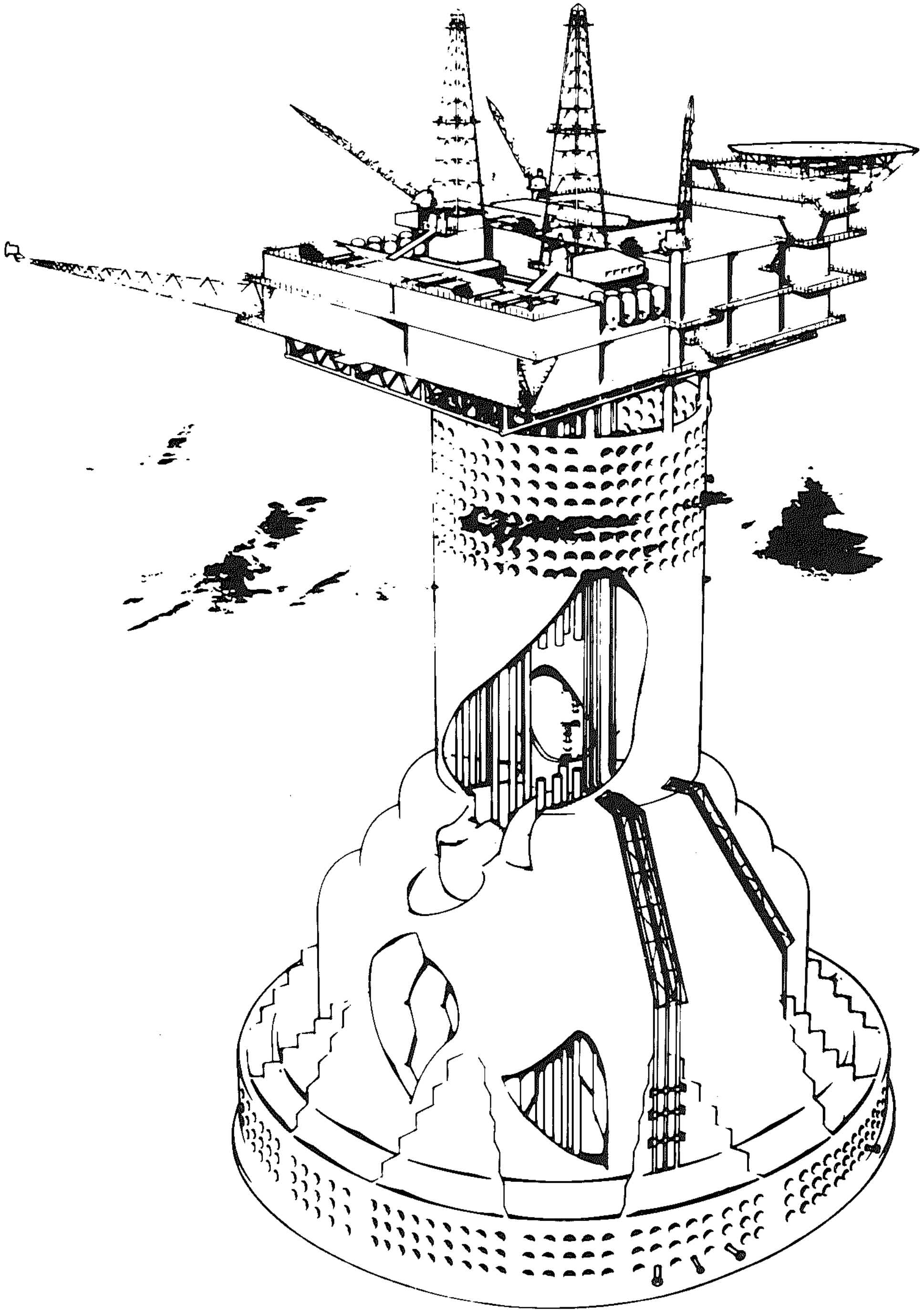
This structure is virtually the same as the Brent 'B' platform described in paragraph 1.2.3 being a Condeep design built in Stavanger, Norway. It was installed in July 1976 in 140 m water depth.

1.2.8 **Statfjord 'A' Platform (Figure 1.7)**[18, 19]

This platform is the tallest platform so far constructed and was installed in a water depth of 145 m in May 1977. It is to be used for oil production facilities by the operating company, Mobil. Based on the Condeep design the platform was built at Stavanger, Norway, by Norwegian Contractors Group. The foundation base has a diameter of 110 m and consists of 19 cylindrical tanks with a total platform weight of 350 000 t.

1.2.9 **Dunlin 'A' Platform (Figure 1.4)**[20-23]

This platform was designed and constructed by the ANDOC group, using a dry-dock near Rotterdam for the first stage with completion of the structure in a fjord near Stavanger, Norway. It was towed to location in 153 m water depth during May 1977 for use in the Shell/Expro Dunlin oilfield east of Shetland Islands. The foundation base is 100m square with steel skirts and a total platform weight of 250 000 t.



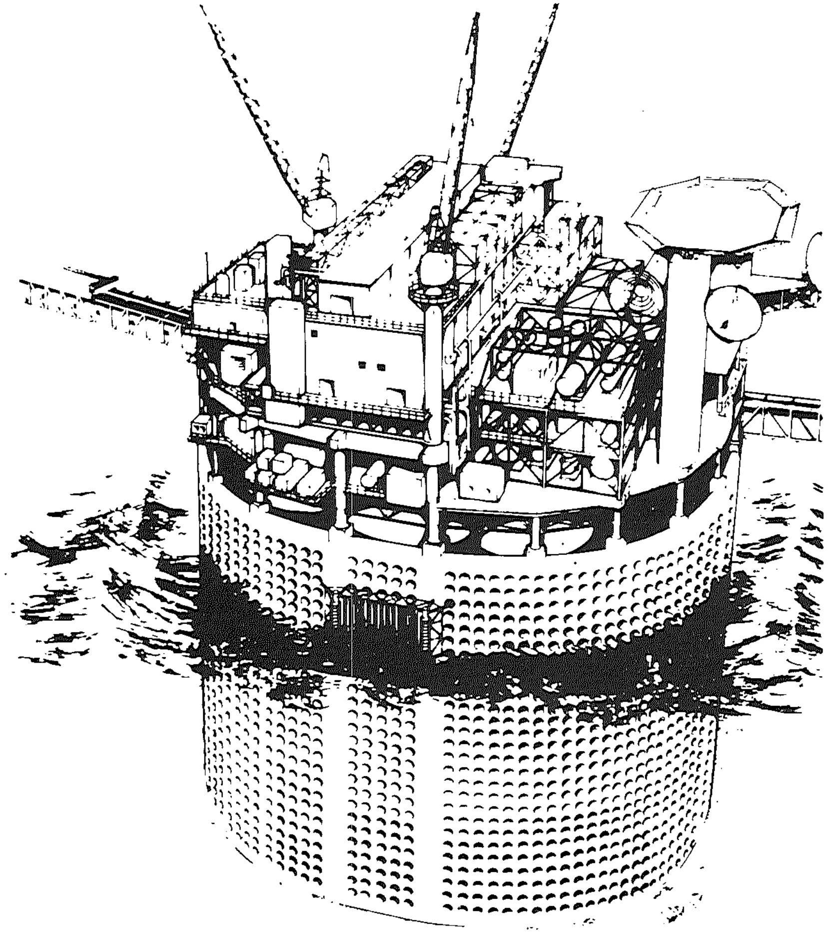


Figure 1.2: Ekofisk tank (Phillips) and Ninian platform (Chevron).

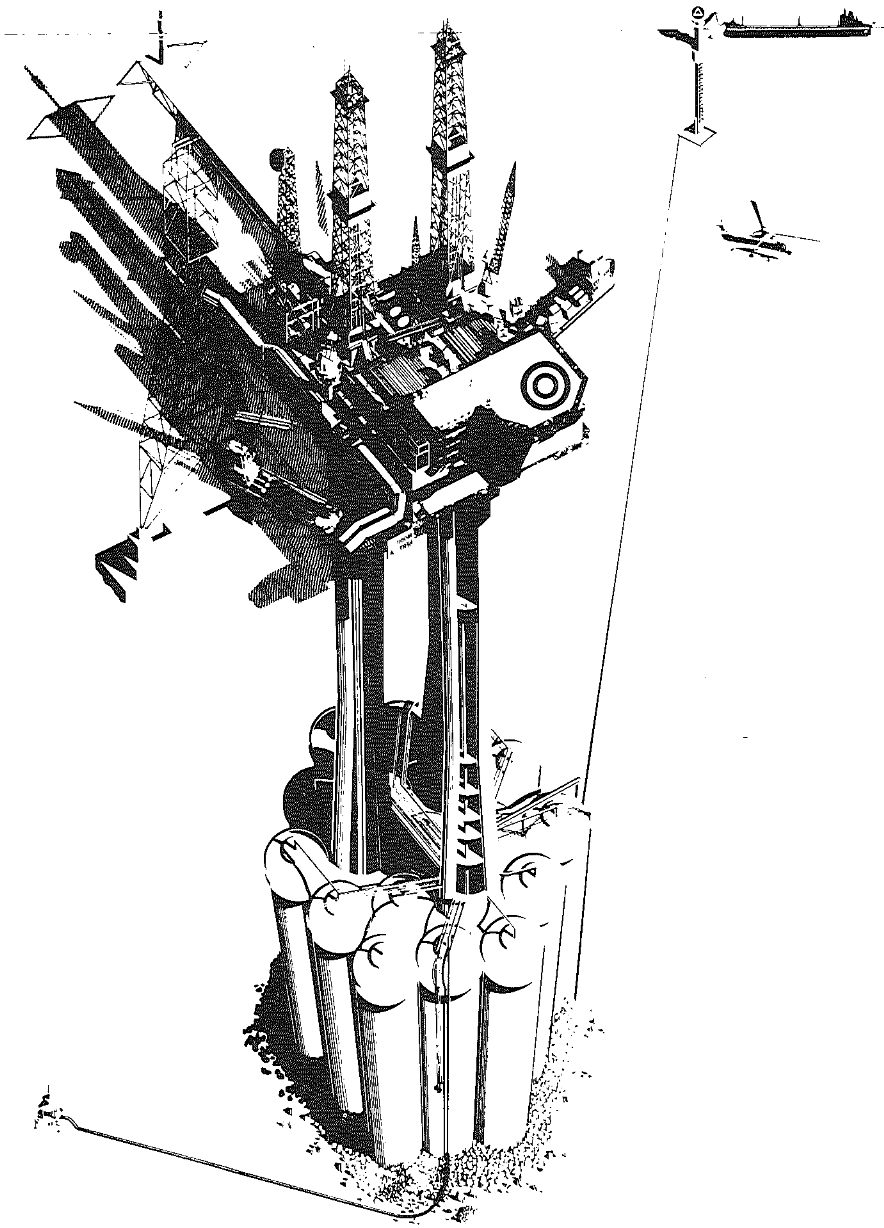


Figure 1.3: Beryl 'A' platform (Mobil).

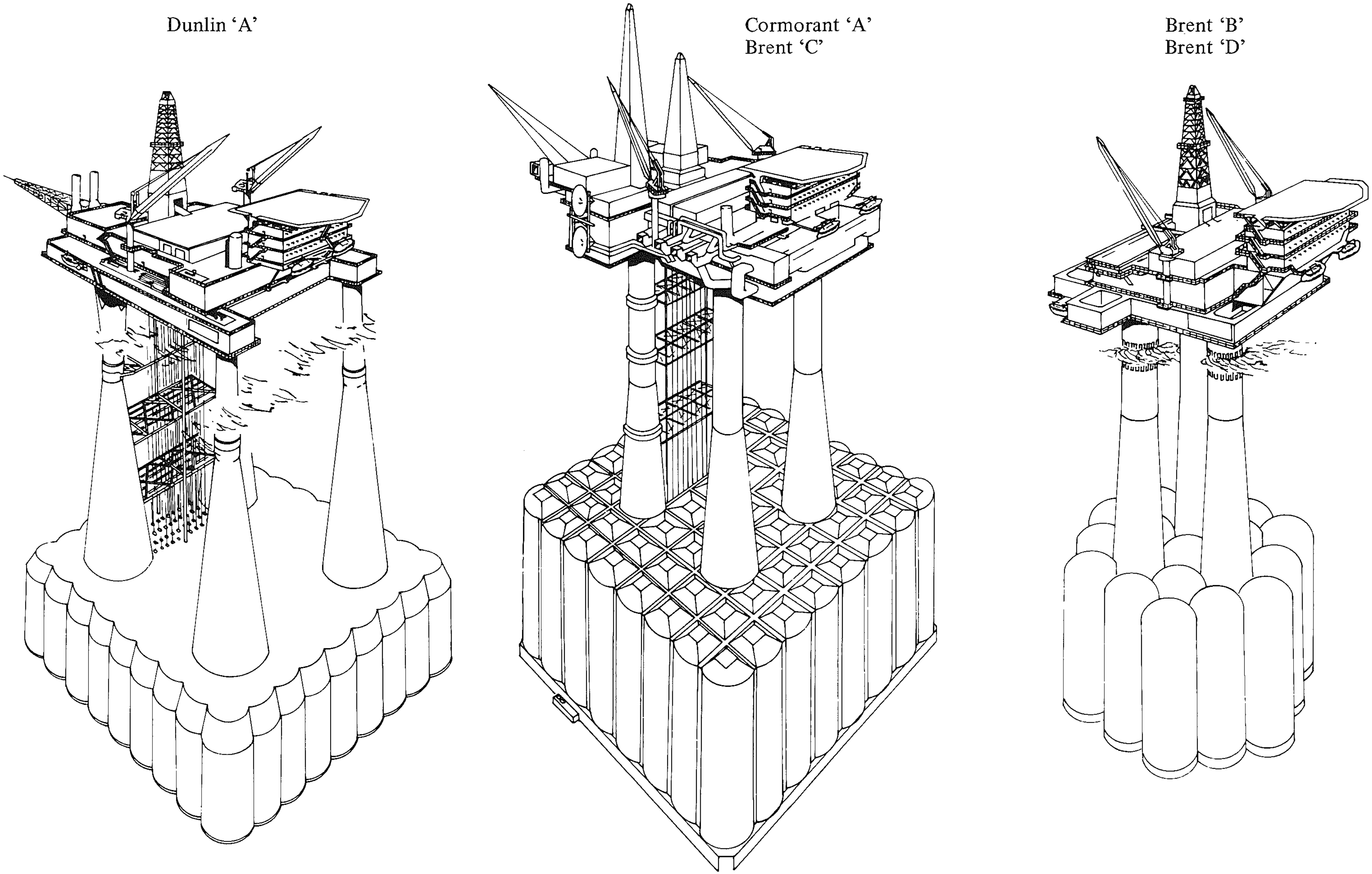


Figure 1.4: Concrete platforms used by Shell Expro.

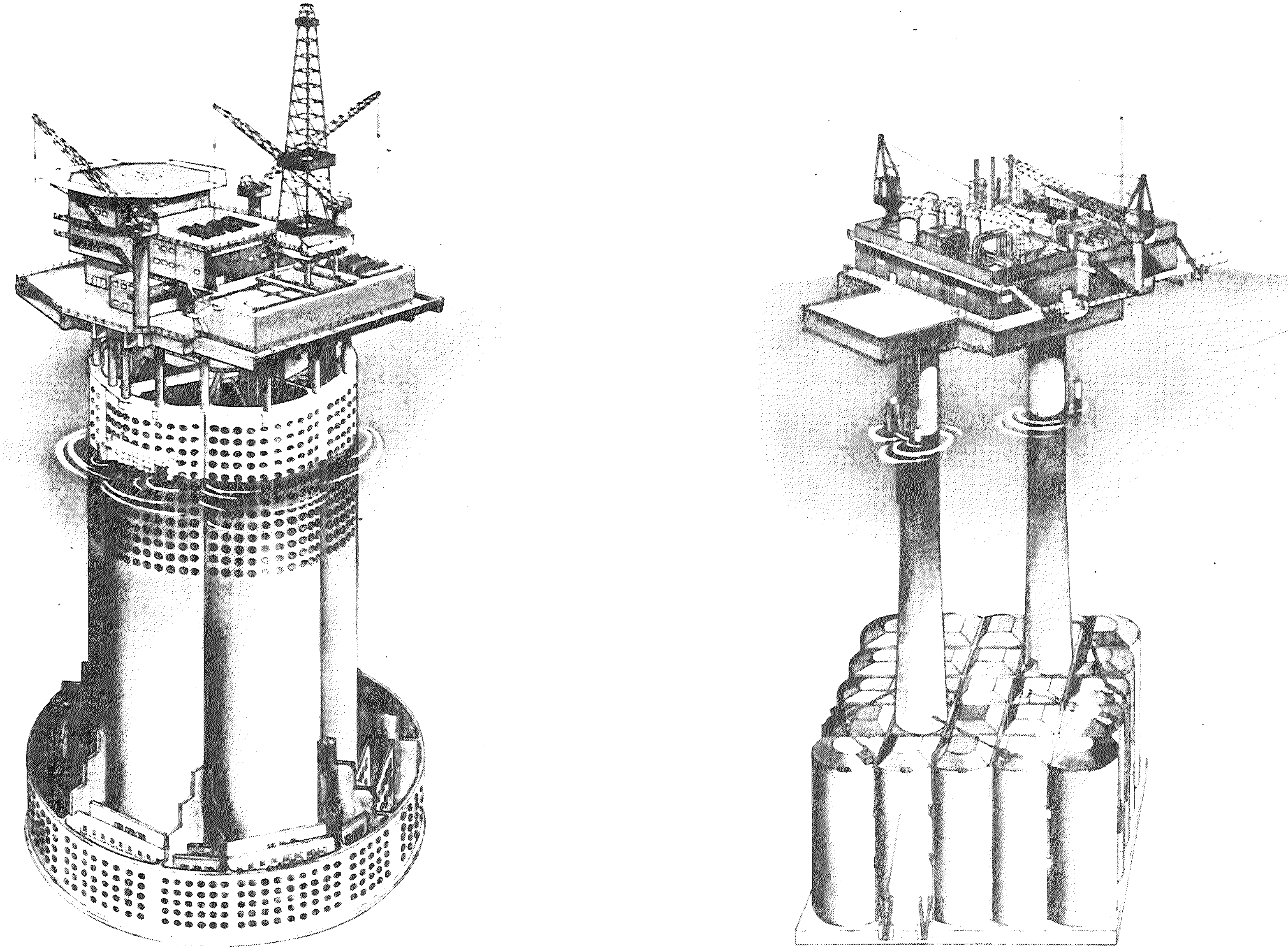


Figure 1.5: CDP-1 (Elf) and TP-1 (Elf) platforms.

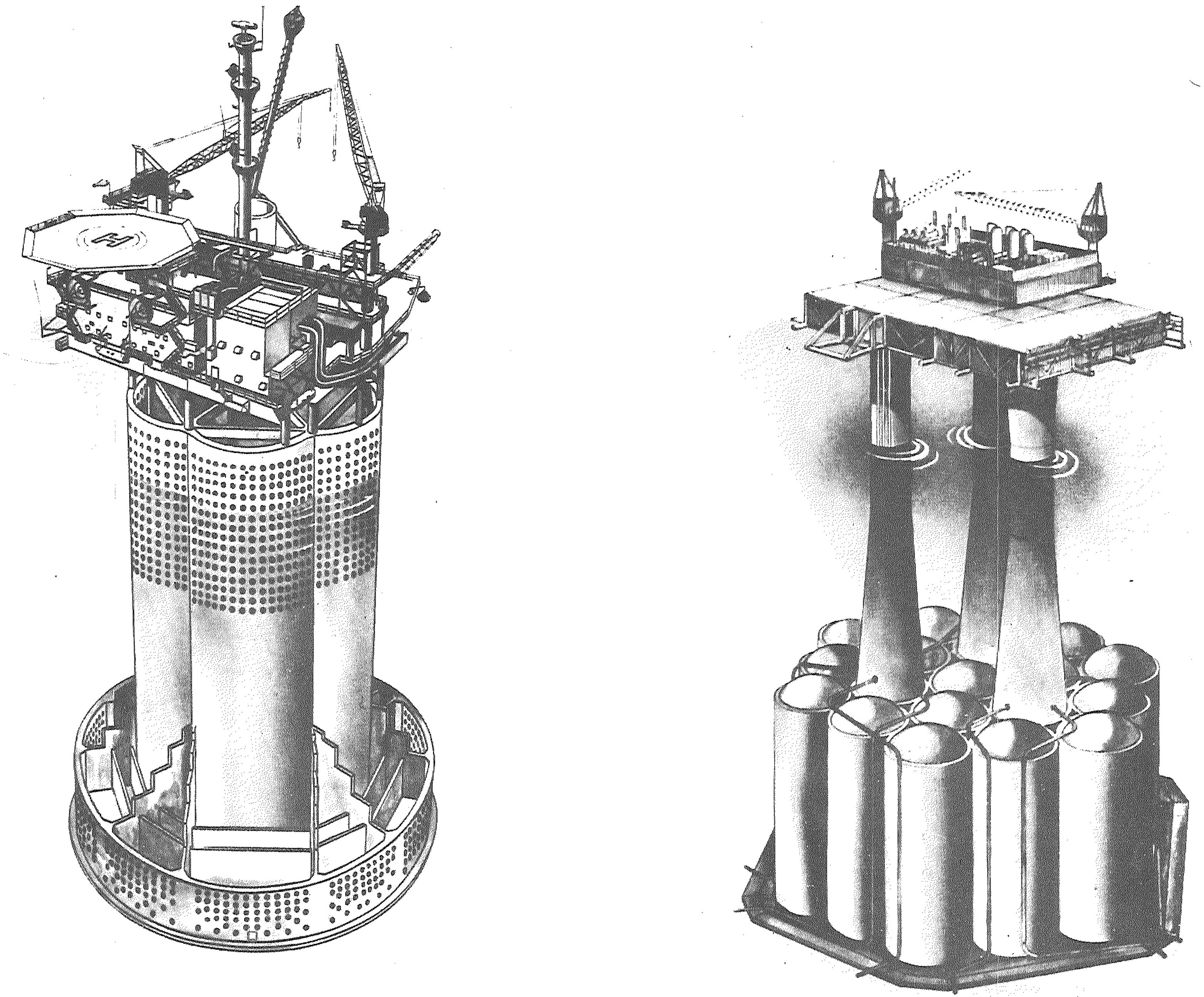


Figure 1.6: TCP2 (Elf) and MCP-01 (Total) platforms.

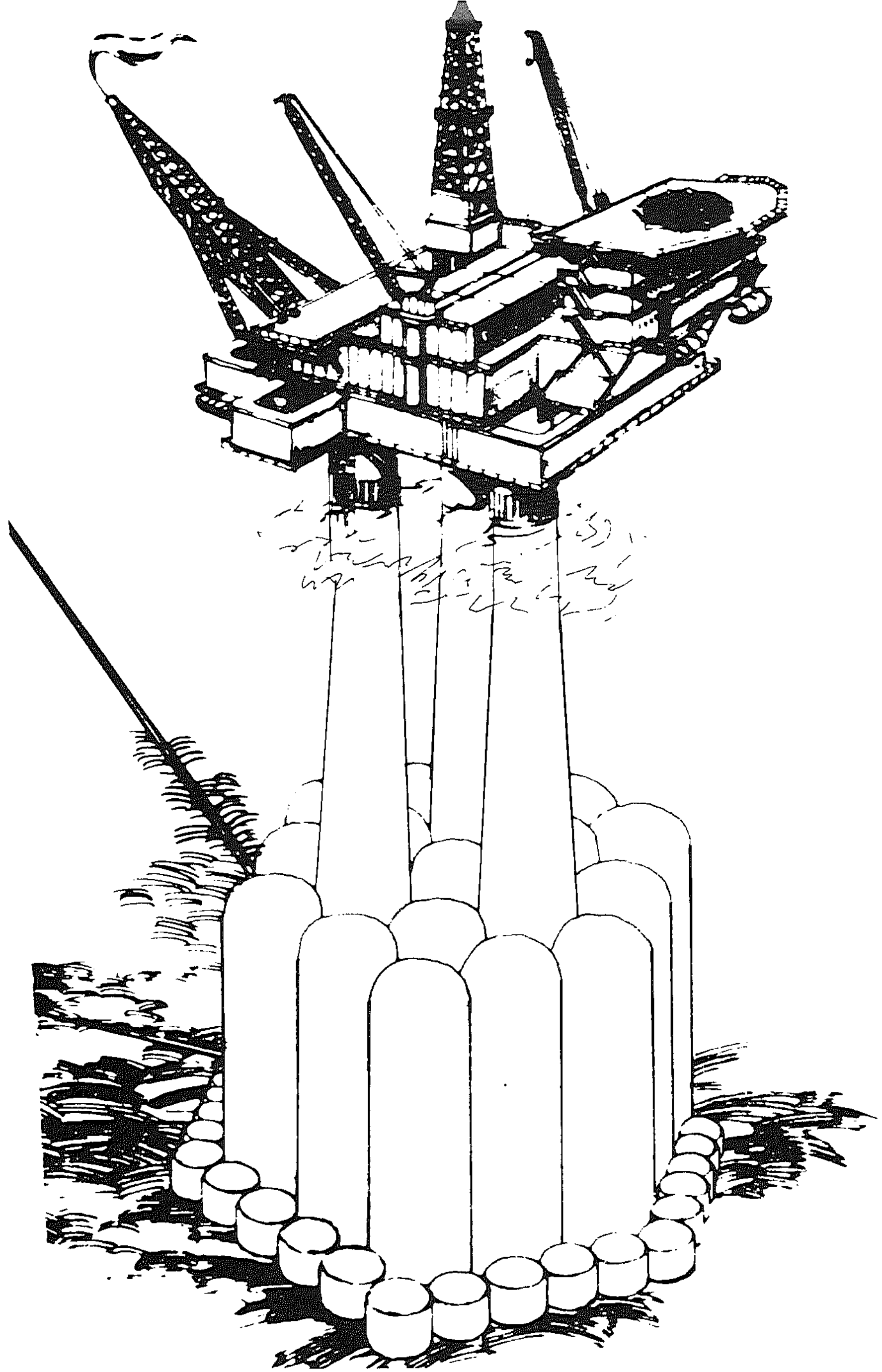


Figure 1.7: Statfjord 'A' platform (Mobil).

1.2.10 Frigg Platform TCP2 (Figure 1.6)[4]

Designed by the Condeep Group and built in Aldalsnes, Norway, by the Norwegian Contractors Group, this platform was located in 102 m water depth in June 1977. It is used to process the recompress gas from the Frigg field. The foundation consists of 19 cylindrical cells on a hexagonal base slab, with a total platform weight (including sand ballasting) of 306 000 t.

1.2.11 Ninian Platform (Figure 1.2)[24-26]

This platform was designed by the Howard-Doris group and was completed at their Loch Kishorn site in Scotland for installation in 136 m water depth during the summer of 1978. It is the largest concrete structure to be built to date for North Sea use, and will have a total platform weight (including ballasting) of 600 000 t. The platform will be used by Chevron for the oil production facilities on the Ninian oilfield east of Shetland Islands. The foundation base is circular of 140 m diameter and uses steel skirts.

1.2.12 Brent 'C' Platform (Figure 1.4)[27]

This platform was designed and constructed by the McAlpine–Sea Tank group at Ardyne Point, Scotland and is being completed near Stavanger, Norway, for installation in 140 m water depth. It will be used by the Shell/Esso group as an oil production platform for their Brent oilfield east of Shetland Islands. The foundation base is 91 m square with 64 cells, using concrete skirts and a total platform weight of 282 000 t.

1.2.13 Cormorant Platform (Figure 1.4)[27,28]

Similar to the Brent 'C' platform this structure has been designed and constructed by the McAlpine–Sea Tank Group at Ardyne Point, Scotland, and is being completed near Stavanger, Norway for installation in 150 m water depth at the Shell/Esso Cormorant oilfield east of Shetland Islands. The foundation base is 100 m square with 64 cells, using concrete skirts, and a total platform weight of 343 000 t.

1.3 Summary of the state of the art

1.3.1 Background information

There is little accumulated knowledge and experience of the top layers of seabed soils in the presently explored offshore areas of the world. The site investigations for offshore sites are a much more demanding task than onshore. The quality and amount of data which is available to the designer is considerably less for an offshore gravity structure than for a structure of similar size and importance onshore. This deficiency has to be compensated for by choosing the design parameters on the safe side, taking into account the variation in results between borings.

Another factor is the problem involved in placing a large structure, about 100 m in diameter, on an unprepared seabed site. Local variations in soil conditions and bottom topography have to be taken into account. It is also an advantage, from the operator's point of view, that the design of the structure is sufficiently adaptable to allow installation on several different sites. This arises from the possibility that the reservoir appraisal indicates that the optimal platform position is different from the one first envisaged. The requirements for location flexibility are thus very important.

The actual dimensions of the base structure are not only decided by the foundation engineer. More than for most other structures he will work hand in hand with the structural engineer and other members of the design team. Considerations influencing the base dimensions include requirements for hydrostatic stability in relation to deck load during tow-out, the need for oil storage, the structural system selected for the caisson and the admissible draft in shallow water along the tow-out route, as well as the geotechnical considerations normally applied.

This means that the foundation engineer must work very closely with the other designers during all stages of the design programme, so that there is continuity of interpretation of all factors arising from the seabed soil conditions and their behaviour during the installation and working life of the structure.

1.3.2 Foundation design problem

The basic requirement for the foundation of an offshore concrete platform is similar to that for any landbased structure, i.e., a sufficient factor of safety against ultimate failure and no intolerable settlements or movements under the applied loads. However, the offshore conditions and operational use of the platform structures means that there are special problems arising which combine with the placing of these structures on unprepared seabed sites to require new foundation procedures to be introduced. As the economics of the structure depend greatly on the design of the foundation, it is clear that unnecessary conservatism must be avoided.

In dealing with the offshore conditions the foundation engineer has to face severe limitations based on existing knowledge for the following reasons:

(1) For the extreme loading from wave forces there are much larger horizontal forces than are typical for land based structures, and the failure modes to be considered are therefore different from normal experience.

(2) The dynamic nature of the wave loading introduces the special problem of the effects of repeated loading on the geotechnical properties of the soils.

Most structures are equipped with skirts designed to penetrate a certain depth into the seabed. Unless the submerged weight of the structure is sufficient to achieve the required penetration depth, the safety of the foundations may be less than required. If, on the other hand, a larger penetration is obtained local high spots or hard points on the seabed may introduce excessive local loads on the base structure. Normally, these loads govern the design of the base slab.

The skirts also provide protection to the foundation from the effects of seabed scour, and skirt lengths of 1 to 5 m are used depending on the structure and soil conditions. The layout of the skirts must be chosen to allow the submerged weight of the structure to provide adequate penetration into the soils during installation. To achieve this result may require a number of innovative design features such as cutting edges, jetting locating dowels and grouting of the underside of the base slab.

1.3.3 Site investigations

A comprehensive site investigation for a concrete structure in the North Sea can cost approximately £500 000 of which about 10-15% relates to geophysical survey. This means that soils information is obtained progressively during the early stages of a project, and a detailed geotechnical survey is not usually carried out until the result of preliminary feasibility surveys is completed.

To undertake the foundation design requires measurement of soil properties to a depth below seabed of about 150 m. Under offshore conditions this has considerable practical difficulty in planning and executing site investigations. The interpretation of results from in situ tests and laboratory tests needs much skill, judgement and experience to allow the evaluation of design parameters to represent soil conditions during installation and the working life of the structure.

Successful offshore site investigation involves the use of a large number of techniques for investigating various aspects such as regional and local geology, seabed topography, soil types, their genesis and variation, and the mechanical properties of the ground under static and cyclic load. The measurement and interpretation of all these features is far from straightforward. For example, the characteristic in situ static strength of a given stratum is a function of many factors, such as soil fabric, structure, stress history and imposed stress changes, and may differ significantly from the strength derived from routine laboratory tests. These and may other difficulties and limitations must be bourne in mind at all times during the design of the foundations and other elements of an offshore structure.

Using a seafloor jack, push sampling can be carried out to obtain better quality samples. Operationally reliable in situ testing equipment, such as the cone penetrometer and remote vane, have been developed and have proven to be of value. Better quality high-resolution geophysical data can be obtained and integrated with geotechnical data to provide confident site information.

1.3.4 Design procedures

The main geotechnical problems associated with the foundation design for offshore concrete structures are:

Penetration of skirts

Lateral forces during penetration

Local base contact stresses

Stability

Settlements

Dynamic displacements

Effects of cyclic loading on soils

Hydraulic behaviour on soils

Scour action on seabed

The limit state method is now being accepted for the foundation design of concrete structures, the most important checks being on stability in the ultimate limit state and displacement in the serviceability limit state.

The effects of repeated loading lead to important changes in the stress–strain–strength properties of both sand and clay, which have to be accounted for in stability analysis as well as calculations of dynamic motions and settlements. In addition to a conventional quasi-static stability analysis, the risk of failure in cyclic loading has to be considered as a separate failure mode in the ultimate limit state. Empirical guidelines and simple methods of analysis using laboratory specimens as representative of average stress conditions are widely used to assess the effects of repeated loading on the soil properties. Finite element methods for complicated cases and centrifuge model tests for studying mechanisms and behaviour are valuable supplementary tools.

Analyses of penetration resistance of dowels and skirts, as well as reaction forces on the base structure to be expected during installation, are important considerations in the design of a concrete structure. The methods of analysis developed for these purposes, usually relying on soil data from cone penetration tests, appear to give results in reasonable agreement with field observations.

Analysis of settlements may be based on laboratory consolidation tests, but for the sands and stiff clays most common in the North Sea it appears that empirical relations between settlement parameters and the index properties of the soils produce equally good results.

1.3.5 Installation methods

The foundation design must pay particular attention to the critical installation phase as the concrete structure touches down, penetrates, makes base contact and yet preserves the foundation soils. During penetration of the skirts the structure can experience large eccentric soil resistance forces which must be compensated by eccentric ballasting moments within the caisson to maintain a vertical position. The prediction of these forces is therefore an important part of the geotechnical design problem, and methods have been found which give good correlation with the experience obtained from presently installed structures.

In dealing with the contact pressures experienced by the base during installation it has been necessary to introduce load-measuring instruments so that the calculated pressures are not exceeded. Reasonably good agreement has been observed between measurements and predicitions. Grouting of the space between seabed and the base of the structure has been successfully carried out in order to:

Keep the platform level

Avoid piping below the structure

Avoid overstressing

Avoid further skirt penetration

Secure an even soil reaction.

The experience to date has confirmed that this type of concrete structure can be installed without serious difficulty, and each structure has taken only a few hours to achieve its seabed foundation to provide for fixed stability with good locational accuracy.

1.3.6 Operational performance

All structures are reported to be performing their functions satisfactorily following some initial difficulties in the installation of oil-well conductor pipes where they have been needed. Since all the concrete structures are on hard or dense foundation soils, it has been found necessary to install conductor pipes by a combination of drilling a pilot hole and driving the conductor pipe. Each operator has evolved different techniques to minimise the disturbance of the foundation soils.

None of the structures has suffered critical settlements to date. As an example, the settlement of the Ekofisk platform from the installation in 1973 to 1977 has been 26 cm. It experienced the majority of its settlement during the first year, including a movement of 4 cm during one day, before reaching the maximum value.

Assumptions about foundation behaviour made in design are being confirmed relating to stability, soil foundation pore pressures and dynamic motions with extension instrumentation installed on virtually all concrete platforms. Although wave forces equivalent to only 45% of the maximum North Sea design conditions (100-year storm) have so far been experienced, observations indicate that pore pressure have been well within allowable limits.

1.4 Conclusions

The state-or-the-art on the various engineering aspects of the design and construction of safe foundations for concrete gravity structures in the North Sea has advanced over the last few years to give confidence using current procedures.

There are clearly many requirements to be met before both safe and economic design solutions can be found for each location. This leads to each structure relying on considerable innovation in its development for use, and a close link is needed between all of the skills in the design team. It is essential for the foundation engineer to be included at all stages of the project to give continuity of interpretation on foundation problems.

The rapid period over which these very large structures have proceeded from concept to execution has led to considerable demands for technological development being imposed on all concerned. This phase of the North Sea oil and gas offshore activity has given an unparalleled opportunity to demonstrate that concrete sea structures do provide reliable offshore platforms for drilling, production and storage of oil or gas.

The reliability of these concrete platform structures will continue to depend on their foundation performance. Predictions of long term behaviour will be confirmed from the data obtained during monitoring in use.

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2 GEOTECHNICAL INVESTIGATION PRACTICES

2.1 Introduction

Offshore concrete structures, resting on the seafloor, rely upon the underlying soils to provide adequate support against foundation instability; and to undergo small deformations so that horizontal and vertical structure movements are within tolerable operating limits. In addition to investigating geologic stratification of the supporting soil and rock strata, geotechnical properties of the strata must be determined from in situ testing and laboratory testing of core samples, in order to predict soil-structure response under anticipated structural and environmental loads.

The Ekofisk oil storage tank was the first concrete gravity platform to be installed in deep water in the North Sea in the summer of 1973 (Marion, 1974). The water depth is 70 m and the near circular structure is 90 m tall and about 93 m in diameter. The base width of concrete gravity platforms generally ranges from 75 to 125 m, so that soil and rock conditions should be explored to penetrations of about 150 m. These large and heavy structures have been placed in the North Sea on strong overconsolidated clays and dense sands to minimize potential structure deformations.

In the Pleistocene glacial period, the northwest continental glacier advanced and retreated several times across the North Sea continental shelf (Loken, 1976). The moving ice sheets eroded irregular surfaces into the underlying sediments that were infilled with glaciomarine and fluvioglacial sediments during interglacial periods, while much of the southern North Sea remained above sea level. The repeated advances and retreats of the glaciers produced complex sequences of glacial, marine and fluvial sediments which are overconsolidated from the weight of the ice sheets. Since the end of the Pleistocene period, the North Sea basin has been submerged below sea level and Holocene deposits of silts, clays and fine sands are relatively thin, except for infilling of erosion depressions remaining from the last glacial retreat. The Holocene sands are in a dense condition from the compacting effect of ocean wave induced pressures on the seafloor (Bjerrum, 1973) that have no beneficial effect upon the strength characteristics of clays. Figure 2.1 shows typical soil conditions in water depths greater than 150 m in the northern North Sea and in the central North Sea where water depths are about 75 m.

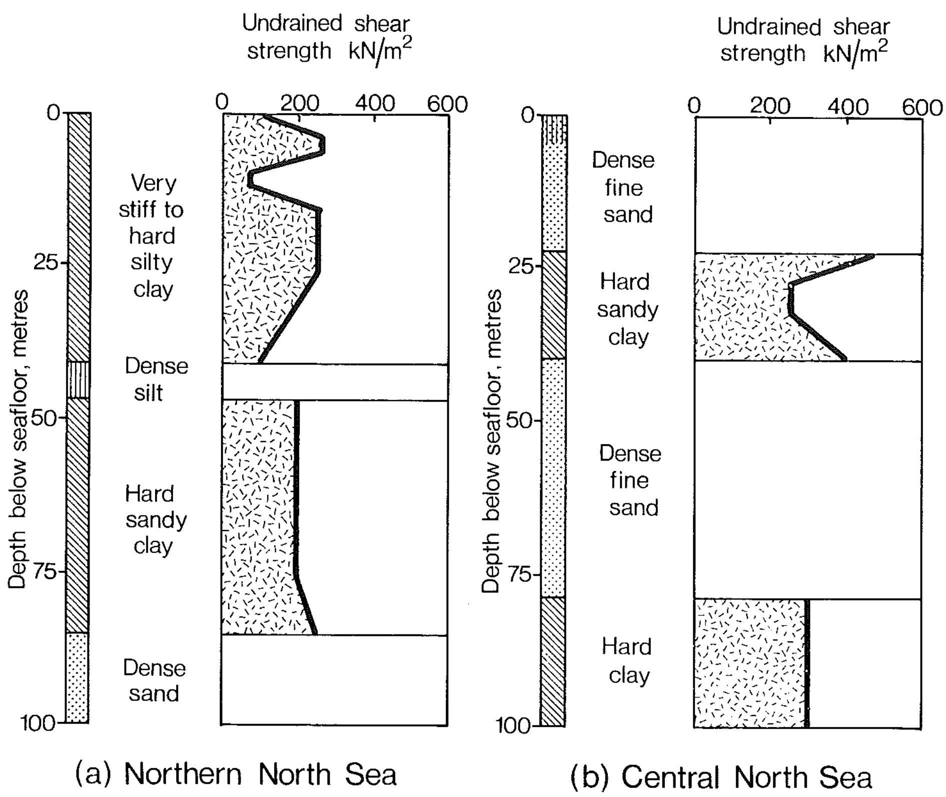


Figure 2.1: Typical soil profiles.

2.2 Planning of investigation

A comprehensive foundation investigation of a gravity platform site in the North Sea might cost between £400 000 and £600 000, of which 10 to 15% relates to the geophysical survey. Consequently, the investigation of a potential site is generally undertaken in progressive stages, so that structural concepts can be developed with due regard to soil conditions. The shallow geological environment of the area is examined from a geophysical survey, together with one or two soil borings for correlating stratigraphic boundaries interpreted from the seismic profilling data. The geotechnical investigation is ideally delayed until the results of the shallow geophysical survey have been studied and the actual platform location has been selected, with due consideration to the development of the oil or gas field.

2.2.1 Preliminary area survey

The feasibility of placing a concrete gravity platform on the seabed is frequently assessed while exploration drilling of the prospective field is still in progress. In addition to gathering bathymetric data and topographic information of the seafloor, the shallow seismic survey yields data for mapping of geologic stratification and features such as buried channels infilled with materials of differing compressibility, faults across which differential displacements might occur, and areas of seafloor slides (Milling, 1975).

Geotechnical properties of soils and rocks are not determinable from acoustic measurements, with the result that soil properties for use in feasibility studies must be developed from laboratory tests on samples recovered from one or two borings, drilled to penetrations of about 100 m. Seismic survey vessels are highly manoeuverable without anchoring capabilities to stay on location, so soil borings must be drilled with larger anchored coring vessels or by placing a mobile rotary drilling rig on the jack-up or semi-submersible oilwell platform during exploration drilling.

2.2.2 Detailed foundation investigation

Development of soil parameters for analysing platform stability, skirt penetration and soil-structure response usually involves drilling and sampling of four or more borings, to penetrations of 100 to 150 m and conducting 10 to 15 cone penetrometer tests, of which four or five tests should penetrate to about 30 m or less depending upon stratigraphy and the remaining tests to shallower penetrations of about 10 m. The areal extent of weak surface soils can be examined rapidly by additional cone penetrometer tests. Other types of in situ testing such as gamma logging, plate loading, and pressure meter have been attempted at North Sea sites to improve soils information for foundation design.

Detailed topographic data on an intended platform location are generally obtained with small submarines (Hitchings *et al,* 1976), fitted with differential pressure sensing devices and having a surface support vessel. Topographic maps can be developed to vertical accuracy of ± 10 cm and seafloor features such as bolders and marine debris are recorded on videotape and photographic film.

2.2.3 Position fixing

Marine surveys demand accurate navigating of seismic vessels along pre-selected survey lines and positioning of anchored coring vessels on soil boring and in situ testing locations. For ocean navigation, vessels are equipped with some type of low frequency shore-base radio positioning system, such as Omega, but this is not sufficiently accurate for survey work (McQuillin and Ardus, 1977). Positioning of geophysical vessels is usually achieved by one of the several medium frequency chains (Decca-Hi-Fix, Toran, or Loran) having an accuracy between 5 and 30 m or by the more accurate microwave system (Trisponder, Autotape or Hydrodist) when within radio line of sight of fixed reference points.

Positioning of coring vessels is usually controlled by an array of four acoustic transponders laid on the seafloor providing relative accuracy of ± 3 m. Laying of the transponders is done with a vessel positioned by one of the medium frequency chains, which can be operated with satellite navigation giving an accuracy better than 10 m when the vessel is held stationary for a period of about 24 hours.

2.3 Geophysical survey

Sub-bottom geologic features are explored with a multi-sensor acoustic system of towed devices having different frequency responses (McQuillin and Ardus, 1977), and other acoustic devices record simultaneously bathymetric data and seafloor topography. A typical array of towed devices is illustrated in Figure 2.2. Survey vessels in the North Sea are typically 45 to 55 m long, driven by a slow revolving single screw to minimize acoustic noise and having two conventional bow anchors. Deck winches and an A frame are required for handling the various seismic tow devices over the stern, and one or two 3 t cranes for lowering shallow penetration sampling devices on the side.

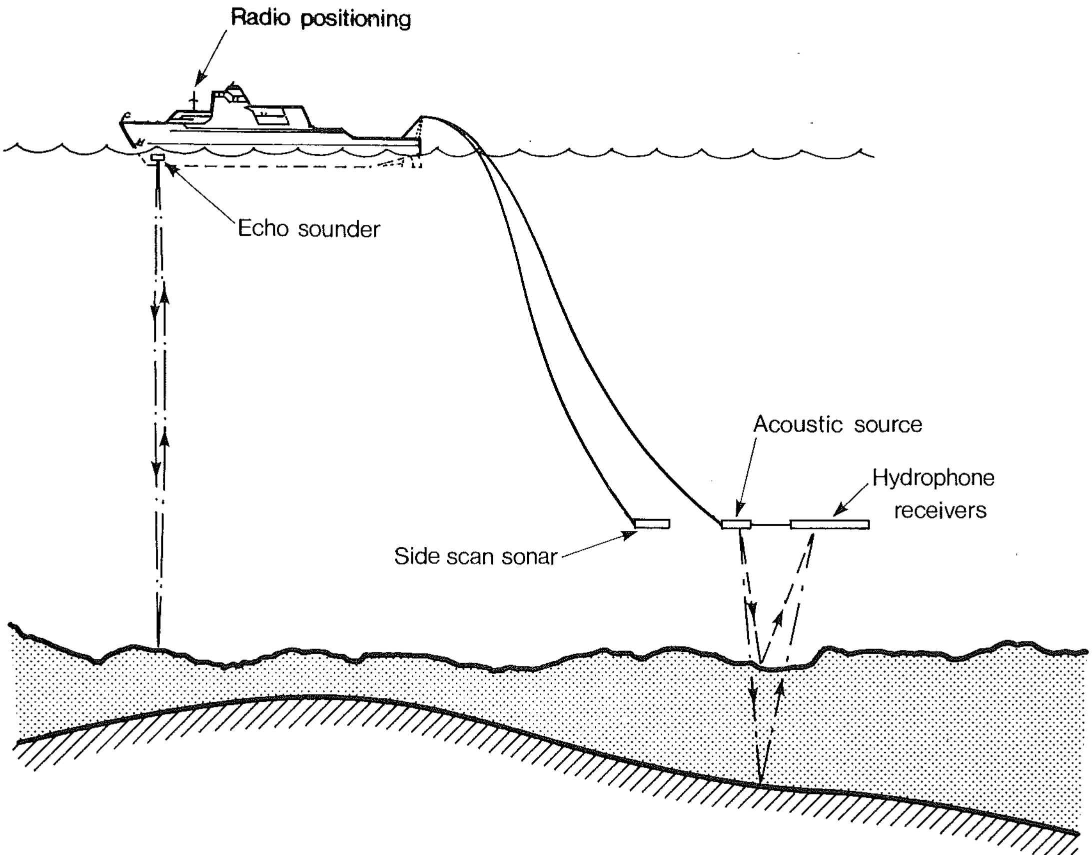


Figure 2.2: Diagram of towed seismic devices.

Geology of the area surrounding a prospective platform site is generally explored over an area of 2 to 4 km on a coarse spacing of survey lines at 300 to 500 m apart in both north-south and east-west directions. More detailed information on geologic features within the immediate location of a platform is obtained by reducing the grid spacing to 75 or 100 m in one survey direction to form an area of 1 km by 1 km. Seismic data gathered along the survey lines are routinely interpreted to construct maps showing bathymetry, shallow geologic features, shallow sediment isopachs, and construction hazards at, or slightly below, the seafloor.

2.3.1 Bathymetry

Water depth is normally measured by a high precision echo-sounder emitting high frequency acoustic signals of about 40 kHz that are reflected back to the transducer as an echo. An echo-sounder is generally mounted on a seismic survey vessel to produce a seabed contour map of the area, or on an anchored drilling vessel to measure water depth and tidal variations. Echo-sounders operating at higher frequencies up to 200 kHz are used to detect gas seeps at the seafloor.

2.3.2 Seafloor topography

A side scan sonar fish will provide a sonar picture of irregularities at the seafloor and augments bathymetric data acquired along track lines. The fish transmits high frequency pulses of 50 to 200 kHz in a thin, fan-shaped pattern in a plane perpendicular to its tow path, and receives echoes from the seafloor. The acoustic beam can scan as far as 500 m to each side of the fish but is limited to about 150 m when the fish is towed from 20 m to 40 m above the seafloor to achieve good resolution. Steel or iron objects ranging from sunken ships, pipelines, anchor, and telephone cables can be detected with a marine proton magnetometer bottle which measures the total magnetic field intensity in gammas along the tow line.

2.3.3. Continuous sub-bottom reflection profiling

High resolution seismic profiling in rough sea states that prevail for long periods in the North Sea, requires deep towing of the tow fish to minimize motion of the sound source and receiving sensor, otherwise seismic profiling operations are limited to relatively calm sea states. The deep-towing technique also reduces acoustic noise from the vessel and loss of acoustic energy from absorption and divergence in deep sea water, and improves resolution of thin soil strata. The raw seismic data are preferably digitised and recorded on magnetic tape on the vessel or recorded in analog form for processing to improve vertical resolution and to minimize multiples at the seafloor. Data from deep seismic surveys for oil and gas exploration can be used to complement the high resolution data in better understanding the geologic environment.

An appropriate seismic profiling system depends upon the required depth of penetration the desired degree of resolution, and the acoustic opaqueness of the shallow formations. Seismic survey vessels usually carry two or three types of sound sources in order to provide a broad range of frequencies, and selection of this equipment is made on the advice of an experienced geophysicist. Identification of soil and rock materials cannot usually be made based on sub-bottom reflection profiling alone so one or two soil borings are required for geologic correlation of the seismic data.

Table 2.1. High resolution profiling systems.

Acoustic Source	Energy Joules	Frequency Hz	Resolution m	Penetration m
Stacked sparker	4000-10 000	80-200	± 10	350-900
Sparker	20-200	500-1200	3-4	15-100
Deep tow sparker	200-800	1000-3000	0.5-1.5	15-60
Multi-electrode sparker	200-1000	300-3000	1.5-3	15-120
Boomer	500-1000	300-3000	2-4	30-120
Deep tow boomer	400-600	800-1000	0.5-1.5	15-60
Precision boomer	100-500	400-15 000	0.5-1	15-75
Microprofiler	1-100	2000-12 000	± 0.5	max. 30

Generally low frequency, high energy systems produce deep penetration with low resolution whereas high frequency systems yield limited penetration with high resolution. The common types of seismic profiling systems used in the North Sea are listed in Table 2.1 together with typical operating characteristics such as energy, frequency, resolution, and penetration. High energy systems used in engineering surveys are operated at frequencies between 100 and 400 Hz. capable of producing penetrations of 200 to 300 m with resolution in the order of 6 m below penetration of 10 to 20 m. Higher frequency multi-electrode sparkers achieve less penetration to about 150 m with improved resolution of 3 m while boomers operating at about 2 kHz can penetrate 50 to 75 m with resolution of 1 to 1.5 m. Low energy micro-profilers and pingers emitting a sound pulse with a frequency of 5 kHz give resolution of about 0.5 m and can penetrate about 25 m into soft clay but only a few metres into dense sand and gravel.

2.3.4 Shallow penetration sampling

Seabed sampling with gravity corers from seismic survey vessels produces soil samples suitable for classification testing to map shallow soil stratigraphy, but generally unsuitable for providing reliable soil shear strength data. Piston or open-barrel gravity corers are lowered to the seafloor on a wireline and the samplers frequently have a triggering weight, so that the sampling tube falls freely over a pre-determined distance before penetrating the seafloor (Noorany, 1972). The diameter of the sampling tube of these one-shot devices ranges from 50 to 150 mm, and penetrations greater than 5 m are seldom achieved in soft to firm clays. The length of undisturbed clay samples recovered in a sampling tube is restricted to 15 to 20 times the inside diameter of the sampler, while in sands the ratio is only about 10. Consequently, when the penetration of a 75 mm sample exceeds about 1.25 m, the recovered samples suffers some disturbance.

Another shallow penetration technique is to drive a thick-wall tube, which can range from 100 to 270 mm in diameter, into the seafloor with a vibratory hammer. The vibro-corer is lowered on a steel cable to the seafloor with a tether line for flow of compressed air to the mechanical vibrator. Penetration ranges from 8 m in soft-to-firm clays and loose sands to only 0.5 to 2 m into strong clays, because of damping of the vibratory energy by the surrounding clay.

A remote controlled seabed sampling device, designed to carry out push sampling to 10 m, has undergone sea trials in the North Sea. The seabed unit consists of a rotary drilling device with a rotating supply disc carrying 10 sampling tubes of 55 mm diameter and 90 cm long. A tube is pushed from the seafloor, retracted and stored in the supply disc, and then the unit drills a hole to 0.9 m before another sample tube is pushed from 0.9 to 1.8 m. This push sampling and drilling sequence is continued to penetration of 9 m.

2.4 Geotechnical investigation

Vessels for drilling borings and conducting in situ testing must be capable of remaining in position for periods of several days in moderate sea states. These vessels are usually 65 to 75 m long, driven by twin screws with a bowthruster so that three bow anchors and three stern anchors can be laid without assistance from an anchor handling boat. Anchoring in 200 m of water requires at least 1750 m of wireline attached to each 3 t anchor and the drum of the anchor winches should hold about 2250 m of wireline. The vessels have a 4 m by 4 m moon pool through which seafloor devices are lowered and raised, and are fitted with a permanent drilling derrick and draw works.

2.4.1 Rotary drilling and wireline sampling

The North Sea vessels utilize 115 to 130 mm drill pipe with drill collars and an open-centre drag bit at the bottom of the string, and the top of the string is connected to the motion compensator in the crown of the derrick so that the drill string is constantly held in tension. The drill string is internal flush of 100 mm internal diameter, allowing wire line tools up to 90 mm in diameter to be run through the drill pipe. Borings are advanced by rotating the drill pipe with a power swivel or power tongs and the drill remains in the drilled hole until the boring is completed. The drilling procedure shown in Figure 2.3 requires a continuous re-supply of drilling mud of desirable viscosity and fluid weight for stability of the borehole, since all the drilling fluids exist from the hole at the seafloor. Drilling and sampling operations are generally closed down when vertical vessel motion approaches 3 m.

The most commonly used sampling technique is percussion sampling. Percussion sampling, illustrated in Figure 2.3 is accomplished by driving a 75 mm thin-wall tube into the soil below the bottom of the boring by blows of a 130 kg hammer dropped approximately 1.5 m. The sliding hammer is attached to the sampler and is operated on a wire line. The technique is fast and effective in procuring samples, but produces significant disturbance of clay samples and no quantitative measure of the in situ condition of sand samples. Sample disturbance is reduced by push sampling but is restricted to relatively calm sea conditions, because the drill pipe produces the necessary reaction load. This technique involves latching the sample tube into the drill bit that is pressed into the ground by reducing the tension load in the drill string.

Controlled push sampling in stiff clays and sands has been conducted in North Sea borings to penetrations of 125 m, with the seafloor jacking unit of Stingray. The 20 t seafloor jacking unit has hydraulically operated horizontal clamps, which grip the drill pipe and the vertical hydraulic rams, having a stroke of 1 m, push the latch-in 75 mm sampling tube into the soil. The weight of the seafloor jack prevents vessel motion being transferred to the sampling tube.

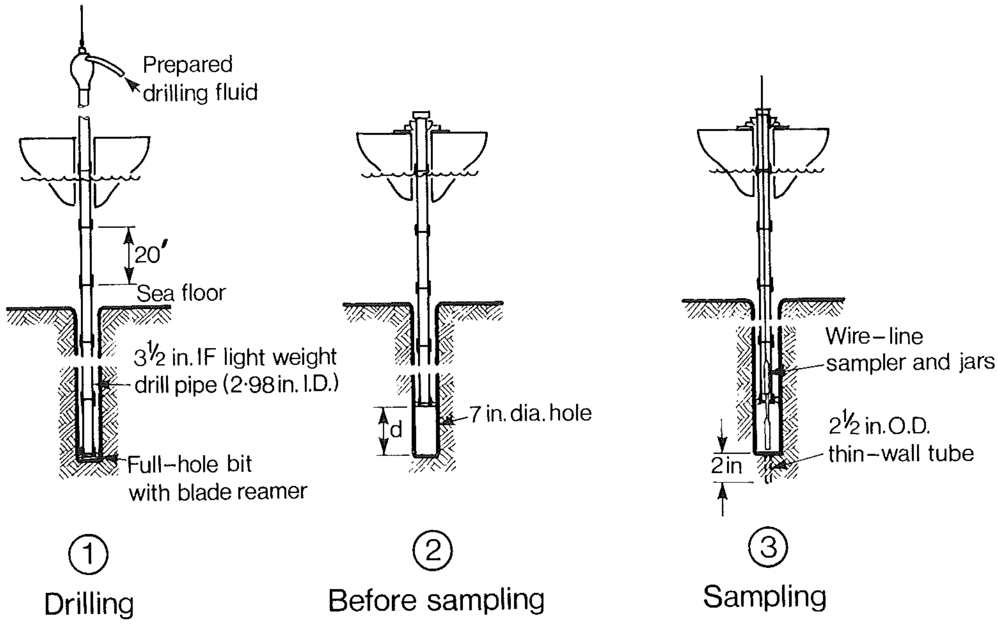


Figure 2.3: Wire-line percussion sampling.

The two lifting lines to the seafloor unit serve as a guide system to lead the bit through the jack and back into the drilled boring, thereby facilitating replacement of the open-centre drag bit with a rock bit, whenever rotary rock coring is necessary. By using a wire line core barrel latching into the drill bit, cores of 40 to 50 mm in diameter can be recovered inside a plastic liner.

Push sampling has been successfully performed with Stingray in strong, over consolidated North Sea clays with undrained shear strength approaching 500 kN/m^2. Percussion wire line sampling was also conducted in the same clays and revealed that the sampling technique had no marked effect upon the shear strength, but the solid stiffness was reduced by 25 to 50% (Sullivan, 1978). Comparative sampling tests in a moderately sensitive clay, with undrained shear strength increasing from 40 to 100 kN/m^2, have shown that percussion sampling produced sufficient disturbance to lower the strength by 25 to 40% below strengths measured on push samples (Emrich, 1970). Push sampling provides better quality samples, thereby allowing greater confidence in selecting design soil parameters that in turn reduces conservatism in foundation design.

Recovered pushed or hammered samples are carefully extruded from the 75 mm sampling tube, allowing visual-manual identification of the soils before packaging in cardboard or plastic tubes and selected samples can be retained and sealed in the steel sampling tube for transport to an onshore laboratory. A sample is first wrapped in plastic foil, then in aluminium foil and the annular space between the soil specimen and the tube is filled with non-shrink molten wax. Disturbance of stiff soils from extrusion is minimized by immediate packaging in cardboard tubes, while samples retained in the sampling tubes are exposed to drilling fluid trapped in the top portion of a sample that can cause swelling of stiff clay with associated loss of strength.

2.4.2 Cone penetrometer testing

This in situ testing technique is widely used in the North Sea to explore soil conditions, since a test can be performed quickly giving a continuous record of resistance. Interpretation of strength and stiffness of clay and sand from cone results is based on empirical rélationships and requires mature geotechnical judgement.

The seafloor cone penetrometer unit Seacalf has been used considerably in the North Sea (de Ruiter, 1975) and a diagram of the unit is given in Figure 2.4. It is a 20 t unit lowered to the seafloor through the moon pool of a drilling vessel and provides the reaction for thrusting a 10 cm^2 cone into the soil, at a rate of penetration of 2 cm/s. The cone test rods extend above the top of the seabed unit requiring support from a motion compensated wire line and cone penetration is limited to 6 to 8 m in hard clays or dense sands. The Wison wire line cone pentrometer was developed to conduct cone testing at a greater depths in companion borings by latching into the drill bit. It relies upon the drill string to provide reaction, with the result that the rate of cone penetration is not closely controlled and cone penetration into stiff clays is limited to about 1 m, because the reaction load from the drill string seldom exceeds 3 t.

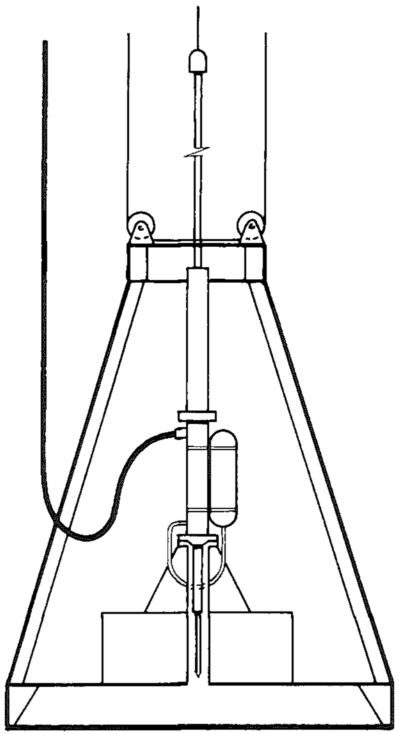
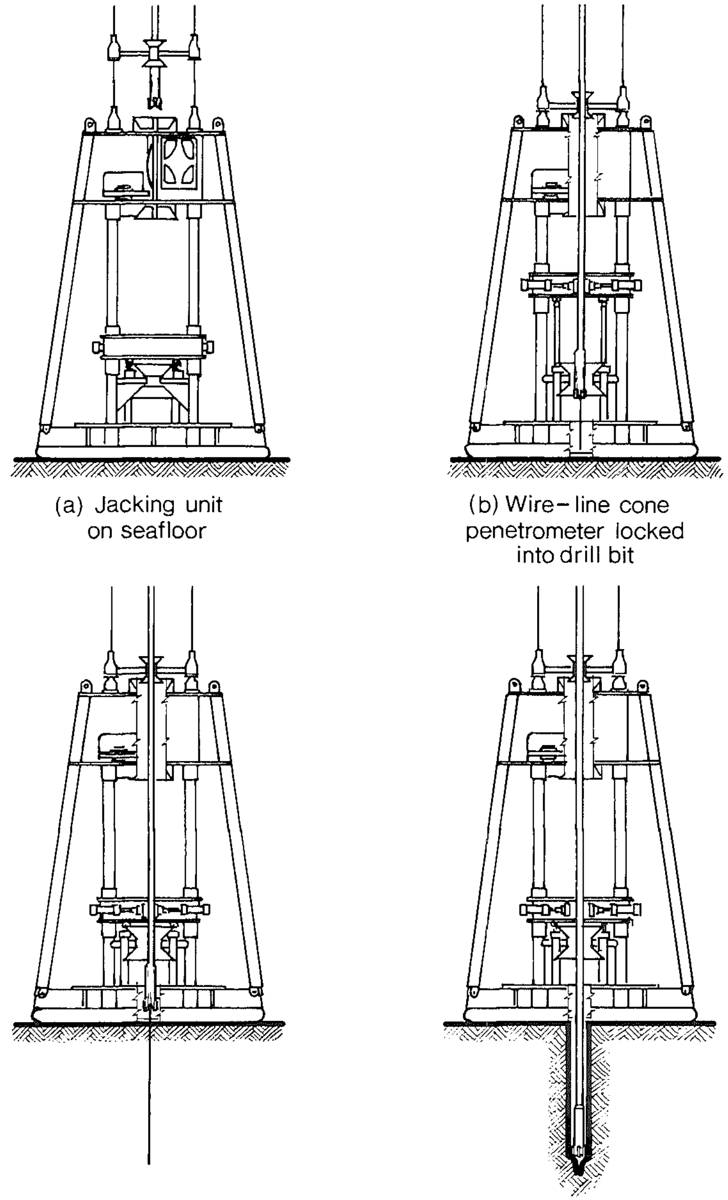


Figure 2.4: Diagram of seacalf.



(a) Jacking unit on seafloor

(b) Wire-line cone penetrometer locked into drill bit

(c) Cone pushed to 5-m penetration by repeated jack strokes

(d) Hole drilled to 5-m depth ready for next cone test cycle

Figure 2.5: Operation of Stingray.

A successful seafloor jacking device is Stingray (Ferguson *et al,* 1977) which has performed cone testing from the seafloor through hard clays to 25 m penetration. Cone penetrometer testing with the versatile seafloor jacking unit is shown in Figure 2.5. The vertical hydraulic rams force penetration at 2 cm/s of the 10 cm^2 cone in increments of 1 m until the 5 m rod length is fully utilized or refusal is reached. The cone and rod are retrieved by the wire line procedure and the drill pipe is advanced by conventional rotary drilling to a level just short of the cone's maximum penetration. Thereafter, the sequence of pushing the cone and rotary drilling is repeated to any desired depth.

2.4.3 Other in situ testing

Soil conditions are investigated with other currently available in situ testing devices. Since continuous soil sampling in deep penetration borings is not standard practice, gamma logging has been used (Guyod, 1963) to obtain a continuous profile of soil stratification. The measuring sonde is run through the drill pipe since natural gamma rays from clay materials have good transmission through steel. Density and porosity logging through the drill pipe is not effective because of the influence of the drill pipe and the mud filled gap between the drill pipe and the side of the boring upon the transmission of neutron-gamma and neutron-neutron rays to the measuring sonde.

Load testing of a 30 cm diameter plate has been carried out on the seafloor in the North Sea to assess contact stresses and penetration resistance of domes on the bottom of the concrete base of a platform. The plate loading test was performed using a remote controlled seabed device such as the Seacalf unit.

Although in situ vane shear tests have not been conducted in the North Sea, this type of test is commonly used in the Gulf of Mexico (Doyle *et al,* 1971), to determine the undrained shear strength of firm-to-stiff clays. A remote wire line vane is available that is implanted into the soils below the bottom of the boring under the weight of the drill pipe or pushed into the soil by the Stingray seafloor jack to measure undrained shear strengths less than 125 kN/m^2. The remotely controlled 65 mm diameter vane operates independently of the drill pipe and has a set of reaction blades positioned above the test vane to provide the necessary torque reaction.

Pressure meter testing has been performed in oversized drilled holes (Menard, 1957) and with devices jetted or vibrated into soft clays and sands. These techniques to insert the pressure meter into soils alter the stress-strain response of the surrounding soil due to stress relief, or disturbance, or densification. More recently the stress-strain properties and lateral stresses in soils have been successfully investigated on land with self-boring pressure meters (Baguelin *et al,* 1972) and pressure meters pushed into soil over an undersized pilot hole. These installation techniques ensure intimate contact between the tool and the surrounding soil, and minimize disturbance which influences the in situ soil measurements. Modifications to these devices for offshore investigations are being undertaken and a successful sea trial was made in 1977 to insert a 90 mm push type pressure meter into stiff clay with the Stingray seafloor jack.

2.5 Laboratory testing

2.5.1 Shipboard testing

Soil testing on board the drill ship is limited to routine classification tests and determination of undrained cohesive shear strength on about 25% of recovered samples, thereby leaving ample material for detailed testing at an onshore laboratory. Upon extrusion from the sample tube, visual-manual description of colour, plasticity, grain size, fabric and structure are recorded for the sample in addition to percentage of sample recovery and blow count, when sampling with a wire line hammer. Natural moisture content and density determination is made on most clay samples and the consistency is estimated from hand penetrometer, miniature vane, and fall cone tests. Sieve tests are performed on selected sand samples for grain size identification and shear strength of clay samples is determined from unconsolidated undrained triaxial tests.

2.5.2 Onshore testing

Testing of samples in an onshore laboratory is directed towards evaluating soil shear strength, soil stiffness under static and dynamic loading, loss of shear strength under repeated wave loading, and consolidation under the weight of a platform. Moisture content and density determination are made on all undisturbed samples and soil macro-structure is recorded by taking colour photographs of split samples. Classification tests of liquid and plastic limits are performed on clay samples, grain size analyses are conducted on sand samples, and both liquid and plastic limits and grain size analyses are carried out on glacial clay tills. Selected sand specimens are viewed under a microscope to describe grain shape and mineralogy.

Shear strength and soil stiffness (modulus) of clays and sands are evaluated by various types of triaxial tests, and simple and direct shear tests in an effort to reproduce in situ stress conditions and anticipated modes of soil deformation beneath a platform. Consolidated-undrained triaxial compression and extension tests are carried out on 75 mm diameter specimens under conditions of no lateral yield (K_o) and conventional testing conditions allowing lateral sample yielding. Pore pressure measurements are recorded so that effective stress paths can be computed and and plotted for each test. Simple shear tests are conducted under undrained conditions with pore pressure measurements, while direct shear tests are restricted to drained loading conditions. Soil shear strength of stiff brittle clays and dense sand is interpreted from the maximum shear stress whereas the shear stress associated with a critical strain is often used with softer soils exhibiting significant plastic deformation. Oedometer tests are performed on undisturbed clay samples to investigate soil consolidation characteristics under anticipated platform loading.

Ocean wave loading of foundation soils can produce failure in sand as well as in clay by cyclic loading. Liquefaction of sand (Lee and Focht, 1975) is the better known phenomenon but a similar failure mode may be critical for foundations on clay. Clays will also suffer reduction in undrained shear strength and deformation modulus (Andersen *et al,* 1976), before pore pressures generated by ocean waves are dissipated. Two-directional cyclic triaxial and/or simple shear tests are conducted to investigate the response of sand and clay subjected to sinusoidal loading. In cyclic load testing, the period of ocean waves is customarily taken as about 10 s, which is an order of magnitude greater than periods of 0.5 to 1 s used in earthquake engineering. Failure in cyclic loading is generally considered to occur when accelerating shear strain develops and this condition takes place at cyclic shear stress levels much less than the undrained strength of the soil. Model testing in a centrifuge is another technique for investigating displacements and failure modes of foundation soils beneath a platform under combined static and ocean wave loading (Rowe, 1975).

The engineering properties of foundation soils vary both with penetration below the seafloor and laterally across a site. Consequently it is desirable to perform semi-continuous sampling in some borings, to provide sufficient quantity of representative soil for testing and developing correlations. Sampling in sands yields disturbed specimens resulting in laboratory tests being conducted on remoulded samples of granular soils. Sands in situ, as well as clays, have fabric so the results of cyclic testing on remoulded sands may differ from the in situ response of a sand stratum.

2.6 Evaluation of design soil parameters

Evaluation of design parameters for overconsolidated North Sea glacial soils must consider the wide variation in composition, fabric and stress history resulting from the complex depositional and post-depositional process. Consequently, quantitative interpretation of the results of small in situ tests and laboratory tests on small samples should be undertaken with due regard to the macro-response of the foundation soils. Current practice is to select design soil parameters based primarily on the results of extensive laboratory testing and relying upon in situ testing to provide supplementary data. The variability of soil properties at a site is assessed from classification tests on soils samples, from in situ testing, and from geological information interpreted from geophysical data.

2.6.1 In situ testing

By inserting small devices into the ground, the response of the surrounding soil to applied vertical or horizontal forces can be measured. The cone penetrometer test is the most common in situ test performed in the North Sea and interpretation of the results is based on the empirical relationship for the cone factor, N_k, which is commonly taken as about 17 for shallow overconsolidated clays. Based on correlations between cone tests and other forms of small scale laboratory or field tests, it is customary to assign a value to N_k between 15 and 20 for stiff overconsolidated clays, and a lower range from 10 to 15 for normally to lightly overconsolidated clays. However, research using the undrained shear strength determined from 865 mm diameter plate tests on a stiff overconsolidated glacial clay, reveals that N_k can vary from 1 to 27. Figure 2.6 (a) shows a comparison between cone resistance and bearing capacity from a large plate in glacial clay at a UK onshore site at Redcar (Marsland, 1977) where a value of N_k of 27 is required to give the correct characteristic strength. The wide variation in N_k is

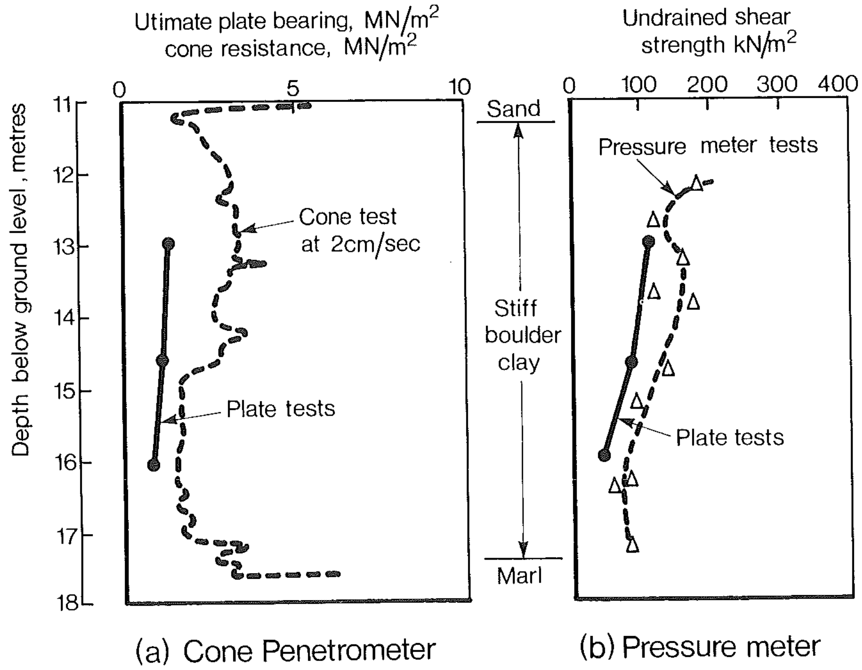


Figure 2.6: Comparison of soil strength from different in situ tests.

influenced by soil composition, scale and nature of the soil fabric at both micro-and macro-levels, the in situ stress level in the ground, and the speed of cone penetration. These factors also influence the interpretation of vane tests which in stiff fissured clays can overestimate the undrained strength by a factor or two.

Pressure meter tests in pre-drilled boreholes in glacial clay at Redcar indicated shear strengths of 100 to 150 kN/m^2 which was slightly greater than strengths determined from deep large plate tests, however, the lower trend line was very close to the average shear strength from the plate load tests as illustrated in Figure 2.6 (b). Soil stiffness as measured by the shear modulus from the pressure meter, ranged from 20 to 33% of the modulus determined from the large plate tests on the glacial clay. These results emphasise the difficulties in forming undisturbed holes in clay tills.

2.6.2 Laboratory testing

In a laboratory, testing is conducted on small samples extruded from thin wall sampling tubes either pushed or driven into the soil with a sliding weight. Soil fabric and structure must be considered in selecting characteristic strength and stiffness values, since laboratory testing of conventional size soil samples may significantly overestimate the in situ strength of stiff fissured clays. Figure 2.7 (a) and 2.7 (b) show the comparison on undrained strength and soil stiffness from high quality 98 mm samples, with the values obtained from large plate tests in glacial clay at Redcar (Marsland, 1977). These results reveal the difficulty in selecting characteristic strength and stiffness values from the results of laboratory tests, and emphasise the need for further research on the influence of soil fabric, structures, and stress history on the interpretation of laboratory tests.

The undrained shear strength of clays for use in total stress analyses is usually determined from triaxial and simple shear tests to simulate boundary conditions along a potential failure surface beneath a platform. Results of each individual strength test are interpreted from stress or strain criteria, to produce a design strength. The characteristic strength profile is then selected from conservatively assessed mean values of the design strengths for each soil stratum.

When a platform is placed on the seafloor the underlying clay soils undergo elastic deformation, followed by consolidation settlement, until the excess pore pressures are dissipated, when secondary creep may occur. The undrained Young's modulus of the soil is interpreted from triaxial tests to determine the immediate settlement. Consolidation settlement is the major component of the total settlement of a platform and is computed from the results of oedometer tests in which the preconsolidation pressure of a specimen must be determined accurately. Since vertical soil stresses beneath concrete gravity platforms are less than the soil pre-

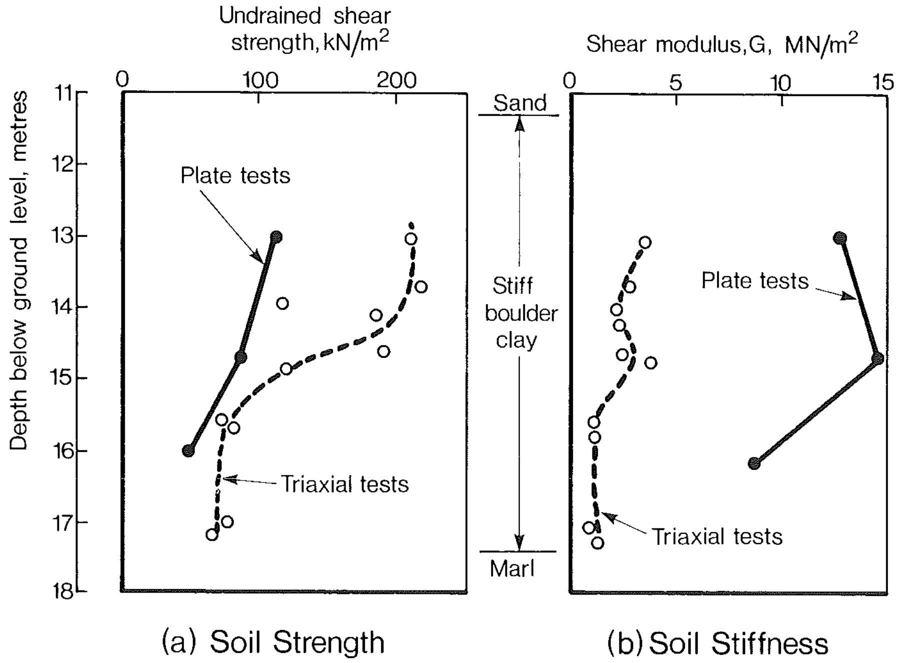


Figure 2.7: Comparison of results from deep plate load tests and laboratory tests.

consolidation pressure, oedometer tests should include one unload and reload cycle from a pressure greater than the pre-consolidation pressure of the test specimen because consolidation settlement of stiff clay should preferably be computed from the reloading curve. Results of oedometer tests tend to under predict the rate of settlement of structures resting on over-consolidated clays.

2.7 Conclusions

A foundation investigation of a concrete gravity platform site in the North Sea costs about £500 000, so careful planning is required in the selection of suitable positioning systems for survey vessels, geophysical instruments to explore shallow geology, and sampling and in situ testing techniques to acquire good quality geotechnical information. Investigations are generally undertaken in progressive stages so that structural concepts can be developed with regard to soil conditions, and the scope of each investigational stage can benefit from information already acquired at the site.

Continuous high resolution reflection profiling across a prospective platform site provides suitable records to map geologic stratification and features which include hazards such as faults and shallow gas pockets. Several types of acoustic sources have been developed over the last 20 years ranging from high frequency, low energy systems that achieve limited penetration with high resolution, to lower frequency and higher energy systems producing deeper penetration to about 750 m with less resolution. Recent development of deep-towed acoustic devices minimises noise from a survey vessel and improves resolution of thin soil strata, while digitising of raw seismic data permits computer processing thereby allowing greater scope in analysis of the data with improved interpretation of geologic stratification and features.

Geotechnical properties of soils and rocks to penetrations of 150 m are determined from laboratory testing of recovered samples. Shallow penetration sampling, with gravity corers and vibro-corers, produces some sample disturbance which will impair the strength of the soil. Recent development of a remotely controlled sea-bed sampling device will permit recovery of better quality samples to penetrations of about 10 m. Deep penetration sampling requires rotary drilling and wire line percussion or push sampling from an anchored vessel. Push sampling, with a seafloor jacking system, improves both sample quality and length of core recovery. Stiff overconsolidated clay samples removed from below the sea and exposed to the atmosphere tend to swell due to stress relief but the effect of stress relief upon the undrained strength may be minimal because laboratory tests tend to overestimate the characteristic strength of stiff clays; however stress relief has a significant effect upon soil stiffness. Efforts should be made to develop a sampler to minimize the effects of pressure changes on recovered samples.

Increased emphasis is being placed on in situ measurements of soil properties by inserting small devices into the ground, such as cone penetormeter, pressure meter and remote vane in softer clays. These small devices can overestimate the characteristic strength of stiff boulder clays, since they do not measure the macro-response, which is influenced by soil fabric and structure, nevertheless the pressure meter provides better estimates of soil stiffness than values obtained from laboratory tests on recovered samples. With improved analytical procedures a push-in piezometer will be needed to supplement pressure meter tests for evaluating in situ effective stress conditions. Development of new in situ testing equipment should recognise the need for adaptability to operating in a harsh marine environment otherwise the cost effectiveness of even the most sophisticated devices will quickly vanish.

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3. FOUNDATION DESIGN METHODS FOR GRAVITY STRUCTURES

3.1 Introduction

In general, the foundation design of a gravity structure may be said to be the process of selecting the structural configuration of the base structure, such as size, skirt configuration, slab thicknesses, and such like. In this report, however, the emphasis is rather on the considerations behind the choice of a particular configuration, in particular related to investigations of the adequacy of a prepared design. We shall concentrate on the following aspects:

Specification of geotechnical problems which must be solved during the final design of a gravity structure;

Assessment of the tools available for solving these problems;

Assessment of the current knowledge regarding determination of the soil properties entering into the various analyses.

In addition to his geotechnical skills, the engineer analysing the foundation for a gravity structure must also have a good understanding of the environmental loads acting on the structure, as well as the structural design and the operational requirements. For the North Sea, which is the main source of experience for this contribution, the environmental loads arise mainly from wave loading, but for other areas the design may be governed earthquake loading.

In the analysis of the foundation design it is most common to make use of theoretical methods. Accordingly, the major part of this report concentrates on such methods. However, model testing has also proved to be a useful tool and these methods are considered in Section 3.9.

The geotechnical problems related to the foundation design of large gravity structures were critically examined by Bjerrum (1973a). More recent contributions to the same topic were given by Eide (1974), Yong, Kraft and Focht (1976), and Hoeg (1976).

3.2 Design principles

In the analysis of geotechnical problems, the safety consideration is usually introduced in one of two ways: (1) as an overall safety factor, normally applied to the shear strength of the soil, or (2), as partial safety factors, normally applied to loads and shear strength. The first approach is the traditional one, and may be considered as a special case of the second. The second method reflects the principles of limit state design and is therefore usually referred to by this name.

The limit state method of design is widely accepted for concrete gravity structures and increasingly also for their foundations. In this method the level of safety of the design is reflected in load factors associated with characteristic loads and material factors related to the characteristic material strengths. Thus, the partial safety factors will depend on the definition of characteristic loads and strengths and on the limit state under consideration. The main advantage with this method is that a uniform standard of safety level may be achieved for widely different conditions as regards geometry and load combinations. A structure or part of a structure is considered unfit for use when it reaches a particular limit state, in which it infringes one of the criteria governing its performance or use. In one of the most recent rules (NPD 1977), the limit states are categorized as follows:

The ultimate limit states, (ULS), related to the risk of failure or large inelastic displacements or strains of a failure character.

The fatigue limit states, (FLS), related to the criteria associated with the effect of repeated loading.

The limit states of progressive collapse, (PLS), related to the risk of failure of the structure under the assumption that certain parts of the structure have ceased to perform their load-carrying functions.
The serviceability limit states, (SLS), related to the criteria governing normal use or durability.

The DnV regulation, (1977), gives a similar definition of these limit states whereas FIP, (1977), considers only the ultimate and serviceability limit states which are defined in more detail.

The possible failure modes, i.e., instability of the foundation, with due account to hydraulic effects, are to be considered for the ULS. The characteristic shear strength of the soil used in the calculations must take into account the effects of repeated loading.

The requirements for partial safety factors in the ULS according to several recent recommendations and rules are given in Table 3.1.

Table 3.1: Load factors and material factors in current use.

Reference	**Load factors**				**Material factors**	
	P	**L**	**D**	**E**	**Cohesion or undrained strength**	**Effective friction**
FIP (1977)	1.1/0.9	1.3/0.9	1.1/0.9	1.3	1.4	1.2
DnV (1977)	1.0	1.0	1.0	1.3	1.3	1.2
NPD (1977)	1.0	1.0	1.0	1.3	1.3	1.2
DEng (1977)	1.0	1.0	1.0	1.0	1.5*	1.5*

Note: P = permanent load D = deformation load
L = live load E = environmental load
* = Overall factor of safety

In a total stress analysis the material factors for undrained strength will apply. In a drained or undrained effective stress analysis, the material factor for cohesion applies to the cohesion intercept (or the 'attraction') and the material factor for effective friction to the $\bar{\sigma}$ tan ϕ component where $\bar{\sigma}$ is the effective normal stress and Φ is the angle of friction. Material factors for pore pressure have not been defined and are normally assumed equal to unity.

According to the *FIP Recommendations,* two sets of load factors are given for extreme loading conditions. Normally the higher load factors given for P and L loads apply to foundations on clay, whereas the lower value, $\gamma_f = 0.9$, apply to foundations on sand. In principle, however, both alternatives should be considered in each case. For deformation loads the most unfavourable value is chosen in each case.

The DnV and NPD rules are in complete agreement with respect both to load factors and material factors, whereas the DEng regulation gives load factors $\gamma_f = 1.0$ for all types of loads. This makes the material factor an overall factor of safety.

When using partial safety factors the resulting safety level, considering both the load factor and the material factor, depends on the ratio of horizontal to vertical load. This ratio varies with the sea state variations during the life of the platform. It is thus necessary to consider the transition from ordinary to extreme loading condition and the resulting variation in safety level. In order to assure a minimum acceptable safety level for the entire range of environmental loading conditions a load factory $\gamma_f = 0.7$ combined with $\gamma_f = 1.3$ for P and L loads and $\gamma_f = 1.0$ for D loads. In the *FIP Recommendations,* the load factors for these conditions are $\gamma_f = 1.2$, $\gamma_f = 1.6$, $\gamma_f = 1.1$ (0.9) and $\gamma_f = 1.4$ for P, L, D and E loads, respectively. The factor of safety for ordinary loading conditions according to the DEng regulation is 2.0.

The settlements and displacements, including differential settlements, permanent horizontal displacements and dynamic motions have to satisfy the serviceability requirements for the actual platform. None of the references in Table 3.1 gives specific requirements to the SLS. These will therefore have to be agreed upon between the Owner and the appropriate regulatory agency. The criteria to be agreed upon are listed in Table 3.2. In addition, the rate of deformation may be important. Adjustments may occur under very slow rates, but if the same magnitude occurs at a faster rate it may cause distress within the structures or between the structure and conductors and pipelines.

The fatigue limit state, (FLS), is normally not checked independently, but the effects of repeated loading on shear strength and stress deformation properties are taken into account in the analysis for the ULS as well as for the SLS. In particular, it is important to consider the possibility of failure in cyclic loading, see 3.8.

Table 3.2: Criteria to be agreed upon for the SLS.

Type of deformation	Criteria with respect to
Seafloor deformation due to reservoir subsidence	Total settlement
Settlements	Total settlement Tilt Differential settlement Settlement after installation of conductors Settlement after installation of pipeline risers Rapid settlements during storms
Long-term lateral displacements	Total After conductor installation After installation of pipeline risers
Dynamic displacements due to wave action	Effects on structure (extreme load effects and fatigue) Effects on equipment Human reactions

The progressive collapse limit state has not yet been applied to North Sea gravity structures.

The most important problem in the foundation design concerns the stability of the foundation, under the combined influence of vertical and horizontal forces transferred through the structure into the foundation soil. The wave-induced forces are, of course, dynamic in nature but in the stability analyses they are treated as quasi-static forces multiplied by appropriate dynamic amplification factors, which are functions of the foundation spring constants. The design extreme loading condition is normally associated with the 100-year storm. For this loading condition, two failure modes are considered—the quasi-static analysis, the foundation is designed to resist, with a prescribed level of safety, the probable largest wave force occurring once during a period of 100 years. In addition to loads acting directly on the structure, the effects of wave loads on the seafloor next to the structure should be considered. Failure in cyclic loading, a failure mode which has received less attention than quasi-static failure, is the accumulated effect of the many waves during a storm, which may introduce a complete failure, even if all load amplitudes are considerably below the undrained strength of the soil. This failure mode includes the phenomena of liquefaction and cyclic mobility, which have been extensively discussed for sand (Casagrande 1976), and Seed (1976), as well as similar phenomena for clay (Foss, Dahlberg and Kvalstad (1978)).

For the installation phase, a design installation wave is determined, based on the platform location, duration of the installation work, season of the year, and the consequences of the predicted environmental conditions being exceeded. This wave is normally smaller than the 100-year wave.

It might be speculated whether the quasi-static assumption results in a realistic design criterion. An alternative would be to design against deformations in the extreme loading conditions, as the limited duration of the loading will limit the displacements even when the strength of the soil has been exceeded. For typical gravity structures, however, very large deformations result even from a small exceedance of the total resistance. Therefore, the conservatism involved in the quasi-static approach is of minor significance: the additional resistance due to inertial effects is not sufficient to prevent the occurrence of large deformations.

In general, the safety level of the foundation must be controlled both for the installation phase and for the operation phase, applying the most unfavourable load combinations.

The characteristic value of the soil properties has to be established in each different layer. NPD, (1977), and DnV, (1977), define the characteristic value as a conservatively assessed mean value, based on the results from laboratory tests and in situ tests. The stress conditions during testing, as compared with the actual stress conditions in the layer considered, have to be taken into account. Correspondingly larger conservatism is to be applied in the assessment of characteristic values if the number of tests is small or the scatter is large.

Although onshore foundation design experience contributes extensively to the design philosophy under development for offshore structures, it should be bourne in mind that the size, loading conditions, method of installation, etc., typical for a gravity structure, have forced the offshore foundation engineer to develop geotechnical solutions which in many ways are unique for offshore conditions.

The geotechnical problems associated with offshore gravity structures are: (1) foundation stability, (2) local contact stresses against the base structure during installation, due to uneven sea bottom, (3) penetration of skirts, (4) settlements and dynamic displacements of the structure, (5) effects of repeated loading on soil properties, (6) hydraulic effects leading to softening of the soil and consequent reduction of bearing capacity, (7) scour and adequate means for scour protection and (8) marine slope stability.

In the following, these problems will be defined in more detail, followed by a review of the analytical methods currently in use for solving the actual problem and the input soil parameters required for these analyses. The presentation is based upon the assumptions that:

All loads are well defined;

Adequate site investigations have been carried out;

All necessary laboratory investigations have been performed.

These assumptions are not necessarily valid in a specific case, and it is of utmost importance that the geotechnical engineer has a feel for the confidence which he can place in his assumptions. In order to identify the critical parameters, a sensitivity study should be carried out for all important analyses. By assigning a range of values to the parameters the effects on the final result will indicate how to make a suitably safe assumption for each parameter.

3.3 Stability

3.3.1 Problems

This section considers quasi-static stability failure only. Failure in cyclic loading as a separate failure mode is considered in 3.8.

The critical loading condition for the foundation of an offshore gravity structure originates from a combination of vertical and horizontal forces, the latter being predominantly wave induced or, in certain cases, due to earthquake forces. The dynamic nature of the horizontal forces and the large dimensions of the structures make undrained analyses a major tool for evaluation of foundation behaviour both in clay and fine-grained frictional soils. All analyses must, however, consider the deteriorating effects on the soil shear strength of repeated loading and the susceptibility of near-surface soil to scour.

In principle, large gravity structures depend on their weight to resist horizontal loads. The ratio of the horizontal to the vertical load is so large that the sliding mode of failure needs separate consideration. In particular, this failure mode is important for soil profiles with soft or loose strata overlaying stiff or dense soils or interbedded between two stronger layers, a common situation in the North Sea. This has led to the use of 'skirts' in order to transmit the loads through weaker top layers. The detrimental effects of scour and hydraulic instability at the edge of the platform base are also minimized by the presence of skirts.

For most platform designs, the voids remaining after completed penetration of the skirts between the platform structure and the seabed are normally filled as part of the installation phase, see 4.6. Until that operation is completed the dynamic response of the soil-structure is largely influenced by the entrapped water in the skirt compartments. Due to the rocking motions of the platform during wave loading, fluctuating loads are generated which are balanced mainly by water pressure variations in the skirt compartments. These excess water pressure variations generate, in turn, hydraulic gradients in the foundation soil which have to be kept within certain limits in order not to introduce piping failure at the perimeter of the platform, see 4.4.1.

During the operational phase the stability calculations have to verify that the platform has an acceptable level of safety under the most critical loading conditions considering all possible modes of failure. The most important are:

Failure due to sliding along base or skirt tips;
Deep seated shear failure.

In addition, combined failure modes, e.g., comprising sliding at the base slab and local failure at skirt tips have to be checked.

A number of typical failure modes are illustrated in Figure 3.1.

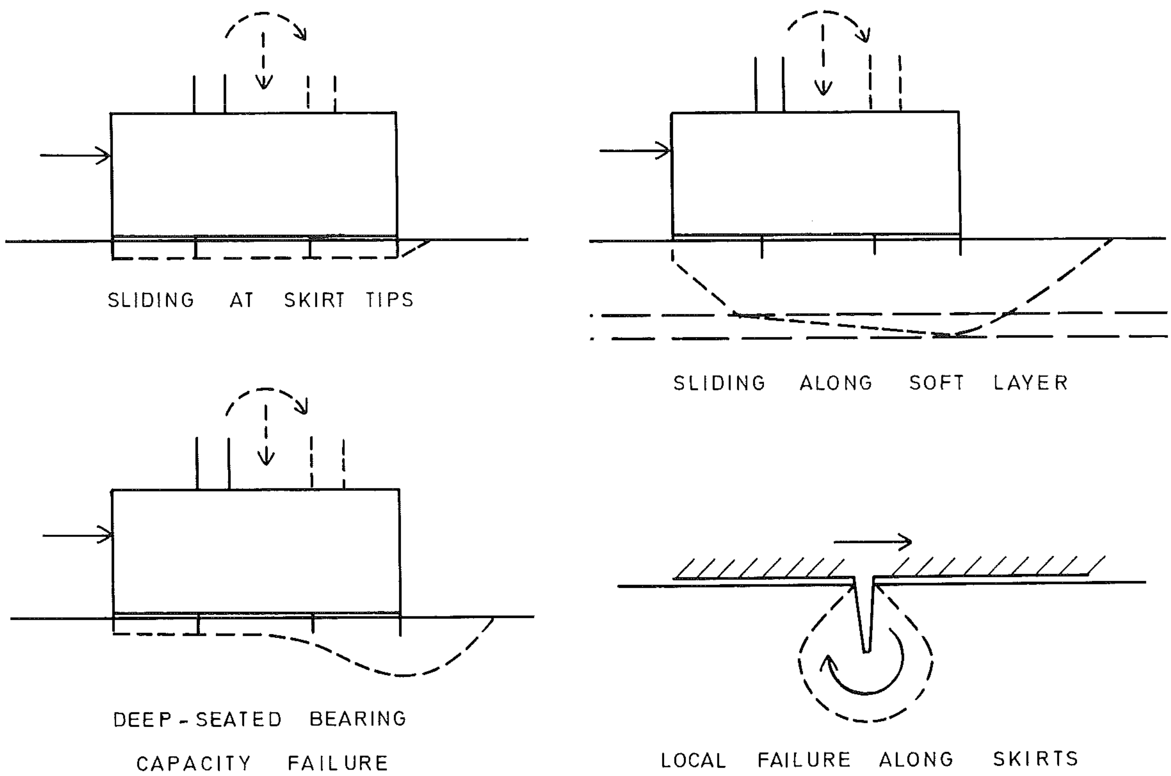


Figure 3.1: Examples of failure modes.

For structures founded on two or more separate footings the safety against overturning also has to be investigated. In the case of structures on a single footing bearing capacity failure under the edge of the foundation will take place prior to overturning.

If the hydraulic gradients along the perimeter of the platform are excessive under dynamic motion there is a risk of piping or internal erosion under the structure. This eventuality has to be studied separately and it may be desirable to establish criteria for the exit gradient. Such criteria may replace the normal requirement to a compressive contact stress at all times during operation. Figure 3.2 gives an example on how skirts and an inverted gravel filter may be used to reduce the exist gradient.

For structures on a sloping or even on a flat seafloor, the risk of seafloor failure must also be considered. Such failures are most common in loose deposits of fine sand and coarse silt (Bjerrum, 1971), or very soft clays (Henkel, 1970) and may be triggered by seabed pressures caused by waves.

3.3.2 Analytical methods

The stability analysis of a foundation exposed to a combination of horizontal and vertical loads is, in principle, a stability problem of the same kind as analyses of slope failure, bearing capacity of foundations or earth pressure. These are among the classical geotechnical analysis problems and are normally solved by means of limiting equilibrium methods. The plastic equilibrium (moment and forces) is checked for the body limited by a possible sliding surface, such as that indicated in Figure 3.1, and the most unfavourable sliding surface is sought.

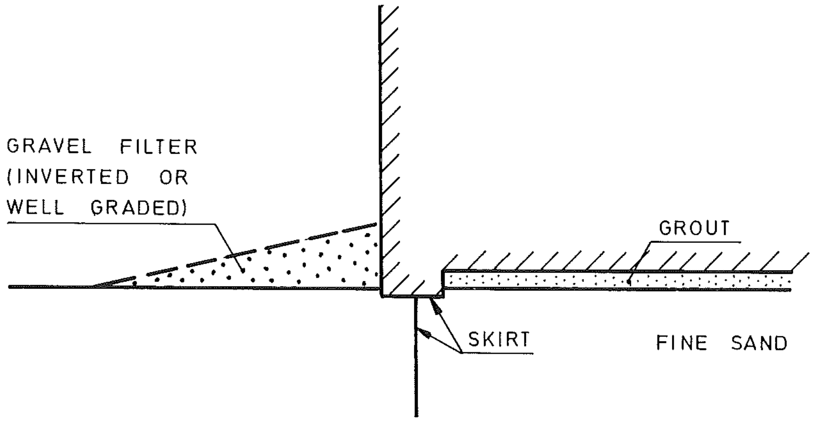


Figure 3.2: Use of filter to reduce exit gradient.

The Bishop method of slices (Bishop, 1955), described in standard text books on soil mechanics, is one of the methods which are best suited for this purpose. Some of the most advanced applications of this method have been published by Janbu (1954; 1973) and by Morgenstern and Price (1965).

Two problems of particular significance to such analyses should be mentioned. The first is the end effects. The analysis normally assumes an infinitely long strip foundation. For a limited length of foundation, a certain shear force on the ends of the sliding body may be assumed. These forces will tend to improve the stability of short footings on cohesive soils, compared to infinitely long footings. For frictional soils, the capacity of short footings may, however, be smaller than that of a long footing. For certain loading conditions, this effect may be accounted for in an approximate way by using bearing capacity formulae, (see below).

A second consideration concerns the kinematics of a failure. A failure mode, such as the deep-seated bearing capacity failure shown in Figure 3.1, is only kinematically possible when a simultaneous failure of the structure and the soil takes place. The stress distribution at the interface is highly uncertain under these conditions. In practice, the problem is solved by assuming either a linear distribution of stress or a constant stress on a part of the foundation only, see Figure 3.3.

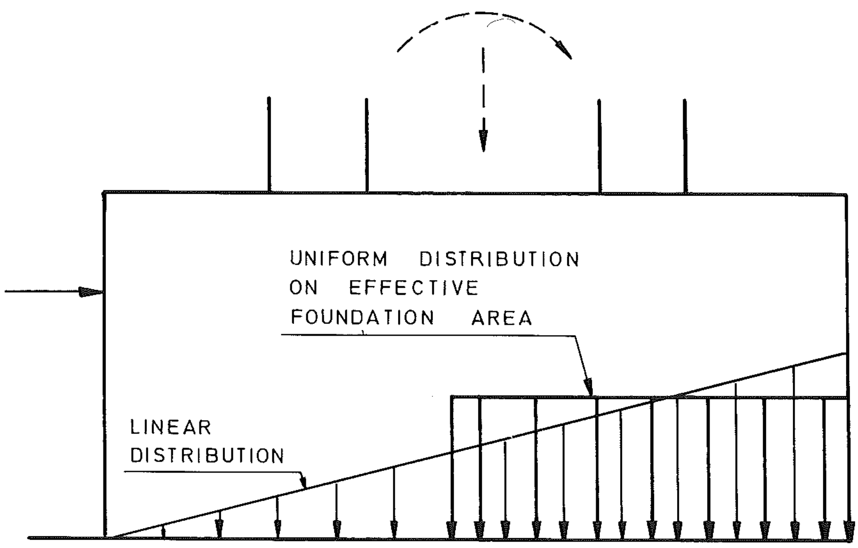


Figure 3.3: Simplified stress distributions.

For homogeneous soil conditions, bearing capacity formulae may be used in lieu of complete stability analysis. The most commonly used methods are those published by Brinch Hansen, (1970), and Meyerhof, (1963). For strongly inclined loads, these formulae should be used with caution, as they are mainly based on small scale tests. The problems related to undrained analysis of foundations on sand have been studied by Hansen (1976).

A critical examination of the procedure currently in use for stability analysis of gravity structures on clay and equipped with penetrating skirts was given by Lauritzen and Schjetne, (1976). The stability of gravity structures is also treated by Yong, Kraft and Focht, (1976) and by Murff and Miller, (1977).

Lauritzen and Schjetne, (1976) outlined the basic assumptions related to bearing capacity formulae. Since similar assumptions have to be made in other stability calculations as well, it may be worthwhile to summarize them here.

Irrespective of the actual foundation shape, the foundation area is transformed to a square;

Moment equilibrium is satisfied by applying the vertical load at the base concentrically on the 'effective foundation area', see Figure 3.3.

From the assumption of full contact between the structure and the subsoil over the whole platform base, it follows that a horizontal failure plane cannot develop unless the failure has developed under the whole platform area;

The part of the horizontal load assumed to act simultaneously with the vertical load on the 'effective area' is found by subtracting the shear force taken by the area outside the effective area from the total horizontal load at the base. All the calculated forces that involve the shear strength of soil are assumed to have the same degree of strength mobilization, i.e., a constant mobilized material factor.

The calculated level of safety is considerably influenced by the assumptions made regarding the horizontal resistance offered by skirts and side areas of the sliding body of soil, in addition to that mobilized along the critical failure surface.

The reduction factor accounting for the effect of load inclination in bearing capacity formulae demonstrates a dramatic decrease in vertical load carrying capacity, as the ratio of horizontal to vertical load increases. Since all foundation soils are more or less non-homogeneous, the use of methods allowing an arbitrary choice of the shape and location of the failure surface will usually lead to more reliable estimates of the foundation stability. For this reason, bearing capacity formulae are not recommended for final design of foundations with a high ratio of horizontal to vertical loading. In such cases one should revert to the method of slices.

Local failure at the skirts, as illustrated in Figure 3.1, may normally be analysed by using a bearing capacity formula for the horizontal loading. In a fully developed failure, such as illustrated in the Figure, the resistance of the skirt will be twice as high as for a footing on the surface. The actual resistance is expected to be less than this, e.g., 1.3 to 1.6 times that of a surface footing.

.3.3 Soil properties

All calculations have to be based on actual site data composed of results from field and laboratory investigations. The field investigations, mainly cone penetration tests, are an invaluable supplement to the laboratory investigations in the assessment of the actual in situ shear strength. The sample disturbance due to sampling, handling, storage, etc. leads to laboratory shear strength values which are normally smaller than those of the corresponding element in situ. Certain 'macro-effects' in situ, e.g., in slickensided soils, are exceptions to this rule. For such soils the 'macro shear strength' in situ may be lower than the laboratory shear strength determined on a disturbed sample. Therefore, all possible care has to be demonstrated to assure a minimum of mechanical disturbance to the sample. There is a promising development of push sampling techniques for offshore use presently under development, see 2.4.1, which will bring forward the possibility of recovering offshore soil samples of onshore quality, which is not the general case today.

The approximate shear strength determinations carried out on board the vessel, i.e., fall cone and pocket penetrometer tests, are part of the soil classification testing and the scatter is normally very large. The stability calculations require high quality triaxial or simple shear test results, where the sample prior to shearing has been subjected to a stress history, bringing it back to the actual field stress conditions. Since the wave loading is of short duration, it is assumed that shearing under undrained conditions will yield the most relevant shear strength values for clays. It is recommended that the sliding body be divided in different parts and representative stress-path tests be performed for each part of the sliding body. This involves use of both active and passive triaxial tests, as well as simple shear tests.

The correct determination of the shear strength of clay remains one of the most important problems in the foundation design. Some of the principal problems include the effects of soil structure, (e.g. fissures), progressive failure and anisotropy. These and other problems in relation to stiff clays are extensively discussed by Skempton, (1977), and Burland, (1977), and, in relation to soft clays, by Bjerrum, (1973b).

The undrained (dynamic) strength of saturated sand depends largely on sand porosity, i.e., whether the sand dilates or contracts when sheared. A contracting sand may loose its strength completely (liquefy) while the effective stress path of a dilating sand moves up along the failure line without reaching a definite state of failure. This additional strength mobilization is, however, normally accompanied by large strains, and it is therefore not advisable to make use of strength accompanied by negative pore pressures. This philosphy is followed by Hansen, (1976), when he defines an available shear strength dependent on the initial state of effective stresses. The resulting calculation method is closely related to one assuming undrained failure. In the North Sea most sands are dense to very dense and thus, dilating. Hansen's approach is therefore applicable to these sites.

The static and dynamic strength of sand is also affected by the fabric of the sand as reported by Oda, (1972 a, b and c), and Mulilis, Seed, Chan, Mitchell and Arulanandan, (1977). Since most testing on sand is carried out on reconstituted samples, the sample preparation procedure will be very important for the recorded behaviour of the sand and the conclusions made with respect to the in situ behaviour.

An important aspect of the foundation stability calculations is the effect of repeated loading on the shear strength. Based on examination of the behaviour of the foundation structure and the mechanism of repeated loading, along with results from carefully designed and conducted cyclic laboratory tests, it is possible to estimate the deteriorating effects of repeated shear stress applications on shear strength of the soil. This correction of the static shear strength is to be included in the design shear strength. The problems related to repeated loading effects are further discussed in section 3.8.

3.4 Long-term deformations

By long-term deformations are meant initial and time-dependent settlements and permanent horizontal displacements. The major experience from gravity structures so far is from the heavily overconsolidated clays and dense sands encountered in the North Sea.

3.4.1 Problems

The most important problems related to long-term settlements are total settlement, differential settlement, and the rate of settlement.

Large total settlements may reduce the air-gap between the wave crest and the platform deck below acceptable margins, and may also introduce additional stresses in conductors due to negative skin friction. In order to minimize such stresses, the conductors should not be fixed to the platform by group clamping or other means. When assessing this effect, the possibility of permanent horizontal displacements should also be considered.

Differential settlements and tilt may not be compatable with requirements to the platform and the process equipment. Tilt may be caused by non-uniform soil conditions or loading.

The rate of settlement and consolidation influences the safety of the foundation, as the safety may be less before consolidation is complete than after. The rate is also important for the installation of conductors and equipment, since only the settlements taking place after such installations will normally affect these. Settlements or lateral displacements have less influence on the design of pipelines and risers than the governing factors for these components, which are normally the temperature stresses and elongations, or the stress conditions imposed during pipeline installation.

3.4.2 Analytical methods

The analytical methods used for calculations of settlements for gravity platforms are generally the same as those used for other types of structures with shallow foundations.

One method is to assume the soil to be a homogeneous or layered elastic medium. Formulae for settlements are then readily available, (e.g. Poulos and Davis, 1974) and the soil parameters required are Young's modulus and Poisson's ratio.

Another method is to calculate the stress distribution assuming the soil to be an elastic medium, and then to assess a compressibility for each layer. The compressibility may be defined as a tangent modulus M derived from the stress-strain curve of laboratory tested soil samples. The formation for M proposed by Janbu, (1963), has been widely applied:

$$M = mp_a \left(\frac{p'}{p_a}\right) 1\text{-}a$$

where m is a modulus number, a, a stress exponent and p' the actual effective stress level. The atmospheric pressure p_a is included to make the formula dimensionally correct. Three problem groups may be defined on the basis of the value of a

a = 0   normally consolidated clays

a = 0.5 sand

a = 1.0 overconsolidated clay and rock.

The compressibility of the different soil layers may also be determined from empirical relationships based on in situ tests such as the cone penetration test.

The time-settlement relationships derived on the basis of Terzaghi's relatively simple case of a one-dimensional consolidation will normally predict rates of settlement which are too small. Important departures from the assumptions related to Terzaghi's theory of consolidation arise from the soil containing permeable layers or seams and local sand pockets allowing for lateral drainage. The large dimensions of the gravity platforms will, on the other hand, lead to ratios between the width of the loaded area and the thickness of the compressible layer which might justify the use of a cone-dimensional consolidation theory were the soil homogeneous.

The methods for settlement analysis outlined above may seem crude, but bearing in mind the uncertainties involved in selecting material properties they are still considered adequate. More refined methods, such as finite element methods, are at present not expected to improve the overall result.

At present, no reliable analysis method is available to predict the effect of repeated loading on settlements. The best analyses are therefore based on empirical evidence from other types of structures subject to variable loads, see e.g., Bjerrum, (1964). Rowe, (1975), has demonstrated how settlements during a single storm may be assessed from centrifuge model tests.

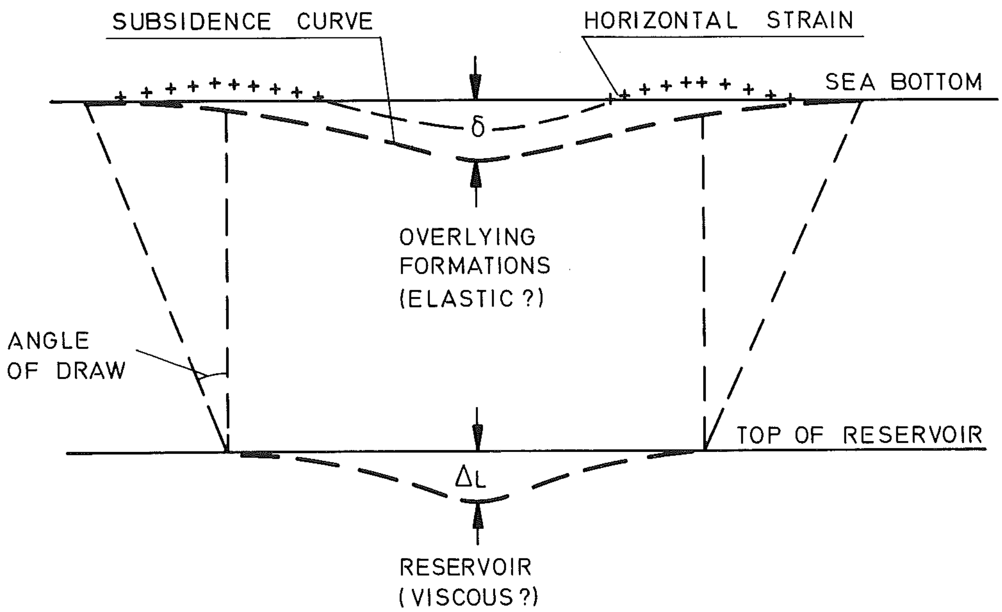


Figure 3.4: Sketch of subsidence due to assumed drop in fluid (or gas) pressure in the producing reservoir.

The subsidence of the sea bottom, (Figure 3.4), due to possible pressure reduction in the producing reservoir has to be estimated by the petroleum engineers and added to the calculated settlements. This contribution to settlements will affect the air gap but is probably less important for the stresses in the structure. As indicated in Figure 3.4, the prediction of subsidence requires a knowledge of the load-deformation characteristics of the reservoir rock and the overlying formations as well as boundary conditions. The most fruitful approaches seem to be those assuming a more or less elastic top mantel over a viscous lower layer.

As an estimation of the difference in settlement likely to take place over the foundation area requires a knowledge of the variation in the foundation deposits across the site, and information regarding the preferred wind and wave directions. Due to wave action, rocking motions are induced which tend to soften the soil more along the periphery of the foundation area than at the centre. Variations in shear strength along the perimeter of the foundation area may concentrate the effects of repeated loading to a certain part of the foundation area, thus leading to pronounced differential settlements.

Methods for calculation of long-term lateral displacements are not well developed, and the best prospects are probably in basing such methods on empirical evidence. Experiences so far are that these considerations are not decisive for the design of conductors, etc., but that consideration of the problem is required.

3.4.3 Soil properties

For calculation of consolidation settlements of foundations on clay, the preconsolidation pressure p_c' is the property of principal concern, since the compressibility is many times higher above p_c' than below.

The preconsolidation pressure may be roughly evaluated by combining information from:

Carefully conducted laboratory tests on representative samples;

Empirical correlations between p_c' and the undrained shear strength;

Geologic survey (regional and local);

Settlement records from adjacent structures.

Laboratory determinations of the preconsolidation pressure is largely affected by the sample disturbance due to sampling, transportation and time elapsed before the sample is mounted in the oedometer. Additional uncertainties are related to the testing technique and the final graphical determination of the preconsolidation pressure based on measured load-settlement curves.

In the North Sea the loads have generally been well below p_c' and the problems described above have been avoided. For certain oil fields, however, the evaluation of p_c' may become a decisive factor.

A determination of the preconsolidation pressure in sand is even more difficult since the stress history is completely destroyed by sampling. If this knowledge is important one has to rely on geological surveys, observation of adjacent structures and in situ testing.

Table 3.3. Parameters for settlement calculation in clay.

NC	δ_i	$E_u = 250 - 1000\ s_u$	$\nu = 0.5$
	$\delta_i + \delta_c$	$M = mp'$	$m = 10.25$
OC	δ_i	$E_u = 250\text{-}500\ s_u$	$\nu = 0.5$
	$\delta_i + \delta_c$	$E_d = 100\text{-}150\ s_u$	$\nu = 0.1$

In Table 3.3 are listed the moduli in use for prediction of foundation settlements in clay. A distinction has been made between initial settlements (δ_c) in normally consolidated (NC) and overconsolidated (OC) clays, respectively.

For normally consolidated clays, the calculation of initial settlements has been based on an undrained Young's modulus E_u which is often taken as equal to 250 to 1000 times the undrained shear strength of the clay along with Poisson's ratio $\nu = 0.5$. Since the initial settlements take place during the installation phase and normally before the settlement measurements have started, there is little evidence on the applicability of this procedure. Consolidation settlements including the initial settlements may be predicted using tangent moduli in the range 10 to 25 times the effective over-burden pressure, p', as shown in Table 3.3. The modulus number increases with increasing stiffness of the clay.

For overconsolidated clays the undrained Young's modulus E_u associated with initial settlement predictions may be taken equal to 250 to 500 times the undrained shear strength of the clay with $\nu = 0.5$, see Table 3.3. The primary consolidation settlements including the initial settlements are normally calculated on the basis of a drained Young's modulus E_d' with $\nu = 0.1$. The current practice involves use of drained Young's moduli as low as 100 to 150 times the undrained shear strength.

These values are supported by Butler, (1974), who summarized methods used for settlement analysis of structures founded on heavily overconsolidated clays. Based on case histories of buildings on London clays and Gault clay he found by calculation an equivalent Young's modulus equal to 130 times the undrained shear strength when $\nu = 0.1$ was used.

As mentioned previously in the section, the effects of repeated loading on settlements have normally been estimated on an empirical basis. For a typical North Sea platform, founded on clay, settlements averaging 1-2 cm/year and totalling 50-100% of the initial plus primary consolidation settlements have been proposed.

The compressibility of sand will largely depend on the relative density which may be estimated from cone penetration tests, and driving resistance during sampling. For the dense sands encountered on sites for gravity structures in the North Sea, it is normally suggested to use m = 400-600 in the expression $M = m\sqrt{p' p_a}$ for $p' < p_c'$. For pressures $p' < p_c'$ the modulus number, m, may be 50 to 100% larger. The corresponding m-values proposed for loose sands are also found in Table 3.4.

Table 3.4. Parameters for settlement calculation in sand.

for $p' < p_c'$	$M = m\sqrt{p' p_a}$	Loose	$m = 100\text{-}200$ $\alpha = 1\text{-}2$
	$E_d = \alpha q_c$	Dense	$m = 400\text{-}800$ $\alpha = 3\text{-}4$
for $p' < p_c'$	$M' = \beta M$	Loose	$\beta = \eta = 3\text{-}5$
	$E_d = \eta E_d$	Dense.	$\beta = \eta = 1.5\text{-}2$

For an estimation of the drained Young's modulus from the recorded cone penetration resistance q_c, the coefficients α given in Table 3.4 indicate the magnitude and range of these empirical coefficients for different stress levels and sand densities. Review of the settlement calculation methods based on the cone penetration test have been reported, e.g., by Sanglerat, (1972), Dahlberg, (1975) and Mitchell and Gardner, (1975).

The effect of repeated loading is estimated in a similar way for sands as previously described for clay.

Settlement observations are currently being carried out on most of the gravity platforms installed in the North Sea, see Table 4.1.

None of the structures has suffered critical settlements. As an example, the Ekofisk tank settled 26 cm from the installation in 1973 to summer 1974 and an additional 2 cm in the next

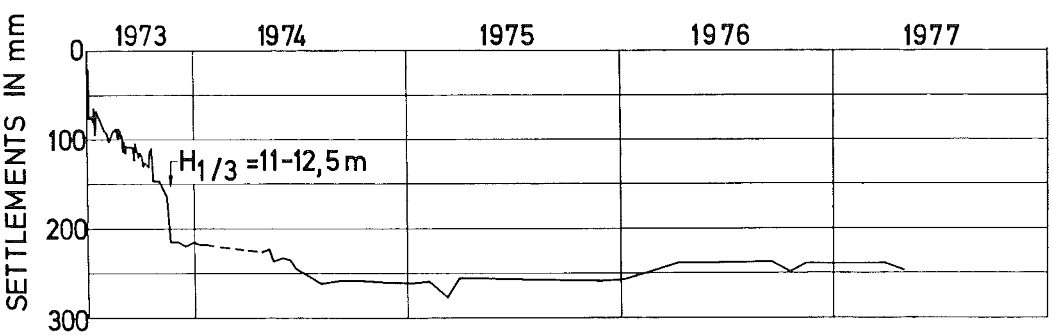


Figure 3.5: Observed settlements of the Ekofisk tank.

three years, see Figure 3.5. A recent re-evaluation and updating of the settlement records revealed that the curve given by Rødren, Foss and Furnes, (1977), is slightly incorrect from summer 1974 to the end of 1975. Within a year, it is expected that the settlement observations going on will provide the information required for more accurate settlement predictions.

Finally it should be emphasized that the assessment of realistic values on deformation moduli is a difficult task which involves several considerations besides those mentioned herein, e.g., scale effects when extrapolating from moderate to large dimension structures.

3.5 Dynamic motions

3.5.1 Problems

The principal purpose for calculating dynamic motions in the foundation soil is to account for oil-structure interaction in the dynamic analysis of the platform. This analysis shall provide information regarding:

The response to extreme wave loading;

The response at frequencies near resonance;

The long-term distribution of response needed for fatigue analysis of the structure;

Deformation loads on external connections such as pipeline risers and conductors;

The effect of repeated loading of the foundation soil.

In cases where earthquake loading may be applicable, the dynamic analysis is of course essential for analysis of the response to such loading, but this is outside the scope of the present report. Reference is made to Watt, Boaz and Ruhl, (1978).

On the geotechnical side, the principal problems in the dynamic analysis are to represent the dynamic stiffness of the foundation, including the non-linear behaviour of soil, and to assess the foundation damping.

The problems related to the effects of repeated loadings will be considered in section 3.8.

3.5.2 Analytical methods

The study of dynamic foundation stiffness has concentrated on one of the two following principal methods:

Continuum methods, (e.g. the elastic half-space method)

Finite element methods.

In the first case the soil is normally modelled as an elastic half-space, and the stiffness and damping properties are modelled by a system of equivalent springs, dashpots and added mass, see, e.g., Richart, Hall and Woods, (1970). In the second case the soil layers are modelled by a finite element model, which for economical reasons is normally two-dimensional. Special elements are often applied at the boundaries in order to represent the energy radiation from the foundation.

A technique based on this approach was reported by Waas, (1972) and Lysmer and Waas, (1972), later generalized by Kausel, Roësset and Waas, (1975).

There is still considerable argument as to which method is most suitable for various applications. Seed, Lysmer and Hwang, (1975), and Hadjian, (1976) review the advantages and draw-backs of both methods and arrive at opposite conclusions.

For offshore gravity structures, there are a few special considerations which appear to make the continuum methods the best choice. First, the embedment of the foundations is normally small compared to the width, thus avoiding some of the principal objections to continuum methods. Second, it is difficult to obtain reliable data for the soil parameters to be used in the analysis. The accuracy of the result is therefore governed more by the quality of the soil data than by the refinement of the analysis method.

Due to the fact that the foundation soil is usually highly layered a straightforward application of the elastic half-space theory may be rather uncertain. Since the frequency of loading is very low the static stiffness of the foundation is a most important factor.

A special effort should therefore be made to calculate the static stiffness in terms of equivalent spring constants. In order to account for non-linearities, this should be done for several loading conditions covering a range from small to extreme loads.

A static two-dimensional finite element analysis is considered to be a suitable tool for this purpose. The analyses will allow the various soil layers to be properly modelled and can also realistically account for the effect of skirts penetrating through soft upper layers. The non-linear soil properties may be introduced in one of the following two ways:

Use of an equivalent secant shear modulus compatible with the strain level in each element. Several iterations are required in order to obtain an acceptable accuracy.

Use of non-linear or bi-linear stress-strain curves for each element.

For a further study of the analysis of soil-structure interaction for offshore gravity structures, reference can be made to Whitman, (1976), Bell, Hansteen, Larsen and Smith, (1976) and Watt, Boaz and Dowrick, (1976).

.5.3 Soil properties

As for other types of analysis, the parameters used for calculations should preferably be taken from laboratory or field investigations. Due to the problems of sample disturbance which affect deformation properties even more than strength, current practice is to use empirical guidelines as a supplement to the test results. Normally, such guidelines give considerably higher stiffnesses than laboratory tests.

These two sets of data may be used to define a range for the stiffness to be used in a parameter study which is essential in the dynamic analysis. On stiff clays, careful in situ tests tend to support the high modulus values of the empirical guidelines.

The laboratory tests most commonly used for investigation of the dynamic stress-strain properties are cyclic triaxial and direct shear tests. These give data for the secant shear modulus G, whereas data concerning internal damping are not always of good quality. These tests tend to give stiffnesses on the low side. In the small strain range ($< 0.1\%$ shear strain) other testing methods, such as torsional shear or resonant column tests would be more appropriate, but these methods have not yet found practical use for offshore structures. When separate cyclic laboratory tests are carried out specially for the project under consideration, high quality samples must be used and tested under representative stresses.

Empirical guidelines for selection of stiffness and damping properties for sand have been summarized by Richart, Hall and Woods, (1970, p. 152), and for clay and sand by Seed and Idriss, (1970), Hardin and Drnevich, (1972 a and b), and Richart (1977).

The non-linear soil properties also introduce an internal damping in the soil. This is hysteretic in nature and thus frequency independent. For small strains, an internal damping of 2-5% may be assumed, but this increases substantially with increasing strains (see Seed and Idriss, (1970)).

Another more important type of damping is the geometrical or radiation damping which is viscous in nature, i.e., frequency-dependent, and accounts for the energy lost due to wave propagation through the soil away from the platform. The expressions for the radiation damping resulting from the half-space theory will often overestimate the actual damping, thus leading to unconservative results. In order to account for possible reflections from boundaries between layers and other uncertainties, Whitman, (1976), recommends that 50% of the theoretical radiation damping for an elastic half-space is used.

On seven of the gravity structures presently installed in the North Sea, instrumentation is provided for monitoring of dynamic motions. The results of these observations are expected to add considerably to our knowledge regarding soil properties to be used for dynamic analyses, in particular as regards verification of the high shear moduli resulting from the empirical guidelines mentioned above.

3.6 Penetration resistance

3.6.1 Problems

With the aim of improving foundation stability and to serve as a means for scour protections, skirts are often used both along the perimeter of the foundation and under the central part of the platform. The skirts may be of steel and/or concrete steel skirts having thicknesses between 20 and 30 mm and the width of concrete skirts ranging between 0.3 and 1.5 m (see Figure 3.6).

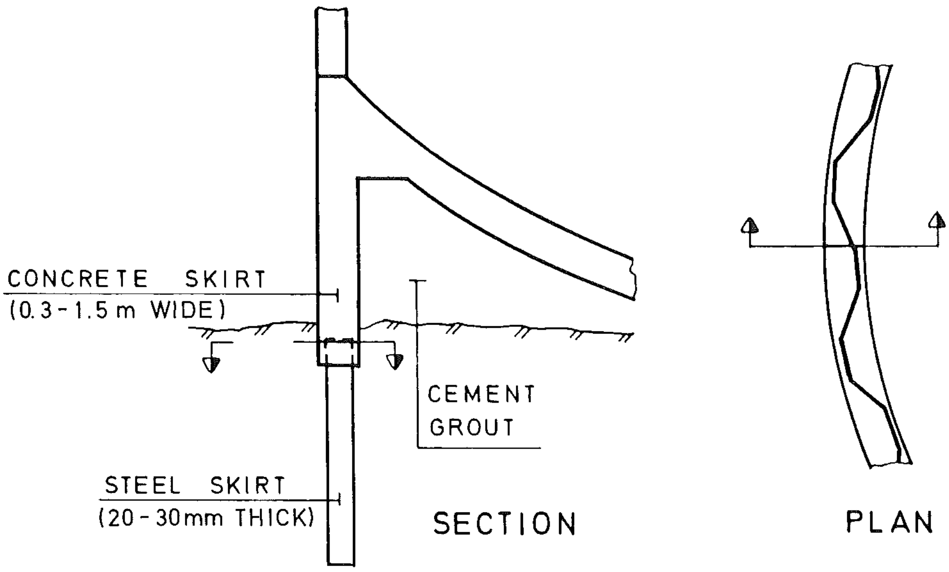


Figure 3.6: Example of skirt design.

In order to facilitate accurate positioning and to minimize horizontal loads on the skirts, dowels projecting a few metres below the skirts are frequently used. These dowels are often hollow pipes of large diameter, 1 to 3 m, and wall thickness 10-50 mm.

The calculation of the penetration resistance of skirts, dowels and other projecting elements is generally based on methods similar to those used to calculate the load-carrying capacity of piles or on conventional bearing capacity theories. It should, however, be emphasized that the calculation of penetration resistance of skirts aims at the best possible prediction of the 'true' penetration resistance in order to be able to fulfil certain specified design requirements.

An under-estimation of the penetration resistance may have serious economical consequences, or invalidate the whole design, due to insufficient sliding resistance at skirt tip level. By over-estimating the penetration resistance, unwanted contact stresses may develop between the base structure and the foundation soil.

The tolerances in the estimates of penetration resistance of skirts are thus dictated by the acceptable deviation from a 'true' penetration resistance. This is in contrast to pile capacity predictions, where tolerances in the prediction is only in the direction of higher bearing capacity than predicted.

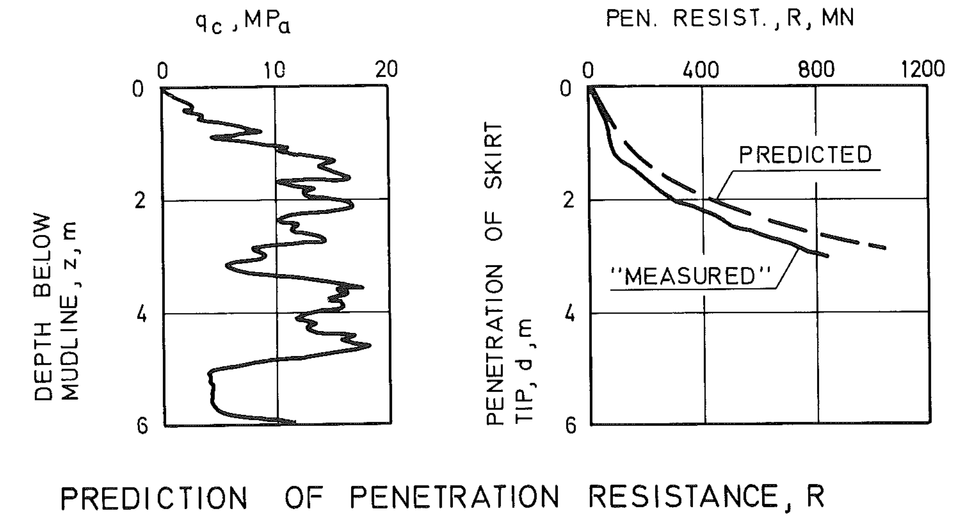
With due consideration to the effects of non-uniform soil conditions, sloping and uneven sea bottom and other reasons for non-uniform penetration resistance, the penetration force available must turn out to be sufficient to drive the skirts to the minimum depth dictated by stability criteria. On the other hand, too large depth of penetration may generate unwanted contact stresses between the base structure and the seafloor.

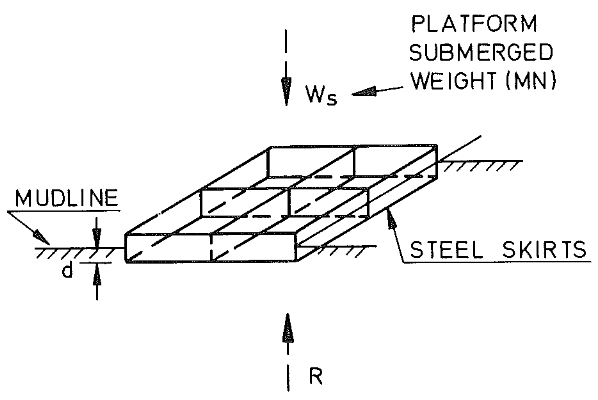
.6.2 Analytical methods

Many different theories and empirical relationships have been adopted to predict the penetration resistance of dowels and skirts, all suffering from the same drawback of not having been calibrated against results from full-scale measurements when they first were applied. Based on the experiences gained so far from the installation of concrete gravity structures in the Northern North Sea, some predictions turned out to compare fairly well with the measured penetration resistance. The success with a certain calculation method depends, however, to a large extent on the quality of the site and laboratory investigations and the interpretation of the results from these investigations, i.e., the assessment of characteristic soil properties.

The most fruitful approaches to calculation of the penetration resistance of steel skirts and dowels are those based on cone penetration test results supported by the result of relevant laboratory tests. The main advantage of the cone penetration test is that a continuous depth profile of the penetration resistance is obtained. The conversion from cone resistance to skirt penetration resistance involves, however, several uncertain assumptions regarding the effects of shape, different rate of penetration, excess pore pressures during cone penetration testing, etc. Still it is probable that the cone penetration test is the best field test presently available for this purpose.

The penetration resistance of dowels and steel skirts is calculated as the sum of the accumulated skin resistance and the point resistance. If the dowels are open-ended pipes, the skin resistance is composed on inner and outer friction, the inner frictional resistance being limited by the point resistance corresponding to the total cross-section of the dowel. Open pipes are therefore calculated for two cases, with or without inner plug.





FORMULA:

$$R = k_p(d) A_p \bar{q}_c(d) + A_s \int_0^d k_f(z) \bar{q}_c(z)\, dz$$

where

k_p, k_f = EMPIRICAL COEFFICIENTS (see 6.2)

A_p = SKIRT TIP AREA (m^2)

A_s = SKIRT SIDE AREA, OUTER + INNER (m/m^2)

Figure 3.7: Approach for prediction of penetration resistance of steel skirts and dowels.

When the results of cone penetration tests are applied directly to calculation of the penetration resistance R of steel skirts (or dowels), the cone resistance, q_c, from a representative number of tests is used to establish an average q_c -profile, $\bar{q}_c$, see Figure 3.7. The corresponding profile for for unit skin resistance of the skirt is then obtained by multiplying the $\bar{q}_c$ -values by an empirical coefficient, k_f, which, for North Sea conditions experienced so far, ranges between 0.03 and 0.05 in very stiff silty clay and between 0.001 and 0.003 in dense to very dense silty fine sand. A depth profile for the unit point resistance of the skirts may be established by multiplying the average $\bar{q}_c$ -values by another empirical coefficient, k_p, which for the North Sea conditions ranges between 0.4 and 0.6 in the same clay and between 0.3 and 0.6 in the sand formations experienced. This approach is further outlined in a guideline to DnV Rules, (1977).

Due to the water escaping below the tips of the external skirts just before the skirts start penetrating the seafloor, the top soil tends to scour to a depth of a few tenths of a metre. The trench thus obtained will of course reduce the penetration resistance which has to be considered in the analysis, in particular for concrete skirts. Due to lateral movement of the platform during 'touch down' the depth of the trench may be more pronounced. This may be considered in the analysis by reducing the above mentioned coefficients by 25 to 50% in the upper 1 to 1.5 m of foundation soil.

In sand, the influence of personal judgement on the predictions is substantial, due to the uncertainties related both to the methods of analysis and the assessment of characteristic soil properties. One of the uncertainties is how to correct for the difference in rate of penetration between the cone and the skirt. The normal rate of penetration of the cone, 2 cm/s, may be several hundred times as large as that of the skirts. Since the pore water pressures generated at the level of the tip is a function of the rate of penetration and the relative density of the soil, the cone penetration resistance in a very dense sand will be much higher at a rate of 2 cm/s than if the rate of penetration is 0.005 cm/s to the generation of negative pore pressures, see Schmertmann, (1974).

When extending the North Sea experience to other areas and sites, the calculation of the unit skin resistance of steel skirts and dowels should consider the original stress conditions in the soil, notably the coefficient of lateral earth pressure at rest,and the relative density of the soil. The increase in the lateral stresses due to the penetration of the low-displacement steel elements is negligible for very loose sand but may be significant for dense to very dense sand. By combining the experience gathered from pile measurements (Meyerhof, 1976) and actual site investigation data, preferably cone penetration test results, fairly good estimates of the skin resistance of skirts and dowels can now be made (see also 4.4.2).

The point resistance of closed-end or plugged dowels may be calculated based on conventional bearing capacity formulae for clay and sand.

The penetration resistance of wide concrete skirts is preferably estimated on the basis of conventional bearing capacity formulae. In order that a sufficiently high estimate for the resistance shall be obtained, formulations giving high bearing capacity factors, e.g., Caquot and Kerisel, (1953), should be used. In a few cases, plate loading tests have been carried out with the aim of determining the angle of shearing resistance and sandy soils and thus increasing the accuracy of the predictions.

With respect to the uncertainties in the choice of soil parameters, etc., it is recommended to calculate a 'highest expected' and 'most probable' penetration resistance. This is achieved by using a range of penetration resistance coefficients as indicated above. The required eccentric ballasting capacity should also be estimated, taking into account the effects of variations in soil deposits across the site, sloping sea bottom, etc., (see also 4.4.2).

3.6.3 Soil properties

The soil information of major importance for the calculation of penetration resistance of skirts and dowels at:

Layer sequence including lateral variations;

Undrained shear strength or cone penetration resistance of clay layers;

Undrained shear strength of the clay layers;

Relative density of cone penetration resistance of sand layers;

Average slope of sea bottom;

Bottom topography;

Lateral distribution and thickness of erodable top layer;

Initial stress conditions, (e.g. overconsolidation ratio).

The expected rate of platform submersion at 'touch down' and the permeability of the principal layers will also constitute useful information. The above information should be collected through adequate site investigations such as grab sampling, shallow borings, cone penetration tests, plate loading tests, TV-survey, bottom slope and topography assessment and relevant laboratory investigations on recovered samples, (see Section 2).

3.7 Reaction forces on base structure

3.7.1 Problems

The reaction forces of the soil have to be considered in the design of the base structure. The structural members of the base which come into contact with the soil are the dowels, the skirts and the base slab or domes.

The dowels and steel skirts must be able to resist the bending moments and the axial forces encountered during installation. In order to keep the bending moments or lateral pressures within acceptable limits, certain restrictions to wave height and period are defined for the installation phase, see 4.3. Lateral loads on dowels and skirts are mainly due to horizontal environmental forces. In addition, a lateral pressure is produced by the excess water pressures in the skirt compartments caused by overturning moments, and this pressure also has to be accounted for.

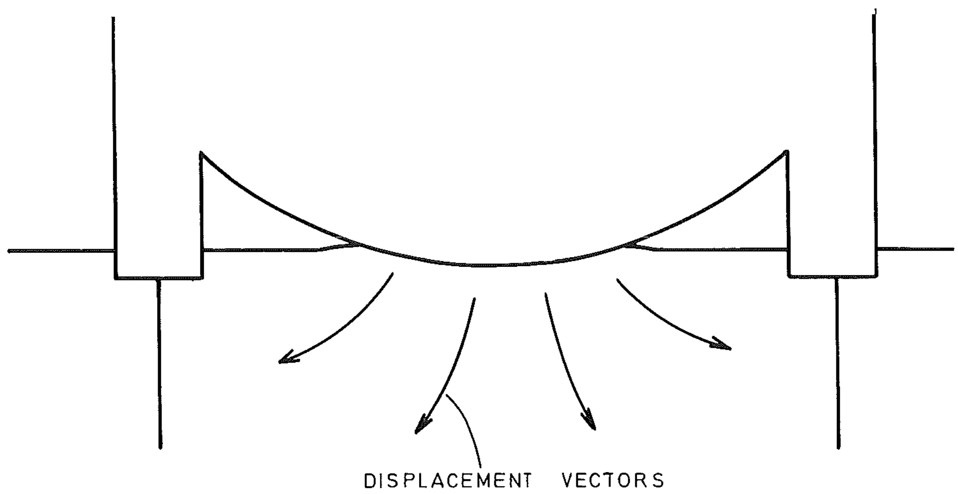


Figure 3.8: Lateral loads on skirts due to dome penetration.

Additional lateral loads on skirts may be introduced if the base structure is dome-shaped (see Figure 3.8), or because of geometrical imperfections giving non-axial penetration of the skirts. The presence of boulders may have the same effect.

The unevenness of the seabed and the shape of the base structure may lead to the development of high local contact pressures on the base of a gravity structure. With the extremely dense and stiff soil formations encountered in the North Sea such loads may, in cases, govern the design of the base structure.

The choice of design loads for the base structure may be based on one of the following three principles:

The base structure is designed for the most unfavourable loads compatible with the soil conditions and seabed topography;

Instrumentation is provided to monitor the loads on the base slab during installation, and the process is stopped when a pre-set design load is approached;

The configuration of the base is such that the base slab will not get in contact with the seabed unless conditions are very non-uniform. Local contact stresses are thus avoided. This condition may be achieved, e.g., by providing wide and high concrete skirts.

The two first principles have been used in practice for North Sea structures, and the third is being applied on Statfjord B, see also 4.5.

In all cases it must be borne in mind that the local reaction pressures considered in this report are the effective soil pressures. In addition, there is the load caused by hydrostatic water pressure.

3.7.2 Analytical methods

The evaluation of the lateral pressures against dowel during installation may be carried out as described in the literature on the analysis of laterally loaded piles. Normally, full passive earth pressure is assumed and the problem is reduced to the determination of the effective width of the dowel. Usually, this is taken as one to three times the actual width, depending on the depth of penetration.

The axial forces on the dowel correspond to the penetration resistance, see 3.6.

During the first part of the installation, skirts are designed to resist the limiting passive earth pressure, which is a function of the horizontal touch-down speed (rate of loading), type of soil, and depth of skirt penetration. Based on model tests and dynamic analyses, a limiting depth of penetration is determined as a function of the environmental load related to various sea states with due considerations to the load contributions from wind, current and tugs. At penetrations exceeding the limiting depth, the lateral loads on dowels and/or skirts correspond to the environmental forces. This is one of the reasons why a limiting sea-state has to be specified for the installation, (see 4.3).

The magnitude and the distribution of the contact stresses on the base slab or domes are highly dependent on the stress-strain-strength properties of the soil and their local variations, the degree of unevenness of the seabed and the geometry of the base of the structure.

The unevenness of the seabed assumed in the design should be chosen in accordance with the results of a sea-bottom survey, taking the accuracy of the survey method and the density of the topography measurements over the foundation site into account. The influence of significant seabed formations and obstructions of different kinds are to be considered as well.

During installation the compression of high points on the seafloor will first introduce only minor stresses suggesting that an elastic analysis of the soil will be acceptable. Later in the process, yielding in the soil may be expected, and a plastic analysis based on a bearing capacity theory will be more appropriate. Both types of analysis have been used previously and have demonstrated that, for given soil conditions, the size of the contact area determines which conditions give the highest loads.

The non-linear stress-strain behaviour of soils indicates that a non-linear elastoplastic analysis will serve as a better approximation for the calculation of contact stresses. An analysis method based on this principle has been included as a guideline in the DnV Rules, (1977). The method assumes a modified hyperbolic stress-strain curve and takes into account the gradual increase in the size of the contact area. The principal parameters in the analysis are the equivalent modulus of elasticity for small strains and the ultimate bearing capacity.

In order to obtain conservative, i.e., high values of the reaction stresses, the use of high values for the shear strength and bearing capacity factors is recommended, (see 3.6.2).

Current research indicates that actual reaction stresses during installation may be reasonably accurately predicted by methods such as those described above. Measurements on full-scale structures during installation in the North Sea indicate that the very high local contact stresses predicted by early analysis must still be considered as realistic.

3.7.3 Soil properties

Soil properties of primary importance for the analysis of local contact stresses are:

Shear strength properties, i.e., angle of shearing resistance and cohesion or undrained shear strength;

Elasticity properties, i.e., Young's modulus and Poisson's ratio.

The characteristic values of shear strength and stiffness of the soil must be chosen to give upper limit estimates of the contact pressure.

For sands, the most representative description of the shear strength and deformation properties is obtained from in situ tests such as cone penetration tests. For the interpretation of the results from such tests, reference is made to relevant literature on the topic, e.g., Sanglerat, (1972), Dahlberg, (1975), and Mitchell and Gardner, (1975). Based on current onshore experience, the constrained modulus of sand for loadings beyond the preconsolidation pressure, i.e., for $p' < p'_c$, may be two to four times the average cone penetration resistance within the depth influenced by the loading. The drained Young's modulus for initial loading may then be estimated to vary between 2 to 4.5 times the average cone penetration resistance if Poisson's ratio is taken equal to about 0.3. The larger value is representative for dense sand. For loads within the preconsolidation range, substantially larger values have to be assumed, see Dahlberg, (1975). However, by relating the stiffness to the cone penetration resistance the ratio of the preconsolidation modulus and the modulus applying to virgin loading will be limited to about 5 for loose sand and about 1.5 for dense sand, see Table 3.4.

For clays, the initial Young's modulus of both normally and overconsolidated clays may be taken equal to 250-500 times the undrained shear strength, determined on the basis of laboratory and in situ tests.

.8 Effects of repeated loading

.8.1 Problems

Repeated loadings due to wave action affect soil parameters required for many of the analyses described in previous sections. There are also special problems related to the behaviour of soil in cyclic loading not mentioned previously. The problem is therefore considered in full in this chapter.

Starting with the references to previous parts of this section, it was pointed out in 3.3 that repeated loading may reduce the undrained shear strength, in 3.4 that such loading may increase the long-term settlements and in 3.5 that it may reduce the dynamic stiffness of the soil. The most important special problem is the risk of failure in cyclic loading, such as the widely discussed phenomenon of liquefaction of sand. Another special problem is the redistribution of base contact stresses.

The analysis of all these problems is based on an understanding of the basic mechanisms governing the behaviour of a soil element subjected to repeated loading. These mechanisms will govern the effects on stiffness and undrained shear strength after cyclic loading as well as the risk for total failure in cyclic loading. It should be realized at the outset that repeated loading effects is a problem of equal concern to sand and clay. For North Sea gravity platforms, the soil conditions decide where the emphasis is for a particular project.

In the following the basic mechanisms governing the behaviour of a soil element are first considered and then the analysis methods used to extrapolate from the behaviour of small elements to a bulk soil volume are discussed.

.8.2 Basic mechanism of soil behaviour

The behaviour of soil during cyclic loading is governed by the effective stress σ, which is defined as the average stress transmitted in grain to grain contact. The total stress σ applied to soil element is carried partly as effective stress and partly as pore water pressure u, i.e., $\sigma = \sigma' + u$.

To explain the mechanism, let us consider an isotropically consolidated soil element (Point 0 in Figure 3.9), subjected to cyclic shear stresses, with an equivalent constant amplitude as defined by the distance of point 1 to the σ'-axis. If the cyclic loads are applied without allowing the pore water to escape (undrained loading), the imposed cyclic loads have to be balanced by internal water pressure variations in the soil element. For a single load cycle the effective stress will be almost unaffected by the loading, but if the shear stress level exceeds a certain threshold value, each load cycle will generate a small increment of residual pore water pressure which adds to that already accumulated in previous load cycles. As a result of the development of residual pore water pressures, the effective normal stress will decrease and displace to the left in the diagram.

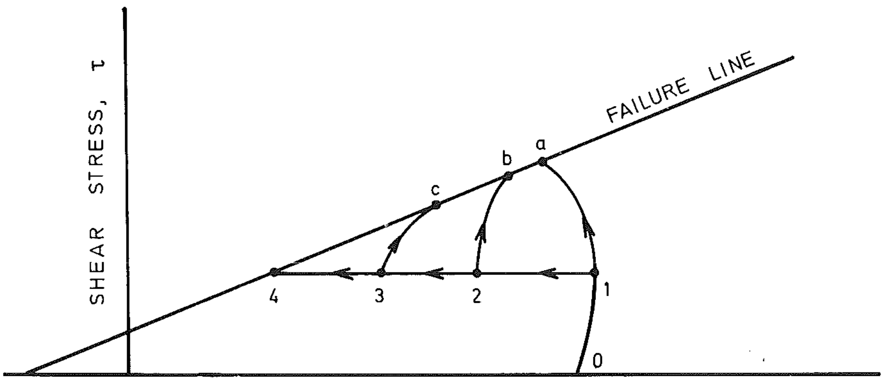


Figure 3.9: Effects of repeated loading on soil behaviour.

The effective stress path may stop before it reaches the failure line (point 2) if the shear stress level is moderate or the number of cycles is small. A static shear test carried out from point 2 follows an effective stress path which terminates in point b. If the preceding cyclic loading is assumed to express the effect of a design storm, stress path 2-b illustrates the quasi-static failure mode. The undrained shear strength after cyclic loading (point b) is somewhat lower than the static shear strength (point a) of a non-cycled soil element. Under favourable loading conditions, a certain soil element may, however, accumulate pore water pressures so large that the effective stress path touches the failure line (point 4). Before reaching this state the shear strain starts to increase rapidly.

From this condition two things may happen. The soil may lose its strength completely and behave like a heavy liquid. It can no longer carry the constant amplitude shear stress. This corresponds to a movement down to the left on the failure line. In this condition, indefinite strains may take place which would represent a true liquefaction failure such as may be observed in a loose sand. The other alternative is that the soil still has an additional strength which may be mobilized in a static test following the cyclic loading. This is representative for a dense sand or a stiff overconsolidated clay and corresponds to a stress path moving up to the right on the failure line. The additional shear strength available beyond the cyclic shear stress amplitude previously applied is mobilized after a definite but large strain (say 5-20%), thus halting the deformation. This phenomenon has been termed 'cyclic mobility' (Casagrande, (1971)). The term 'failure in cyclic loading' may be used to cover both failure modes, irrespective of whether we consider sand or clay.

The number of cycles to failure will depend primarily on the soil type and the stress conditions. For sand in particular, the testing procedure, including the sample preparation procedure, will in addition greatly affect the results, often leading to over-conservative results.

At present it must be admitted that reliable testing procedures for all types of sand are not available. There is a trend in the United States to make more use of empirical data based on the behaviour of sand deposits during earthquakes.

Experiences with North Sea platforms indicate that liquefaction of sand is not a serious problem for the dense sands found there. Pore pressure build-up during storms has been recorded, however, pointing to the need for concern. A comprehensive review of the state-of-the-art regarding liquefaction and cyclic mobility of sand was presented by Seed, (1976).

For clay, understanding of the effects of cyclic loading was greatly enhanced by an industry-sponsored research project carried out in 1974-75. Some of the results have been published by Andersen, (1976), Andersen, Brown, Foss, Pool and Rosenbrand, (1976) and Brown, Andersen and McElvaney, (1977). This project revealed much of the basic information and information parameters referred to in this chapter.

As already mentioned, the undrained shear strength of a clay subjected to cyclic loading (points b, c, Figure 3.9) may be found to be less than for a non-cycled sample (point a). The degree of strength reduction has been found to depend on the strain reached in cyclic loading and the applied number of load cycles. For cyclic (or permanent) shear strains less than 3%, generated by less than 1000 load cycles, the reduction may be up to 25%. This effect must be considered in all stability calculations where relevant. The determination and introduction of this effect in a quasi-static stability analysis and a cyclic loading type of failure analysis, respectively, are described elsewhere, see e.g., Foss, Dahlberg and Kvalstad, (1978).

The deteriorating effects of repeated loading on the soil properties are also demonstrated by the significant decrease of the shear modulus with increasing shear strain. Also the damping ratio is affected by the shear strains. This effect is primarily of concern for large strains. In cases where the cyclic shear strain is less than about 0.1% the effect is normally insignificant.

Another effect which has to be considered in connection with repeated loading is that of previous repeated loading on the cyclic behaviour of the soil in a later storm. In this case it is assumed that any excess pore pressures are drained out between the storms.

The present experience indicates that overconsolidated clays at low consolidation stresses may be less resistant to cylic loading after preshearing, while for normally consolidated clays tested at higher consolidation stresses, preshearing is favourable. For sand a moderate amount of preshearing is favourable, while large strains during preshearing may possibly have the opposite effect.

The analysis of cumulative damage is rather different for soils than for other materials, such as steel or concrete. As pointed out above, the long term effect of repeated loading may very well be beneficial rather than contributing to an eventual fatigue failure. The problem to consider is, therefore, the short-term effects in a single storm, during which undrained conditions are normally assumed.

A rational study of this problem appears to have been done only for clays, where reference is made to Andersen, (1976) and Andersen, Hansteen, Hoeg and Prevost, (1977). Possibly a similar method will be applicable to sand. Andersen's method may be used to calculate the cyclic strain at any stage of a random loading process from a knowledge of the stress-strain curve of the soil during the first loading cycle, and the results of a number of cyclic tests with constant cyclic stress ranges, covering a range of stress conditions. The method is based on the assumption that the accumulated cyclic strain at a certain stage of the loading process is one of the principal parameters governing the future development. Good results have been obtained with the method for a number of test cases, but the procedure is still under development. A recent survey of the effects of cyclic loading on clay strength was presented by Lee and Focht, (1976).

.8.3 Analytical methods

The most commonly adopted procedure for analysis of the bulk volume of the foundation is to assume representative average stress conditions which are modelled by means of small laboratory specimens. As long as stress and strain conditions are reasonably uniform, this simple method is expected to give satisfactory results, and it has been accepted for most of the gravity structures in the North Sea.

In cases where the distribution of stresses and strains is highly non-uniform, a more sophisticated method is desirable. A few such methods, based on the finite element method, have been proposed. The first was reported by Bonin, Deleuil and Zaleski–Zamenhof, (1976). This method uses a two-dimensional finite element model of the foundation soil. The soil elements are given non-linear stress-strain properties according to a procedure developed by Kulhaw, Duncan and Seed, (1969). The tangent modulus and Poisson's ratio may then be corrected at each stress increment on the basis of the behaviour of the soil in the first cycle after change (increase) of the stress level. The cumulative effect of a storm may then be analysed by a stress-path method which incorporates the application of initial stresses to the soil elements, the placement of the platform and the storm loading sequence.

Two other methods have been proposed by Andersen, Hansteen, Hoeg and Prevost, (1977). One of the methods is a direct extension and mathematical formulation of Andersen's method for calculation of cumulative damage. The other method is based on an elasto-plastic material model tailored to the results of cyclic triaxial and direct simple shear tests.

The last of the models referred to above may also be used to calculate permanent vertical displacements and changes in base contact stress due to repeated loading. These phenomena are associated with the permanent deformations resulting from a combination of permanent and cyclic shear stresses acting in the same direction.

The finite element methods mentioned above must at present be considered as an interesting start of a new development

3.9 Centrifuge model tests

Traditionally, model testing has been used extensively in geotechnical engineering. The most common method is the plate bearing test for determination of the bearing capacity and settlements of footings. Usually such tests are carried out in situ, thus testing the foundation soil with no assumptions or approximations other than uniform soil conditions. The tests are normally carried out as static tests, but in recent years cyclic testing has also been successfully done.

Model testing in a centrifuge, see, e.g., Rowe, (1975), was introduced some years ago and is an interesting alternative to other model tests. The principal advantage over 1 g tests is that the stress conditions in a deep soil deposit may not be correctly modelled. The test may thus be used to elucidate the physical mechanisms of complicated soil problems, or problems of soil-structure interaction. For example, critical failure modes may thus be revealed.

Within the present state-of-the-art, models of offshore gravity platforms may be built to a scale of about 1:100. Consolidation, static and cyclic undrained loading may be applied. Instrumentation may include stresses, displacements, pore pressures, etc.

For offshore structures the model has to be built from a soil similar to that on the actual site, since sufficiently large samples cannot be obtained from the site. Any errors involved in site investigations and interpretation of soil conditions are thus reflected in the model. A second circumstance is that the time scale for the model is the square of the linear scale. For a linear scale of 1:100 the time scale is thus 1:10 000. This is normally not a problem for clay, but for sand it may be difficult to maintain undrained conditions during the test.

Centrifuge model tests should be considered a useful tool in identifying mechanisms and behaviour, in particular when studying new types of structures. Theoretical methods are expected to remain the principal tool of analysis, but may be confirmed and sometimes extended by centrifuge tests.

3.10 Summary

This review covers specification of the design problems related to the foundation design of gravity structures, assessment of available analysis tools and the knowledge regarding soil properties needed in the analyses.

The limit state method is now being accepted for foundation design of concrete structures. The most important checks are on stability in the ultimate limit state and on settlements in the serviceability state. In both cases the effects of repeated loading have to be accounted for. With respect to foundation stability, the risk of failure in cyclic loading has to be checked independently, in addition to the conventional quasi-static stability analysis.

The principal stability problems are shallow sliding and deep seated shear failure. The method of slices is recommended for the quasi-static stability analysis. The principal problems are related to the correct determination of the undrained shear strength of the soil, sand as well as clay. The shear strength should preferably be obtained according to the stress path method.

The most important long term deformations are related to settlements. The analysis is based on fairly simple methods, and empirical parameters for the compressibility often have to be used. Settlement observations currently being carried out on most of the North Sea gravity structures are expected to increase the confidence in the settlement analysis methods within a short time.

Calculation of dynamic motions is part of the total dynamic analysis. The most important factors related to the foundations are stiffness and, to a smaller extent, damping. These properties may be expressed by equivalent spring constants and dashpots, but static finite element analysis is recommended for investigation of the stiffness. The analysis must take non-linear stress-strain properties into account. Due to the problems of sample disturbance, empirical guidelines are used as a supplement to laboratory test results for determining the soil parameters.

An assessment of the penetration resistance of dowels, steel and concrete skirts is required for a safe installation of gravity structures. The principal remaining problem in this respect is determination of the point and side resistance in sand. Field experience has added considerably to our knowledge in this respect. The results of static cone penetration tests have proved useful for prediction of penetration resistance.

Reaction forces on the base structure due to undulations in seabed topography and local variations in soil conditions may be the governing factors for the design of the base structure. Elastoplastic analysis of stress-deformation relationships combined with soil data from cone penetration tests appear to give results in reasonable agreement with field observations.

The effects of repeated loading influence many types of analysis, e.g., stability, settlement and dynamic displacement. Failure in cyclic loading, such as liquefaction is a separate failure mode and has to be considered as such. The basic mechanisms of repeated loading effects are fairly well understood. For sand, laboratory tests are of questionable value, and supporting field evidence is often needed for design. For clay, laboratory tests are widely used. A promising method has been developed for calculation of cumulative damage effects due to random loading. Current design is mostly based on simple analysis, using the laboratory specimens as representative for average stress conditions. Recent development of finite element methods for more complicated cases appear to be a promising approach.

In addition to the theoretical methods mainly considered above, model tests in the centrifuge may be considered as a valuable supplementary tool, in particular for studying mechanisms and behaviour.

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4. INSTALLATION OF CONCRETE PLATFORMS

4.1 Introduction

The installation of a concrete platform on the seafloor is a demanding task. Neither the structure nor the foundation soil must be damaged nor unfavourably affected by the operation. So far the installations carried out are reported to be successful. Some key information regarding site data, base configurations and installation experience for the platforms installed by 1978 are summarized in Table 4.1.

The different installation phases are discussed under the following headings:

Touch down

Skirt penetration

Base contact

Grouting

Installations through the base slab

The requirement for an installation manual and data acquisition system is also mentioned.

Sea bottom survey has already been described in Section 1.

At some locations, boulders have been registrated on the seafloor. They have been successfully removed by trawling.

4.2 Installation manual

Installation of a concrete gravity platform in deep waters is a challenging foundation job which has to be carried out in a minimum of time under very special conditions. Everything therefore has to be carefully planned and organized beforehand.

The installation manual should contain the following:

All background information needed including structure, ballasting system, instrumentation and site conditions.

Guidelines and instructions for all installation phases, stating the different criteria, alarm limits and contingencies. The different plots which should be kept up to date should be listed.

Specifications for grouting and installations through the base slab.

The installation manual should be accepted beforehand by the client and the certifying authority. It should be used as a guide by the installation crew who consist of all the different specialists. The groups of people on board the platform during installation should be capable and authorized to take any decision.

Some basic data for the 13 concrete structures already installed in the North Sea are given in Table 4.1.

4.3 Touch down

Designers who have equipped their platform with extended steel pipe dowels have so far been successful in preventing horizontal movements of the platform at the time of skirt insertion. However, such base projecting elements may be exposed to high loads, and have to be carefully instrumented to ensure no overstressing. The evaluation of an installation criterion in terms of sea state that can be accepted for installation will normally include the determination of: (i) environmental loads on the platform as a function of the sea state, (ii) structure and soil capacity whichever is a minimum. Item (i) has been determined by model tests and dynamic analysis (Figure 4.1). This approach seems promising, as good agreement between field observations and predictions is obtained. The load contribution from wind, current and tugs has also to be considered.

Table 4.1. Concrete platforms in the North Sea

North Sea Structure				Site data		Base			Grouting	Installations through the base	Experience
Type	**Name**	**Operator**	**Year Installed**	**Water depth m**	**Foundation soil**	**Slab**	**Skirts**	**Dowels**			
Doris	Ekofisk tank	Phillips	1973	70	Dense fine sand	Flat $A = 7400\ m^2$	0.4 m concrete ribs	None	No	Pore pressure probes	Skidding during touch down
Condeep	Beryl A	Mobil	1975	120	Dense find sand over clay	Conical domes $A = 6200\ m^2$	3.0 m steel 0.5 m concrete	3	85% fully ballasted before grouting	Pore pressure probes. Settlement casing Deep drainage wells	High differential skirt penetration resistance Flat seafloor
Condeep	Brent B	Shell	1975	140	Stiff clay interbedded sand	Conical domes $A = 6200\ m^2$	4.0 m steel 0.5 m concrete	3	Fully ballasted before grouting	Pore pressure probes Settlement casing Deep drainage wells	High differential skirt penetration resistance Flat seafloor
Doris	Frigg CDP1	Elf	1976	98	Dense fine sand	Flat, ringshaped $A = 5600\ m^2$	None	None	No	Pore pressure probes. Settlement casing	—
Sea Tank	Frigg TP1	Elf	1976	104	Dense find sand over clay	Flat $A = 5600\ m^2$	2.0 m concrete	None	50% fully ballasted before grouting	Fore pressure probe	High differential skirt penetration resistance Sloping seafloor 1/100
Doris	Frigg-Scotland Manifold	Total	1976	94	Dense find sand	Flat, ringshaped $A = 5600\ m^2$	None	None	No	Not known	—
Condeep	Brent D	Shell	1976	140	Stiff clay interbedded sand	Conical domes $A = 6300\ m^2$	4.5 m steel 0.5 m concrete	3	75% fully ballasted before grouting	Pore pressure probes Settlement casing. Deep drainage wells	Significant differential skirt penetration resistance. Flat seafloor
Condeep	Statfjord A	Mobil	1977	145	Stiff clay	Conical domes $A = 7800\ m^2$	3.0 m steel 0.5 m concrete	3	75% fully ballasted before grouting	Pore pressure probes Settlement casing	Low contact stresses on the domes
Andoc	Dunlin A	Shell	1977	153	Stiff clay interbedded sand	Flat $A = 10\ 600\ m^2$	4.0 m steel	4	35% fully ballasted before grouting	Pore pressure probes	Low differential skirt penetration resistance
Condeep	Frigg TCP2	Elf	1977	102	Dense fine sand over clay	Conical domes $A = 9300\ m^2$	1.2 m steel 0.5 m concrete	3	55% fully ballasted before grouting	Pore pressure probes	High differential skirt penetration resistance Sloping seafloor 1/100
Doris	Ninian	Chevron	1973	136	Stiff clay interbedded sand	Flat $A = 15\ 400\ m^2$	3.75 m steel	None	—	Not known	—
Sea Tank	Brent C	Shell	1978	140	Stiff clay interbedded sand	Flat $A = 10\ 300\ m^2$	3.0 m concrete	None	—	Pore pressure probes	—
Sea Tank	Comorant A	Shell	1978	150	Stiff clay interbedded sand	Flat $A = 9700\ m^2$	3.0 m concrete	None	50% fully ballasted before grouting	Pore pressure probes Settlement casing	—

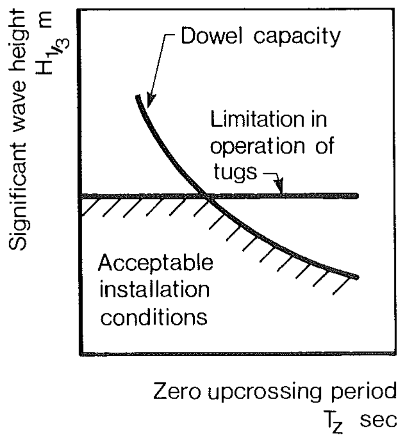

Figure 4.1: Sea state conditions for installation.

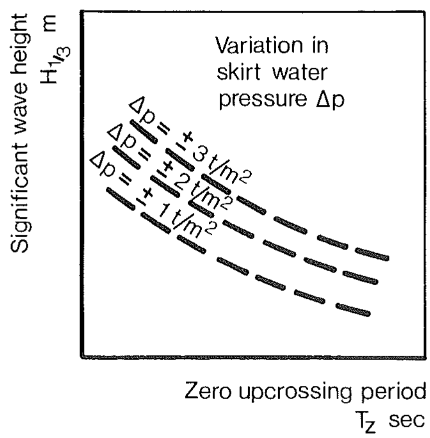

Figure 4.2: Variation in skirt water pressure caused by over-turning moment from sea state.

When determining the utlimate lateral soil resistance against dowels or other base projecting elements, due consideration must be given to the rate of loading, especially when the seafloor consists of dense silt and sand which dilates. The difference in ultimate soil resistance may be dramatic for this material, depending on the rate of deformation. Therefore only correct input values on platform movements at touch down can lead to realistic predictions.

Accelerometer readings during this phase have been carried out on some platforms. The results may vastly increase our knowledge about platform movements.

Experience from installation of structures not provided with dowels or skirts indicate that some horizontal skidding has taken place at the touch down phase.

4.4 Skirt penetration

Items of importance during this phase of the installation are mainly:

Differential skirt water pressures and piping problems.

Necessary ballasting capacity to penetrate the platform skirt system vertically to desired depth, including the case of non-uniform skirt penetration resistance.

Capacity of skirts and soils to resist horizontal loading from sea state.

4.4.1 Differential water pressure across skirts

Experience has shown that piping may occur during the skirt penetration phase. The ballasting system including evacuation of skirt water has therefore to be designed with this in mind. Careful control of the differential water pressure across skirts and regulation of the rate of ballasting accordingly is necessary to limit piping.

During the first stage of skirt penetration the cyclic overturning moments from environmental forces will cause varying water pressure in the skirt compartments as illustrated in Figure 4.2.

In general, however, all efforts should be made to avoid hydraulic instabilities. Piping along the skirts may jeopardize the possibility of applying suction and over-pressure in skirt compartments in order to correct for tilt, and it may also complicate grouting and influence the final stability.

4.4.2 Skirt penetration resistance

The predictions of skirt penetration resistance have to a great extent been based on cone penetration test results, correlated to experience from onshore pile loading tests. For steel skirts in dense sand and stiff clays, the observed total penetration resistance has been of the order 10-40% less than predicted. With all the observations now at hand it will, however, be possible to improve the correlation factors on both point resistance and wall friction for this type of skirt. For concrete skirts the experience so far is much more limited, as only one structure with 2 m deep concrete skirts is installed. In this case, the tip resistance seems to yield a friction angle of approximately 40° in the dense fine sand when utilizing a bearing capacity formula. This friction angle corresponds reasonably well with laboratory test results.

For short concrete skirts with extended steel skirts, the penetration resistance on the concrete skirts has been less than expected, due to erosion taking place at an early stage of skirt penetration. Erosion trenches have been observed from submarine inspections.

Substantial uneven skirt penetration resistance over the platform area has been experienced. This is mainly caused by variation in thickness of dense top sand or gravel layers. The possibility of facing a large eccentricity on skirt penetration resistance has to be evaluated. A great number of cone tests may prove to the best base for this judgement.

The ballasting system must be capable of offsetting this eccentricity for level penetration. A sloping seafloor may also add to the required eccentric ballasting. With a horizontal seafloor, an eccentricity of 10% of the base diameter has been experienced at almost full penetration depth. With the seafloor sloping 1% an eccentricity of 25% of the base diameter has been necessary early in the penetration stage to keep the platform level.

In addition to eccentric ballasting, the tilt of the platform can to a certain extent be adjusted by regulating the pressures within the skirt compartments. Under- and over- pressure can be applied in such a way that it adds to the eccentricity from ballast water. Allowable over- and under-pressure should be clearly stated in the installation manual. Beside the risk of piping, softening of clays may take place when the effective stresses are reduced. Allowable differential water pressure within the skirt compartments should therefore be given with limitations in magnitude and time.

The skirt will continue to penetrate somewhat after the loading is stopped. This is due to relaxation and will be different for different types of soils. The ballasting must therefore be terminated with sufficient margin for this final penetration in order not to overstress the slab or other elements.

4.4.3 Skirt and soil capacity for horizontal loads

The following two items have to be assessed:

Horizontal loads acting on the platform, as a function of sea state (Figure 4.3).

Skirt and soil capacity, to withstand horizontal loads as a function of penetration depth (Figure (Figure 4.4.).

As the skirts penetrate, the resonance frequency is changed and due consideration has to be given to this when assessing the loads. The effect of cyclic loadings must be taken into account for the soil capacity.

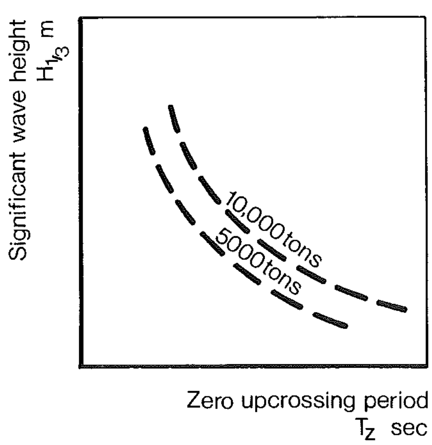

Figure 4.3: Horizontal force acting on the platform as a function of sea state.

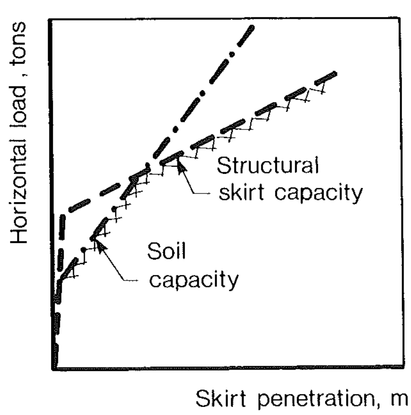

Figure 4.4: Soil and skirt capacity at different penetration depths.

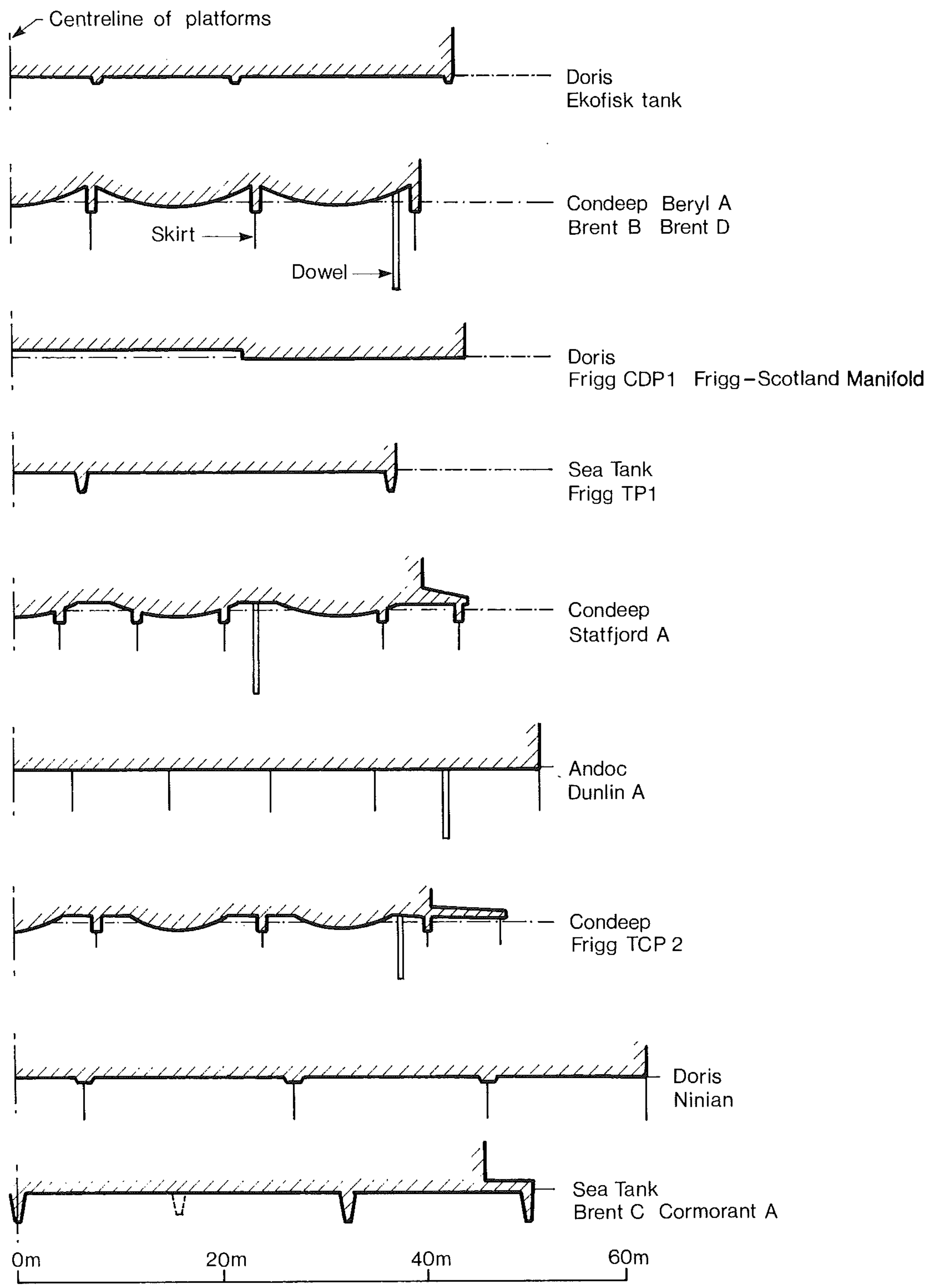

Figure 4.5: Bottom sections of concrete platforms in the North Sea.

4.5 Base contact

The base and skirt configurations for the different platforms are illustrated in Figure 4.5.

Three different design principles seem to be relevant:

The base structure is designed for the most unfavourable loads compatible with soil conditions and seabed topography.

The bottom structure is designed for a certain contact pressure, and earth pressure cells and strain gauges make it possible to ensure that ballasting is terminated before any critical contact pressure develops.

The bottom slab is not intended to contact the seafloor. In this case the distance between the bottom slab and the seafloor has to be measured very accurately.

Installations of structures designed according to the second principle mentioned have so far been successful. The instrumentation for measuring contact pressures and stresses has functioned perfectly on the five Condeep Platforms, and it was possible to stop ballasting at an optimum penetration depth. Reasonably good agreement between observed and predicted contact pressures has also been obtained. With all the observation data at hand it will however be possible to improve the calculation methods.

When applying the third principle mentioned one must be prepared to stop at less ballast-water if the skirt penetration resistance is small compared to the total platform weight. Strong concrete skirts extending somewhat below the slab may however improve the situation.

The base of the structure is very costly and, if the design parameters have to be assessed very conservatively due to lack of information, it may have great consequences for the project. Results of detailed site investigation should therefore be available at an early stage in design.

4.6 Grouting

Grouting of the space between the seafloor and the base of the structure is carried out to:

Keep the platform level;

Avoid piping from water pockets below the structure;

Avoid overstressing of any structural element;

Avoid further skirt penetration;

Secure an even soil reaction force on the base.

The grouting is especially important when the seafloor is uneven or sloping. It requires that the base is equipped with penetrating skirts which divide the base area into separate compartments. Each compartment has sufficient pipes for grouting and to exhaust the entrapped sea water.

The grout-filling processes have, for the latest installed platforms in the North Sea, been based on gravity flow. The grout density has been of the order 12 to 13 kN/m^3.

The grout should fulfil the following requirements:

Strength and compression characteristics not less than those of the supporting soil (28-day compressive strengths have been of the order 500 to 1500 kN/m^2). Bleeding preferably less than 2%.

Low heat emission to avoid high temperature gradient in the concrete structures (less than 20°C).

Time to initial set not less than 6-8 hours.

Low viscosity in order to obtain a high filling rate.

High sweep efficiency in displacing sea water within the compartments. Little mixing of grout and sea water.

As little dry material as possible for cost and storage reasons.

On the evacuation side, two different systems have been applied. In one case the exhaust pipes are terminated and the grout emerges into the sea at a certain elevation, without any possibility to check the quality of the grout at this end. Indication of when the grout is flowing up the exhaust pipe and into the sea is obtained by measuring an increased pressure in the skirt compartment. In addition, the calculated theoretical volume of the compartment should be compared with the amount of injected grout (flow meter).

In the other system, which gives better control of the grouting, the exhaust pipes pass a check point where samples can be taken from time to time, until the quality of the grout is acceptable. Check on skirt pressure and grout volume is included.

Allowable skirt pressure should be given in the platform grouting manual. The criterion is to avoid piping, and skirt pressure may be increased during the grouting process as the seafloor in the compartment is covered with grout. Tilt of the structure due to an eccentric uplift force has to be checked.

Whether full ballasting can be obtained before grouting depends on the tilt of the platform and the stresses in the base. The order in which the different skirt compartments are grouted is determined so as to minimize the tilt. Total grouting time for recent platforms has been of the order of 10 days for a grouting volume of 5000 to 15 000 m^3.

Some information about the different North Sea platforms is given in Table 4.1. So far, grouting experience is good.

Further developments on the grouting technique may be related to the retrieval and possible re-use of the platform at another location. It may be possible to obtain a material which does not adhere to the structure.

4.7 Installations through the base slab

When the platform has been installed and possible grouting is carried out, installations through the base slab will have to be placed. These will include conductor installations, drillings for deep piezometers, deep settlement reference points and possible deep drainage walls in order to improve the foundation conditions.

Conductor tubes with a diameter of 0.76 m are normally installed to a depth of 50 to 80 m below the seafloor. Piezometers are usually installed to a depth of approximately 20 m, settlements reference points to approximately 60 m and drainage wells have been installed to 10-20 m depth.

Installation of conductors which may include as many as 40-50 on one platform will take place over several years. One of the purposes of the 0.76 m diameter conductors is to be able to carry out deeper drilling with mud return to deck level.

Pre-drilling for the conductors is normally carried out with sea water. Some over-pressure is needed to stabilize the hole, but the over-pressure must be kept within certain limits, in order not to cause hydraulic fracturing. It is very important that the installation is carried out in such a way that it does not affect the foundation soil in an unfavourable manner.

The problems encountered may be connected to swelling in stiff, overconsolidated clay which reduces the diameter of the pre-drilled hole, and in sand an oversized hole may develop.

Critical fluid pressures for hydraulic fracturing should be evaluated theoretically on the basis of the soil properties and compared if test results and experiences are available from the actual location. So far, test results and theoretical calculations have been in acceptable agreement.

A safe and efficient way to install conductors is essential. Due to the great height of the drill deck above the sea level, drill cuttings for the 0.76 m diameter conductors have been taken into the drill shaft when drilling is performed inside a shaft. The water level in the shaft has to be controlled to maintain acceptable gradients in the subsoil. It has been shown to be of great importance to measure skirt water pressure during conductor installation.

4.8 Data acquisition system

All observation data should be made available immediately in a visible way to those responsible for the installation. Usually a computer is utilized in the data processing. This makes it possible to present measured values, as well as calculated values, in a convenient form.

Example

Based on experience, the Condeep group has condensed all the information needed to have the full perspective indicated on one sheet of paper, called the Penetration Status.

The content of this Status is as follows:

1. Total weight of ballast water sluiced into the structure after touch down.
2. Total uplift force from excess water pressure in the skirt compartment.
3. Increased tower buoyancy after touch down.
4. Sum of dowel penetration resistances.

5. Skirt penetration resistance (1-2-3-4).
6. Axial force in each dowel
7. Minimum safety factor for the most highly stressed dowel including bending and axial force.
8. Skirt water pressure in the different skirt compartments compared to hydrostatic pressure.
9. Moment and direction of moment from:

    (a) Ballast water
    (b) Skirt water pressure
    (c) Axial forces in dowels
    (d) Skirt penetration resistance

10. Inclination of platform and direction of tilt.
11. Rate of penetration in metres per hour
12. Mean bottom clearance in relation to tip of dowels
13. Bottom clearance at each echo-sounder
14. Draught in relation to the different sea inlets

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5. OPERATIONAL PERFORMANCE OF CONCRETE PLATFORMS

5.1 Introduction

Since 1973 when the foundation configurations of the first Northern North Sea offshore concrete gravity structures, Shell's Brent B and Mobil's Beryl A were decided, 13 of these structures have been built and with the installation of Brent C in June 1978 all are now at their seabed sites. The foundation performance has been monitored by instrumentation for two winters in an industry sponsored project on Brent B, and in less detail on all other platforms for certification purposes.

Operational experience with the foundations of the platforms has been limited to installation of conductors, drilling of a small number of wells and observational foundation performance for at the most five years. Nevertheless, experience has had some influence on the recommendations for future design, although this has been limited by the absence of really testing environmental conditions.

5.2 Conductor installation

All structures are reported to be performing their functions satisfactorily but difficulties in installing the well conductor pipes to the required depth have been experienced. Since all of the structures are placed on generally hard or dense foundation soils, it has been found necessary to install conductors by a combination of drilling a pilot hole and lowering and driving the conductor. Each of the operators has evolved different techniques and, with varying degrees of success, has attempted to minimise the amount of disturbance caused to the foundation soils. Experience of, for example, wash-out which has occurred during conductor installation beneath the drilling shaft of some gravity structures, despite care being taken to avoid it, has emphasized the importance of providing skirts immediately surrounding the conductor areas, in order to restrict the extent of any loss of base support. In one case, a structure, although equipped with a skirt system, suffered sub-base erosion when temporary lowering of the water level in the drilling shaft created a suction pressure beneath the structure which exceeded critical limits.

Careful control of drilling pressures has also been found necessary in order to minimise formation fracture or excessive borehole collapse which, although not particularly hurtful when drilling from a semi-submersible, is unacceptable in the foundation soils immediately beneath a gravity structure.

During drilling operations on a number of gravity platforms, the instrumentation provided for use during platform installation has proved useful in ensuring that design limits were not exceeded, and in the case where sub-base erosion during drilling took place, provided the only means of assessing its extent.

Platform designers should undoubtedly take account of the above factors and most importantly, provide a minimum penetration into virgin foundation soil of an impermeable skirt surrounding the conductor areas. The required penetration should be decided from considerations of the pressures to be resisted, the soil conditions and the overall platform structural skirt system. Platform base and soil foundation instrumentation should be provided and carefully monitored and results should be assessed during conductor installation. Platform base design might assume, for example, that contact with the soil in the conductor areas may either be lost, or alternatively, significantly increased locally by the 'stiff' group of conductors, wells and surrounding grout which will have less settlement than the foundation soil and platform.

5.3 Foundation performance

Assumptions about foundation behaviour made in the original designs are in general being confirmed and knowledge of the fundamental matters of overall platform stability, settlement, soil foundation pore pressure, and dynamic motions is improving with results of the analysis of measurements from instrumentation installed on virtually all gravity platforms. On one platform, instrumentation has predicted design wave mudline displacements, significantly less than those chosen conservatively for pipeline connection design.

Observations have confirmed that pore pressure build-up results from repeated loading of the soil foundation. The magnitude has so far given no cause for concern, being well within conservative design assumptions, and there are indications that, as the soil becomes more consolidated with time, pore pressure build up becomes less.

Settlement records indicate that most of the primary consolidation in clay layers under, for example, the Brent B platform took place in the top 15 m of the seabed and within 10 months. A small secondary settlement is taking place at Brent B but so far no 'shakedown' settlement (rapidly occurring in major storms) has been observed on any platform except the Ekofisk Tank. It is, however, suspected that the 4 cm settlement which occurred in one day in 1973 at Ekofisk may possibly have been as a result of further penetration of the concrete ribs attached to its ungrouted base.

However, forces equivalent to only about 35 to 45% of those predicted for the design condition have so far been experienced on Brent B and deductions made about the foundation behaviour of this and other platforms must necessarily be reserved. The design of critically affected appurtenances such as wellhead flowline, pipeline connection and conductor design therefore should still account for a platform settlement gradually occurring over the first year and an accelerated settlement which may occur over a number of hours. Choice of the magnitudes of these settlements and movements can only be a matter for the individual designer, taking into account the changing state of the art as evidenced by recent measured data from platforms.

The operator's conductor and well design should, in addition to taking account of motions and settlements of the structure and foundation soil which induce displacements and forces in the well, also account for those induced by oil or gas temperature as modified by the foundation.

The design of pipeline connections is being approached in two different ways. Some operators design spool pieces attached at each end with fixed flanges on the caisson or external riser, other make extensive use of pipeline pull-in tunnels in the caisson, whereby excessive pipeline forces are dealt with by assumption that they will cause the line to snake on the seabed.

The cost of conservatism in the prediction of the design parameters is considerable and any improvements arising from recent or future measured data will be of great benefit.

5.4 Use of instrumentation for performance monitoring

While a reduction in certifying authority requirements may occur after a number of years of operation of the early platforms, the basic minimum instrumentation today for assessment of overall foundation safety is settlement, soil pore pressure and caisson motions (acceleration monitoring). Some operators and contractors have added to this significant instrumentation of the base which has proved vital for careful control during installation of the platform and important for measurement of the effects of drilling operators. These factors emphasize the importance of careful consideration at an early stage of the amount and type of instrumentation to be provided. Monitoring systems capable of providing platform personnel with simple read-out in engineering units for comparison with alarm limits have not yet been successfully commissioned for the operators planning this type of control. Settlements are being observed on virtually all presently installed platforms, but in a number of cases the records are incomplete, particularly for the months following platform installation.

5.5 Seabed scour

Out of the 13 platforms installed to date, scour has only been experienced in one case. This was a square platform not provided with scour protection and founded on sand. The platform suffered scour during the first winter on location, whereas a round platform installed on a similar seabed and not provided with scour protection has not been similarly affected. The Christchurch Bay research tower, installed on the south coast of England in shallow water, suffered severe scour on a sand foundation as a result of pumping action at the base exacerbated by its lack of skirts. However no platform founded on a seabed with significant clay layers has yet suffered scour even though not provided with scour protection.

5.6 Operational records

A questionnaire has been circulated to the operating companies on the 13 presently installed concrete structures in the North Sea. A summary of the information provided, relating to purpose, conductor installation experience, foundation performance monitoring, pipeline connections and seabed scour is given in Tables 5.1–5.13.

Table 5.1.

Platform name: Ekofisk Tank	Installed: June 1973	Operator: Phillips Petroleum Company, London.
General data	Platform purpose	Primary: gas process plant, including compression Secondary: oil storage (1 million barrels)
	No. of wells	None
Conductor Installation	Installation method	Not applicable
	Setting depth (ft)	Not applicable
	Number installed (February 1978)	Not applicable
	Experience	Not applicable
Foundation performance monitoring	Measurements	To Autumn '73: settlements by levelling, tilt, pore pressure accelerations. Currently—settlements measured by levelling from adjacent platform
	Logging	in 1973 only
	% Successful recording	
Pipeline connections	Number/sizes	None
	Means of connection	Not applicable
Seabed scour	Seabed soil type	Dense silty fine sand
	Scour protection placed	Plastic filter sheet with rocks on top
	Occurrence of scour	No

Table 5.2.

Platform name: Beryl A	Installed: July 1975	Operator: Mobil North Sea Ltd., London.
General data	Platform purpose	Drilling and production
	No. of wells	40
Conductor installation	Installation method	Drill ahead/lowering sequence
	Setting depth (ft)	±60 metres
	Number installed (June 1978)	39
	Experience	Driving attempted but abandoned in favour of drilling and lower sequences—Major washout beneath base necessitated regrouting.
Foundation performance monitoring	Measurements	Settlement, pore pressures, accelerations, base contact stress and base reinforcement strain (during conductor installation).
	Logging	Manual to October 1977. Magnetic tape since.
	% Successful recording	75%
Pipeline connections	Number/sizes	36 in. and 6 in.
	Means of connection	36 in. hyperbaric weld 6 in. flanged
Seabed scour	Seabed soil type	8-10 m dense silty sand overlying hard clay
	Scour protection placed	Yes
	Occurrence of scour	No

Table 5.3.

Platform name: Brent B — Installed: August 1975 — Operator: Shell UK Exploration and Production. London.

General data	Platform purpose	Drilling and production
	No. of wells	38
Conductor installation	Installation method	Drill ahead/lower sequence with final drive
	Setting depth (ft)	±45 metres
	Number installed (June 1978)	38
	Experience	Change of base support under drilling shafts measured by instrumentation in base which was monitored during conductor installation.
Foundation performance monitoring	Measurements	Pore pressures, base contact stress, settlements, lateral displacement, tilt, dynamic motion. Recording 20 min/3 hours during winter months for 2 years as part of research project.
	Logging	By mini-computer with operator to March '77. Then manual, with automatic system planned.
	% Successful recording	90%
Pipeline connections	Number/sizes	1 No. each of 16, 24, 30 in. and 2 No. 26 in.
	Means of connection	Bolted flanges to external risers with bends in line on seabed out from platform.
Seabed scour	Seabed soil type	Silty fine sand over stiff clay with sand layers.
	Scour protection placed	No
	Occurrence of scour	No

Table 5.4.

Platform name: Frigg CDP1 — Installed: September 1975 — Operator: ELF Aquitaine Norge A/S, Stavanger.

General data	Platform purpose	Drilling and production (Gas)
	No. of wells	24
Conductor installation	Installation method	Drill hole to full depth and lower with driving
	Setting depth (ft)	35-42 metres
	Number installed (June 78)	24
	Experience	Used special mud to hold hole open into which 26 in. diameter conductors were lowered and driven.
Foundation performance monitoring	Measurements	Currently; hydrostatic pressure beneath slab, dynamic motions, settlement by optical levelling. Planned; deep pore pressure, settlement and lateral movement using X-ray markers in borehole
	Logging	Magnetic tape (20 min every 3 hours)
	% Successful recording	(Not known)
Pipeline connections	Number/sizes	2 x 26 in. diameter
	Means of connection	Through pull-in tunnels
Seabed scour	Seabed soil type	Dense sand
	Scour protection placed	No
	Occurrence of scour	No

Table 5.5.

Platform name: Frigg TP1 | Installed: June 1976 | Operator: ELF Aquitaine Norge A/S, Stavanger

General data	Platform purpose	Gas treatment platform
	No. of wells	None
Conductor installation	Installation method	Not applicable
	Setting depth (ft)	Not applicable
	Number installed (February 1978)	Not applicable
	Experience	Not applicable
	Measurements	Accelerations, tilt, settlement (by optical levelling)
Foundation performance monitoring	Logging	Magnetic tape (20 mins every 3 hours)
	% Scessful recording	Not known
Pipline Connections	Number/sizes	32 in., 2 x 26 in., 24 in. (welded), 8 in. and 4 in. (flanged) Future: 32 in. and 24 in. (welded)
	Means of connection	Through pull-in tunnels
Seabed scour	Seabed soil type	Dense sand
	Scour protection placed	No (planned 1978)
	Occurrence of scour	Yes (1½ metres at corners)

Table 5.6.

Platform name: MCPO1 | Installed: June 1976 | Operator: Total Oil Marine Ltd., London.

General data	Platform purpose	Intermediate manifold on Frigg pipeline in block 14/9		
	No. of wells	None		
Conductor installation	Installation method	Not applicable		
	Setting depth (ft)	Not applicable		
	Number installed (February 1978)	Not applicable		
	Experience	Not applicable		
Foundation performance	Measurements	Riser stresses	Water pressure under slab, platform dynamic motions	Platform settlement (differential pressure gauge)
	Logging	Hard copy-teleprinter		
	% Successful recording	80%	15-20%	0%
Pipeline connections	Number/sizes	4 No. sealed atmospheric connections at base of structure with encastré anchor in each riser tunnel		
	Means of connection	Caisson seal connections, (settlement and rocking accounted for in design).		
Seabed scour	Seabed soil type	Coarse sand		
	Scour protection placed	Anti-scour mats		
	Occurrence of scour	Some deposition		

Table 5.7.

Platform name: Brent D	Installed: July 1976	Operator: Shell UK Exploration and Production
General data	Platform purpose	Drilling and production
	No. of wells	48
Conductor installation	Installation method	As Brent B
	Setting depth (ft)	105 metres
	Number installed (June 1978)	24
	Experience	As Brent B
Foundation performance monitoring	Measurements	Settlement, soil pore water pressure, dynamic motions, base contact stress and base reinforcement strain (during conductor installation).
	Logging	
	% Successful recording	90%
Pipeline connections	Number/sizes	1 No. 24 in., 1 No. 20 in.
	Means of connection	As Brent B
Seabed scour	Seabed soil type	Silty fine sand over stiff clay with sand layers
	Scour protection placed	No
	Occurrence of scour	No

Table 5.8.

Platform name: Statfjord A	Installed: May 1977	Operator: Mobil Exploration Norway, Inc.
General data	Platform purpose	Drilling and production
	No. of wells	42
Conductor installation	Installation method	Drill and drive 30 in. conductor (threaded connections)
	Setting depth (ft)	90 metres below seabed
	Number installed (June 78)	None (planned to start 20 June 78)
	Experience	None
Foundation performance monitoring	Measurements	Settlement, pore pressure, dynamic motions, base contact stress and base reinforcement strain.
	Logging	Installation recording system and manual, magnetic tape planned for permanent system.
	% Successful recording	100%
Pipeline connections	Number/sizes	One 36 in. SPM oil line
	Means of connection	Flanged connection
Seabed scour	Seabed soil type	Thin sand (2-10 cm) overlying stiff clay
	Scour protection placed	None
	Occurrence of scour	No

Table 5.9.

Platform name: Dunlin A | Installed: May 1977 | Operator: Shell UK Exploration and Production, London.

General data	Platform purpose	Drilling and production
	No. of wells	48
Conductor installation	Installation method	Drill ahead, lower and drive sequence
	Setting depth (ft)	140 metres
	Number installed (June 78)	7
	Experience	Two conductors met refusal before setting depth reached owing to driving beyond pre-drilled hole.
Foundation performance monitoring	Measurements	Settlement, deep and shallow pore pressure, accelerations
	Logging	Manual for settlement, chart recorder for pore water pressure with automatic data acquisition planned.
	% Successful recording	90%
Pipeline connections	Number/sizes	1 No. 24 in., 1 No. 16 in.
	Means of connection	As Brent B
Seabed scour	Seabed soil type	Dense fine to coarse sand (0-2 m thick) overlying very stiff silty clay
	Scour protection placed	No
	Occurrence of scour	No

Table 5.10.

Platform name: Frigg TCP2 | Installed: June 1977 | Operator: Elf Aquitaine Norge A/S, Stavanger.

General data	Platform purpose	Gas treatment and compression
	No. of wells	None
Conductor installation	Installation method	Not applicable
	Setting depth (ft)	Not applicable
	Number installed (February 1978)	Not applicable
	Experience	Not applicable
Foundation performance monitoring	Measurements	Pore pressure, settlement and horizontal displacement planned: wave pressure, base reinforcement strains, platform dynamic motions
	Logging	– magnetic tape (20 min/3 hours) – settlement by analogue (paper record)
	% Successful recording	Not in operation
Pipeline connections	Number/sizes	1 No. 32 in. diameter, 2 No. 26 in. diameter (welded), 8 in. and 4 in. (flanged). Future: 24 in., 20 in., 18 in., 16 in. (welded).
	Means of connection	Connected through pull-in tunnels
Seabed scour	Seabed soil type	4 m dense fine sand overlying interbedded sand and stiff clay
	Scour protection placed	No
	Occurrence of scour	No

Table 5.11.

Platform name: Ninian Central   Installed: May 1978   Operator: Chevron Petroleum (UK) Ltd., London.

General data	Platform purpose	Drilling production
	No. of wells	42
Conductor Installation	Installation method	Drill and drive
	Setting depth (ft)	Approx. 40 m
	Number installed (June 1978)	None
	Experience	–
Foundation performance	Measurements	Vertical and horizontal settlement, dynamic motions, pore pressure at skirt tips (to be installed).
	Logging	Data acquisition system
	% Successful recording	–
Pipeline connections	Number/sizes	11 risers; 36 in., 24 in., 16 in. connected June 1978.
	Means of connection	Bolted flanges
Seabed scour	Seabed soil type	0.5 m silty clay overlying dense sand and stiff clay layers
	Scour protection placed	No
	Occurrence of scour	–

Table 5.12.

Platform name: Cormorant A   Installed: May 1978   Operator: Shell UK Exploration and Production, London.

General data	Platform purpose	Drilling and production
	No. of wells	36
Conductor installation	Installation method	Drill ahead, then lower and drive
	Setting depth (ft)	Approx. 100 m
	Number installed (June 1978)	None
	Experience	–
Foundation performance monitoring	Measurements	Pore pressure, short and long term settlement, accelerations
	Logging	Manual settlement from date of installation, data acquisition system planned in July 1978
	% Successful recording	–
Pipeline connections	Number/sizes	1 No. 24 in., 1 No. 30 in., 1 No. 36 in.
	Means of connection	As Brent B
Seabed scour	Seabed soil type	Stiff clay
	Scour protection placed	No
	Occurrence of scour	No

Table 5.13.

Platform name: Brent C Installed: June 1978 Operator: Shell UK Exploration and Production, London.

General data	Platform purpose	Drill and production
	No. of wells	48
Conductor installation	Installation method	Drill ahead then lower and drive
	Setting depth (ft)	Approx. 100 m
	Number installed (June 1978)	None
	Experience	–
Foundation performance monitoring	Measurements	Pore pressure, accelerations, settlement
	Logging	Data acquisition system
	% Successful recording	Not yet applicable
Pipeline connections	Number/sizes	1 No. 20 in., 2 No. 24 in., 2 No. 30 in.
	Means of connection	(As Brent B)
Seabed scour	Seabed soil type	Silty fine sand over stiff clay with sand layers
	Scour protection placed	No
	Occurrence of scour	–

5.7 REFERENCES

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