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THE DURABILITY OF PRESTRESSING COMPONENTS IN OFFSHORE CONCRETE STRUCTURES
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The Durability of Prestressing Components in Offshore Concrete Structures

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THE DURABILITY OF PRESTRESSING COMPONENTS IN OFFSHORE CONCRETE STRUCTURES

ABSTRACT

There have been a number of reported failures of land-based post-tensioned concrete structures due to corrosion induced by chlorides. This has led to the need to consider the durability of offshore concrete structures.

This report considers the problems encountered in land-based structures, the different designs of offshore structure, methods of corrosion protection, design codes, grouting specifications, methods of inspecting tendons, and potential consequences of failed tendons.

It is concluded that the earlier structures are likely to be more vulnerable to corrosion since subsequent structures have benefited from the availability of better materials and improved construction methods. Although there have been no problems identified to date, serious corrosion can occur without exhibiting any evidence. Full inspections have not been carried out and there is a need to obtain added assurance that the tendons are in a satisfactory condition. The locations most vulnerable to corrosion are identified, and methods of inspection are discussed.

Recommendations are made on best practice using available technology and topics are identified that require further work.

Keywords:
OFFSHORE  CONCRETE  INSPECTION  GROUTING  PRESTRESSED
DESIGN  CORROSION

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EXECUTIVE SUMMARY

Prestressed concrete structures have been in service in the North Sea since 1973. Early consideration of the long term durability of the post-tensioning tendons concluded that provided the grouting was properly carried out corrosion should not occur. However, experience in recent years of land based structures has indicated that despite best efforts at the time, grouting was not always successful and ducts were found to contain voids, and were sometimes empty. A number of structures were found to have corroded and fractured tendons and two collapsed. The problem was not confined to one country or one type of construction. In 1992 an embargo was placed on new construction of post-tensioned bridges in the United Kingdom. In 1996, this was lifted for all but segmental construction after the industry had developed improved standards that gave greater assurance.

As a result of the experience with land based structures, the Health and Safety Executive contracted Gifford and Partners to review the durability of post-tensioning systems in offshore concrete structures. This review is based on information obtained from discussions with designers, contractors and operators, and a study of publications in the literature.

The different types of design are considered with particular attention to the disposition of the prestressing systems. Offshore structures pose different problems due to their sheer size. However, they have generally been given greater care and attention so that there is a feeling that the construction details and operations have been carried out to better standards than in the smaller land based structures. Quality assurance has been maintained by carrying out fully representative grouting trials beforehand and cutting open the trial ducts to confirm that they are completely filled with grout. Added corrosion protection has been provided by various means including thicker cover, epoxy coating the concrete surface, and steel ducting having thicker walls.

About half of the population of structures in the North Sea were built before 1978. Subsequently there have been improvements to the design, grouting materials and injection methods so that the later structures have been grouted to higher standards. The grout materials used in recent years have been thixotropic having no shrinkage or bleeding under normal conditions.

Due to the lengths of ducts and supply lines from the pumps, high pressures have to be used. This caused grout in early structures to bleed so that water collected at the tops of vertical ducts and the resulting voids had to be filled with fresh grout. Some techniques are used which would not be considered acceptable in land based structures, for example it is not always possible to install vents at spacings of 15m, grout is injected downhill and much higher pumping pressures have to be used.

Grouting specifications of the different national authorities have been reviewed and main aspects of composition, performance and grouting procedures are summarised. There is a move towards using pre-mixed and pre-bagged grouts to avoid the variability in supply of cement and mixing on site. The most modern grouting specification is the Concrete Society Specification published in 1996 which is based on most recent research.

Little work has been done on inspection of prestressing systems in offshore structures. It has been considered that their size is so great that it is not practical but in any case there is no evidence to suggest any potential problems. In contrast, a stage-by-stage inspection method has been developed for land based structures. The most vulnerable locations are identified so that the inspection can be focused and conducted in depth. The final stage is an intrusive inspection of the duct and tendon. The method of stress relief coring can be used to measure values of remnant prestress.

The potential consequences of failed tendons have been considered. The situation is different from land based structures because the tendons do not generally play such a critical role and there is a lower sensitivity to failures. Nevertheless it is possible for failure to occur without obvious signs of distress being apparent beforehand.

It is concluded that the first tranche of structures, constructed before 1978, are more vulnerable to corrosion. Those that were constructed later benefited from having improved grouting materials and procedures so that they are likely to be better protected and less vulnerable. Although there have been no problems to date, serious corrosion can occur without exhibiting any evidence of problems. Full inspections have not been carried out.
and there is a need to obtain added assurance that the post-tensioning systems are in good or satisfactory condition. From considerations of designs of structures it is evident that locations in the post-tensioning systems most at risk are in the vertical ducts in the region of splash zones, at construction joints (where present), and at upper anchorages.

The following recommendations are made:

- design and construction practice currently available in several documents and aimed primarily at land based structures, should be collated and published in a single document,
- effects of cathodic protection on prestressing steel should be reviewed,
- a guidance document on the inspection of post-tensioned concrete should be prepared for use on offshore structures,
- measurement of remnant prestress can be made by stress-relief coring,
- if suitable opportunities arise, such as assessment for extended life or decommissioning, the condition of the prestress system should be investigated,
- encouragement should be given to designers, contractors and operators to publish information about the performance of prestressed concrete in offshore structures.
1. INTRODUCTION

The Health and Safety Executive (HSE) in the United Kingdom has responsibility for safety aspects of offshore structures in British waters. In response to questions currently posed on the durability of post-tensioned concrete structures on land, the Offshore Division of the HSE contracted Gifford and Partners to prepare a report on relevant aspects of post-tensioned offshore structures. The work requires consideration of the different types of post-tensioned structure, review of the offshore design codes, assessment of the likelihood of corrosion of the post-tensioning tendons, and recommendation as to whether special inspection should be carried out.

The experiences of land-based structures have been rather unsatisfactory in that there have been stress-corrosion and, more recently, corrosion failures of steel tendons, some causing collapse and loss of life. Corrosion and stress corrosion were surveyed in relation to concrete offshore structures by Burdekin and Rothwell in a report to the former Department of Energy. They concluded that problems are unlikely to arise during the life of offshore structures provided proper attention is given to selection of materials and best available standards of workmanship but it was possible for failure to occur under certain conditions. They identified areas for further research and recommended that although best specified standards of materials and workmanship are available in various documents, this should all be indicated quite explicitly in the Department’s Guidance Notes.

In response to Burdekin and Rothwell’s report, extra requirements were added to the Guidance Notes; quenched and tempered steels and/or alloy steels with ultimate strengths greater than 1200N/mm² were not to be used for tendons or couplers, any pitting evident to the naked eye should be a cause for rejection of the tendon, electrical plant associated with welding operations should be earthed, independent of reinforcement and tendons, to avoid damage from stray currents, and the maximum time between stressing and grouting should not exceed 21 days.

Partly as a result of the conclusions and recommendations by Burdekin and Rothwell, contractors felt able to cope with the requirements of workmanship using the available expertise. However, in the late 1980’s there were a number of problems reported in highway bridges in the UK and other countries; inspection revealed ducts having voids, and in some cases no grout was present, and corroded tendons were discovered.

In some cases the damage was sufficient to require replacement of entire bridge decks. Two bridges collapsed; Ynys-y-Gwas in South Wales in 1985 with no loss of life and Melle in Belgium in 1992 with the death of a truck driver. As a result of these experiences, the Department of Transport of several countries carried out investigations to determine the state of their post-tensioned bridges and organised research to improve standards. In Belgium, post-tensioned bridges were inspected and it was found that a number were seriously corroded and required replacement. In the UK an embargo was placed on new construction of post-tensioned bridges in 1992, and the industry was invited to prepare improved standards. In response, The Concrete Society developed an improved specification and industry prepared a QA scheme. The embargo was lifted by the Department of Transport in 1996 for all except segmented post-tensioned bridges.

At the time of Burdekin and Rothwell’s report, authorities responsible for land-based structures shared the view that problems were unlikely to occur due to corrosion of the post-tensioning tendons. It was felt that there may have been problems with earlier structures but materials and standards of workmanship had improved. It therefore came as a surprise that there were problems in more recent structures. More importantly, the collapses of Ynys-y-Gwas and Melle bridges both occurred after recent inspections had failed to identify any evidence of corrosion. It was apparent that corrosion of post-tensioning tendons could occur without displaying any external evidence.

This report has been based on information from both land-based and offshore sources. There have also been discussions with designers, constructors and inspectors of offshore structures and their help is gratefully acknowledged.
2. DESIGNS OF PRESTRESSED STRUCTURES

2.1 INTRODUCTION

The discovery of commercial oil and gas fields in the North Sea at the end of the 1960’s led to rapid advances in offshore construction techniques throughout the following decade. For the first time, prestressed concrete was to be used on a massive scale in a harsh maritime environment. To date, 28 concrete platforms have been installed in the North Sea, 26 of which are gravity base structures. The majority of these have been built and operate in the Norwegian sector. Of the 12 that operate in the UK sector, 10 were built during the 1970’s.

Gravity base structures are those that gain lateral and overturning stability from the sea floor under the action of their own weight. Due to wave loading alone, the calculated lateral forces and moments for Frigg CD1 were 690MN and 3x10^8MNm respectively. Such loads are resisted across a wide base area so that bearing pressures are minimised. A system of short steel skirts is also used to resist lateral movement, these being embedded into the surface of the sea floor under the weight of the structure.

Environmental loads are minimised by offering the least possible resistance to wave action, which is most powerful at sea level. A combination of two systems has been used in the North sea. The first uses an outer perforated break-water wall (the Jarlan system), the second by reducing the diameter of the structure at sea level to a minimum.

Gravity base construction allows substantial completion of platform topside facilities in calmer water, before towing out to the installation site. The concrete structures are generally built using the following sequence:

- construction of the base raft in a dry dock or specially prepared basin, including prestressing this portion of the structure
- floating the base raft to a moderate depth of water to a position with good protection from storms and that is easily accessible
- completion of the structure whilst afloat, generally using a system of slipforming
- moving the substructure to deeper water for the installation of the deck and superstructures
- towing to site
- final installation on site, including grouting under the steel skirt foundations.

This method of construction largely determines the design of the structure so that sufficient buoyancy is provided for the construction and tow out stages, and so that later it can be adequately ballasted to ensure overall stability in service.

In the past, the majority of design errors leading to structural damage have been due to overlooking the effects of a construction phase on the rest of the structure. In a few cases this has led to widespread cracking in some structural members. However, structural damage has also been attributed to design numerical errors. In one case, this led to the loss of a substructure when the cell walls ruptured during trials in a fjord.

The types of the concrete gravity based structures may be divided into groups depending on the basic design layout, different systems generally having been used by competing design groups. These design variants affect the structures at a fundamental level, including the level of usage of prestressed or conventionally reinforced elements. Therefore the different design types will be described separately before common prestressing and grouting procedures are described in more detail.

2.2 HOWARD DORIS DESIGNED GRAVITY BASE STRUCTURES

The Howard Doris designed offshore platforms remain a distinct group of structures in the North Sea owing to their wide central column design. On all of these structures, the wave
action is minimised using the Jarlan system of perforated breakwater walls. The principal characteristics of the Doris structures are given in Table 2.1.

<table>
<thead>
<tr>
<th>Platform Name</th>
<th>Ekofisk Tank</th>
<th>Frigg CDP1</th>
<th>Frigg MP2</th>
<th>Ninian Central</th>
</tr>
</thead>
<tbody>
<tr>
<td>Client</td>
<td>Phillips</td>
<td>Total</td>
<td>Total</td>
<td>Chevron</td>
</tr>
<tr>
<td>Water Depth</td>
<td>70m</td>
<td>96m</td>
<td>94m</td>
<td>139m</td>
</tr>
<tr>
<td>Construction Site</td>
<td>Stavanger,</td>
<td>Andalsnes,</td>
<td>Andalsnes,</td>
<td>Loch Kishorn,</td>
</tr>
<tr>
<td></td>
<td>Norway</td>
<td>Norway</td>
<td>Norway</td>
<td>UK</td>
</tr>
<tr>
<td>Installation Date</td>
<td>1973</td>
<td>1975</td>
<td>1976</td>
<td>1978</td>
</tr>
</tbody>
</table>

### Dimensions:
- Base Diameter: 92m
- Diameter at Sea Level: 92m
- Substructure Height: 90m
- Deck Area: 10 000m²
- Storage Capacity: 160 000m³

### Quantities of Materials:
- Volume of Concrete: 80 000m³
- Weight of Reinforcement: 10 000t
- Weight of Prestressing: 3 500t

### Prestressing System
- Freyssinet 12T13 and 24T13, Type 199
- Freyssinet 12T15 Type 294
- Freyssinet 12T15 Type 294
- Freyssinet Monogroup

### 2.2.1 The Ekofisk Tank

The geometry and prestress design of the Ekofisk Tank are described in an article published by Freyssinet International.

**Geometry:**

The Ekofisk Tank was primarily conceived as a storage facility for crude oil, and it retains a unique design amongst the other North Sea platforms. However, many of the construction techniques were re-used for larger and more intricate designs that followed.

The structure is founded on a 92m diameter, 6m high rigid cellular raft. This is built upon a bottom slab of 0.6m to 0.7m depth. Under the main storage tanks, approximately 45m wide, the raft is divided into a 5m square grid by a system of internal diaphragm walls. Beyond this central area, the diaphragms radiate out towards the curved outer edge of the raft. The internal cells so formed are ballasted, and the whole assemblage sealed by a 0.2m thick reinforced concrete slab.

The 70m tall 9-cell reservoir is surrounded over the entire height by a perforated breakwater wall. The structure is divided by diaphragm walls which vary in thickness from 0.65m to 0.50m. They are set 15m apart and run across the two principal axes of the structure. The curved walls of the reservoir span between them. Between the central reservoir and the external breakwater wall, the diaphragms thicken to 1.0m and are perforated to allow the free flow of water within this area.
The external breakwater wall is 1.35m thick, but thickens to 1.83m over the top 32m. The deck structure is supported directly above this upon 20 steel columns. A lower deck of structural steel supports the crude oil handling facilities. Above this, a second precast prestressed concrete deck carries a gas liquefaction plant.

Construction:

The base raft was built completely within a temporary dry dock. Work then commenced on the construction of the outer perforated wall using voided precast concrete blocks that acted as permanent shuttering. These were designed to allow room for the installation of prestressing ducts which run in both a horizontal and vertical direction. These blocks were then joined by in situ concrete.

The reservoir walls and internal diaphragms were built using a system of slipforms across the entire central section. The central slipformed section was finally joined to the outer breakwater by the perforated diaphragms.

The platform took two years to build between 1971 and 1973.

Prestressing:

The vertical tendons in the reservoir walls have a J-shaped profile and pass from a height of 36m down to the base raft. In addition to these, 90m long vertical tendons run over the entire height of the wall, anchored in the base raft and at the wall top.

In the lower half of the breakwater wall, the vertical prestressing is anchored 36m above the sea floor, this comprising both U-shaped and trapezoidal profiles. The trapezoidal tendons prevent shear cracking under differential loads between the wall and diaphragms. The reduced number of vertical tendons are provided in the top half of the wall, these consisting of grouped vertical tendons at the ends of the diaphragms and U-shaped profiles above the anchorages of the trapezoidal tendons.

The horizontal prestressing has been anchored within the confines of the oil storage tanks wherever possible. The prestressing in the breakwater wall is anchored in the diaphragms.

2.2.2 Frigg CDP1 and Frigg MP2

The design and construction of the Howard/Doris Frigg platforms are described separately in articles by Michel, Sedillot and Mondonf.

Geometry:

The Frigg CDP1 and MP2 platforms are almost identical, despite minor differences to accommodate differences in water depth.

The Frigg CDP1 platform bears on a 101m diameter raft, the base slab of which is between 0.6m and 0.8m thick. This is stiffened by a system of concentric 15m high walls and diaphragms. The outer two concentric walls are perforated to reduce scour effects on the sea floor.

The main body of the structure is a lobed 62m diameter cylinder. The lower 68m of the outer wall is 0.55m thick and stiffened by 6 radial diaphragms. Above 68m, the wall becomes a 1.20m thick perforated breakwater, stiffened at the top by 4m deep radial beams. A central 9m diameter shaft encloses the gas pipelines over the full height of the substructure. This central shaft is connected to six radial tunnels across the base of the cylindrical body.

A 4000m² deck is supported by a lattice of precast prestressed concrete girders. These have a depth of 5m. The girders are supported on the central shaft and fourteen 2m diameter steel columns filled with concrete, bearing on the breakwater.

Frigg CDP1 has a second steel deck above the first, and a total deck area of about 10000m².
Construction:

The substructure was built to a height of 15m in the dry-dock before being towed into the fjord. The slipforming of the lobed cylinder then continued to 23m before the outer section of the raft was flooded and the anti-scour wall plugs removed. Slipform construction then proceeded to 68m height.

The breakwater wall of the Frigg CDPI platform was built using the method developed for the Ekofisk Tank where hollow precast units were used as permanent shuttering. However, for Frigg MP2, a new method was adopted where the voids in the breakwater wall were formed by plastic shapers inserted into the slipform as it passed.

Upon completion of the breakwater wall, the radial precast beams were placed, followed by the steel columns and the deck. After installation, the main body of the structure was ballasted with a 52m deep layer of sand to ensure stability.

Prestressing:

The principal prestressing tendons in the raft foundation run parallel to the internal diaphragms under the main body. These then split to form a 'V' towards the outer edge of the base slab, running parallel to the external 15m high diaphragms. Since the main strength of the raft is provided by these diaphragms, the prestressing tendons are provided to counteract tensions at their base. The raft slab itself is therefore essentially a non-prestressed element supported between the diaphragms.

The other walls of the raft act as partition beams and are reinforced by horizontal tendons at their top and bottom. Additionally all of the diaphragms and partition walls contain vertical U-shaped tendons which ensure continuity with the raft slab.

The tunnels are attached at one end to the central column and then supported over the internal walls of the main structural body. The prestressing in the tunnels therefore has a long undulating profile to take account of these support conditions.

In order to withstand the wave loading, the diaphragms and walls of the lobed cylinder are reinforced by long 'J'-shaped vertical tendons embedded in the raft slab and rising to 68m. In the breakwater wall, U-shaped profiles are used.

The cylinder diaphragms and walls also contain horizontal tendons. Special attention was given to maintaining a simple and uniform layout with anchorages placed at accessible positions near the central shaft. This required the tendons in the radial diaphragms to turn through 180° on a 1m radius at the intersection with the external walls.

The precast beams of the deck structure were post-tensioned after installation. The empty ducts were cast into the beams during manufacture and later connected across a 0.40m wide in situ joint between the members.

2.2.3 Ninian Central

The design and construction of the Ninian Field platform is described in a paper by Draisey et al.

Geometry:

The principal design concept of the Ninian Central platform is an extension of that used in the Frigg platforms, the main difference being the provision of three sealed buoyancy chambers around the central body of the structure. These, together with their supporting diaphragms, provide a stepped pyramidal structure between the 140m diameter base raft and the 45m body diameter at sea level.

The base slab has a thickness between 0.85m and 2.50m. Steel skirt foundations of 4m depth are embedded into its underside under the 14m, 63m, 104m and 140m diameter substructure elements. The base slab is completed by 2 additional 15m high walls near the circumference, the outer one being perforated. These are connected by 8 pairs of stepped diaphragms connected in V-formation to the sealed buoyancy chambers.
The sealed buoyancy chambers have a lobate structure strengthened by the 8 diaphragms. The chambers have external diameters of 104m, 84m and 63m and are 55m, 65m and 75m tall respectively.

The structure centres around a 160m tall, 14m diameter central shaft, surrounded by a 45m diameter, 152m high cylindrical wall. The latter acts as a breakwater over its 72m top section.

The platform is topped by a 4000m$^2$ steel deck which is supported by steel columns on the breakwater wall and directly by the central shaft.

**Construction:**

The construction of the Ninian Central platform is distinguished from the earlier Frigg platforms by an increased use of large precast concrete elements. These are particularly apparent in the domed roof structure of the buoyancy cells and in the breakwater walls.

The base was constructed in nine sections, comprising 8 outer segments divided mid-way between diaphragms, and a central core. The segments were cast one against the other with the inclusion of a rubber water-stop joint. Upon completion of each segment, work began on the slipforming of the outer walls and diaphragms which were completed to full height in the dry-dock phase.

The main body elements were built in the wet dock phase using a system of independent slipforms working in concentric stepped formation on the walls. The domed roof was built from factory produced precast elements transported to the site. Each unit weighed 60t and was supported on four steel legs so that it spanned between the lobate walls. Continuity of reinforcement between the walls and the elements was achieved using couplers to reduce the size of the in situ joint.

The 1.5m thick perforated breakwater wall between levels 116m and 149m was built from 2.5m high quadrant match-cast sections. The top surface of these segments were flat whereas the underside had two 140mm wide strips in contact with the unit below, in addition to a limited zone surrounding the vertical prestressing ducts. Between these areas lay a 10mm recess that was grouted in situ. Before grouting this recess, the joint between the two elements was sealed by the injection of an epoxy mortar. The sealing of the joint between the vertical prestressing ducts was further ensured by a mastic gasket set into a groove in the upper face of the lower element.

The vertical joints between the precast elements were cast in situ, with reinforcing and horizontal prestressing passing through them. They were cast following the stressing and grouting of the vertical tendons.

**Prestressing:**

A total of 3160t of prestressing steel was used for the platform, 40% of which was installed, stressed and grouted during the dry-dock phase. The principal prestressing elements include:

- 8320 Macalloy bars of 40mm diameter, each about 3m in length, for connecting the steel skirt to the base slab
- a system of radial and ring tendons in the base slab
- overlapping horizontal ring tendons and vertical U, J or straight tendons in the walls, up to 70m tall
- horizontal tendon systems and vertical tendons in the diaphragms
- vertical J and straight tendons in the shaft
- vertical prestressing tendons connecting the steel corbel ring for the deck structure to the 45m diameter walls.

The Macalloy bars for attaching the steel skirt to the base were sherardized to reduce the risk of corrosion, and were coated in a bond breaker surface tape to allow stressing after the casting of the base slab. Corrosion protection of the top anchorage plates and nuts was by an in situ capping beam, whereas the lower nuts were protected by individual plastic caps filled with bituminous mastic.
The principal prestressing was housed in 76mm diameter, spirally wound steel ducts. Typically these were installed in 6m lengths, coupled by 300mm long external joints. All vertical tendons were threaded into rigid steel tubes connected by 150mm long spigot and socket joints. These joints were spot welded to prevent lifting during the slipforming jacking operation and sealed with tape.

The curved section of the I and J-shaped ducts were built from preformed 85mm diameter pipe. These had a minimum bend radius of 1.2m.

2.3 SEATANK DESIGNED GRAVITY BASE STRUCTURES

The design and construction of the Seatank/McAlpine gravity base structures are described in an article by Derrington.

Geometry:

In total, 3 Seatank/McAlpine structures were built between 1973 and 1978. The structures share a similar geometry, only their dimensions increased with each successive structure (see Table 2.2).

<table>
<thead>
<tr>
<th>Platform Name</th>
<th>Frigg TP1</th>
<th>Brent C</th>
<th>Cormorant A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Client</td>
<td>Elf/Aquitaine</td>
<td>Shell/EssO</td>
<td>Shell/EssO</td>
</tr>
<tr>
<td>Water Depth</td>
<td>104m</td>
<td>140m</td>
<td>152m</td>
</tr>
<tr>
<td>Construction Site</td>
<td>Ardyne Point, Scotland</td>
<td>Ardyne Point, Scotland</td>
<td>Ardyne Point, Scotland</td>
</tr>
<tr>
<td>Installation Date</td>
<td>1976</td>
<td>1978</td>
<td>1978</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Dimensions:</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Caisson side length</td>
<td>72m</td>
<td>91m</td>
<td>100m</td>
</tr>
<tr>
<td>Caisson Height</td>
<td>42m</td>
<td>57m</td>
<td>56m</td>
</tr>
<tr>
<td>Number of Towers</td>
<td>2</td>
<td>4</td>
<td>4</td>
</tr>
</tbody>
</table>

| Tower External Diameters: |           |         |             |
| At Base                 | 14m       | 15m     | 16m         |
| At Top                  | 9m        | 9m      | 9.5m        |

| Substructure Height    | 126m      | 165m    | 172m        |
| Deck Area              | 2 750m³   | 4 000m³ | 4 250m³     |
| Storage Capacity       | Not provided | 105 600m³ | 160 000m³ |

| Quantities of Materials: |           |         |             |
| Volume of Concrete      | 70 000m³  | 107 800m³ | 131 000m³ |
| Weight of Reinforcement | 5800t     | 11 400t  | 139 30t    |
| Weight of Prestressing  | 500t      | 1400t    | 1100t      |

The structures are founded on a square base raft with side lengths ranging from 72m to 100m. The base raft is built over a 3 to 4m thick flat slab, attached to a 2 to 3m long precast concrete skirt. This base slab is cast between stiffening beams at 7m intervals running between the main cell walls, and is bounded by a 3.5m square beam section along its periphery.
The caissons are between 42 and 56m tall. The cell roof structure is made from a series of truncated pyramids or cones, except at Cormorant A where a spherical construction was used.

The deck structure is supported upon tapered concrete towers, 2 for Frigg TP1 and 4 for Brent C and Cormorant A. The towers at the Cormorant platform have an external diameter ranging from 16m to 9.5m, and are 116m tall above the base caisson.

Construction:

The precast concrete skirt sections were placed in a trench within the dry-dock so that the formwork for the main slab could be erected upon the ground. The base slab was built in alternate sections and in two layers, the first 0.5m only being cast in the dry-dock so that adequate buoyancy at float out could be achieved. The joints between the different slab sections were cleaned and bush-hammered before the casting of the adjacent part.

The walls of the caisson were slipformed to 15m in the dry-dock, and then to completion in a series of 10-15m steps while afloat. When sufficient buoyancy was available, the base slab was cast to the full thickness of 3 to 4m in a further 1 to 3 pours. Each joint was injected with epoxy resin.

For the first two platforms, the pyramidal cell roof structures were built above a temporary steel and ply-board deck spanning between pins set into the cell walls. However, this method proved costly in materials and time, and so a different method was developed for the Cormorant A platform. Here, a 100mm thick precast concrete cap was placed over the roof and used as permanent formwork. This cap was post-tensioned at its edges using ungrouted sheathed tendons. The principal reinforcement for the roof structure was attached to it in the prefabrication yard, giving a total unit weight between 70 and 90t.

Slipforming of the towers continued in parallel with the construction of the roof. Screwed bar couplers were used at the joint between the towers and the roof to provide continuity in this highly reinforced area. Slipforming achieved 3m per day and in the splash zone, the concrete was painted with an epoxy resin primer as it emerged from the slipform. For the Frigg TP1 platform, additional stiffening rings were placed at intervals in the tower to contain the prestress anchorages. These were cast after the passing of the slipform. A steel capping ring was placed at the top of the columns and stressed down to provide a connection with the steel deck structure.

Prestressing:

The base slab was lightly prestressed in the principal horizontal directions, the anchorages placed in the peripheral beam. In addition, the towers were heavily vertically prestressed, but otherwise, the structure was built entirely from reinforced concrete. The vertical tendons were placed in rigid metal ducts with welded plastic sleeve joints. These tendons were principally U-shaped in profile, and were stressed at the end of the slipforming operations.

In the Cormorant A platform, a different prestressing system was adopted in the towers to eliminate the need for the intermediate stiffener rings and anchorages. This system used 85 wires of 7mm diameter in the place of strands.

2.4 THE ANDOC GRAVITY BASE STRUCTURE

The design and construction of the Andoc platform is described in the article by Sabatinelli.

Geometry:

The Andoc platform for the Dunlin field was designed with a base raft built from smaller storage cells above which towers support the platform deck. It was designed for a water depth of 153m.

The 32m high base contains 81 cells, each 11m square and in a square formation of 104m sides. The internal walls between the cells are straight, whereas those on the periphery of
the base are curved to resist hydraulic pressures. The cells are based on a 4m high
foundation raft comprising a bottom slab and 400mm thick, 4m deep rib beams spaced at
3.5m in two orthogonal directions. Under the base slab, a steel skirt runs along the
periphery of the structure and under the nine central cells.

The internal cell walls are 0.5m thick, and those on the exterior 0.75m. The cell roof
structure is internally vaulted above each cell, whereas the top surface is flat, therefore
producing a variable thickness slab from 0.80m to 2.15m.

The deck is supported by 4 tapered concrete towers 111m high, the diameter reducing from
22.65m to 6.00m. These are each founded centrally over 4 internal cells. Their walls
reduce in thickness form 1.2m at the base to 0.70m at the top. Two of the towers are
connected by a large steel frame which improves the rigidity of the substructure and gives
support to an array of 48 riser pipes.

A unique feature of this structure is that the concrete/steel interface between the
substructure and the deck is below sea level, the concrete towers being extended by 37m
tall steel towers upon which the deck sits. The deck has a surface area of 4500m².

Construction:

The steel skirt was placed in a trench so that the base raft could be cast at ground level.
When the perimeter walls reached 8.8m, the structure was floated and construction
continued by the slipform method. The rate of climb achieved 1.2m per day.

The cell roof slab was cast in situ upon precast permanent formwork sections.

After completion of the towers, the structure was towed to Stord in Norway. Here it was
ballasted to reduce the freeboard and enable placement of the 30m steel tower sections by
floating cranes. The joints were then sealed and the substructure further ballasted to enable
mating with the deck structure.

Prestressing:

The foundation slab is horizontally prestressed on a grid of approximately 1m spacing.
This is provided by a system of 36mm diameter bars. In addition to this, the steel skirt is
anchored to the structure by 3 separate layers of 36mm prestressing bars embedded in the
foundation slab.

The cell walls are both horizontally and vertically prestressed. The vertical tendons are U-
shaped and are anchored in the top face of the cell roof slab. The intersection of the
external walls and the roof slab is heavily reinforced by several layers of horizontal, curved
tendons anchored in large anchor blocks internal to the cells.

The four concrete tower sections are prestressed vertically. This vertical prestressing is
divided into 2 levels, the lower 72m containing 48 tendons, the upper 32 tendons.
Therefore, the tendons in the towers have vertical lengths between 40m and 110m.

2.5 CONDEEP DESIGNED GRAVITY BASE STRUCTURES

2.5.1 Early Condeep Structures

The design and construction of the Condeep platforms are described in an article by
Carlsen and Vindvik. In 1993 the topside facilities of the Beryl A platform were
upgraded, the work including a re-analysis of the structure under modern codes. Some of
the findings of this work are reported by Galbraith et al.
Table 2.3

Basic Characteristics of the Early Condeep Offshore Platforms

<table>
<thead>
<tr>
<th>Platform Name</th>
<th>Beryl A</th>
<th>Brent B</th>
<th>Brent D</th>
<th>Frigg TCP2</th>
<th>Statfjord A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Client</td>
<td>Mobil</td>
<td>Shell</td>
<td>Shell</td>
<td>Elf</td>
<td>Mobil</td>
</tr>
<tr>
<td>Water Depth</td>
<td>118m</td>
<td>140m</td>
<td>140m</td>
<td>104m</td>
<td>145m</td>
</tr>
<tr>
<td>Construction Site</td>
<td>Stavanger, Norway</td>
<td>Stavanger, Norway</td>
<td>Stavanger, Norway</td>
<td>Andalsnes, Norway</td>
<td>Stavanger, Norway</td>
</tr>
<tr>
<td>Installation Date</td>
<td>1975</td>
<td>1975</td>
<td>1976</td>
<td>1977</td>
<td>1977</td>
</tr>
</tbody>
</table>

Dimensions:
- Base Area: 6 300m², 6 400m², 6 400m², 10 000m², 7 900m²
- Caisson Height: 48m, 60m, 57m, 45.5m, 68m
- Number of Cells: 19, 19, 19, 19, 19
- Number of Towers: 3, 3, 3, 3, 3
- Tower External Diameters:
  - At Base: 20m, 20m, 20m, 20m, 20m
  - At Top: 12.2m, 12.2m, 12m, 10.4m, 13.2m
- Substructure Height: 149m, 164m, 166m, 130m, 176m
- Storage Capacity: 144 000m³, 160 000m³, 192 000m³, None, 208 000m³

Quantities of Materials:
- Volume of Concrete: 62 000m³, 65 000m³, 65 000m³, 62 000m³, 87 000m³
- Weight of Reinforcement: 8 500t, 10 700t, 14 000t, 14 000t, 12 000t
- Weight of Prestressing: 820t, 1 150t, 1 500t, 600t, 2 600t

Prestressing System:
- Freyssinet
- 12T13
- Type 199

Geometry:

Within the 2 years from 1975 to 1977, a total of 5 Condeep platforms were commissioned. All of these shared the same basic design, with only slight adaptations to meet the particular requirements of each scheme. The variations between the early Condeep structures are shown in Table 2.3.

The basic Condeep platform has a base built up from 19 cylindrical, 20m diameter storage cells joined in a 100m diameter hexagon and joined as a single unit. Each cell is closed at the bottom by an inverted dome, the walls of the cell extending downwards beyond the base to form a concrete skirt. Depending upon the soil conditions at the installation site, some of the walls are further extended by a 3 to 3.5m long steel skirt.

The interface between the lower domes and the cell walls is designed as a ring beam to support the lateral forces from the dome structures. This ring beam is also important during the towing out phase from the dry-dock since it provides an additional air cushion, so reducing the necessary draught.

Sixteen of the 19 cells are also closed out at the top by domes, while the remaining three cells continue up to form tapered concrete towers supporting the superstructure. Two of these towers are used for oil drilling purposes, the third for controlling the water and oil levels in the storage cells. The tower walls vary in thickness from 1.15m to 0.50m.
Construction:

In the dry-dock phase, the lower portion of the storage cells were constructed to a level giving the required buoyancy at float out. Upon floating, the top domes and covering slab had not yet been built, hence the structure was still fairly flexible and care was required to avoid undue moments and consequent cracking of the concrete.

The steel skirt was erected at ground level, with interconnecting temporary access holes for construction. The concrete skirting and steel skirt mounting were built above on scaffolding, a few cells at a time so that the shuttering could be re-used. This was followed in a similar manner by the lower dome construction. Once the foundations were complete, simultaneous slipforming of the 19 cell walls began.

During slipforming in the wet dock phase, the base was ballasted by water in the cells so that a constant freeboard was maintained. The slipform operation progressed at a rate of about 1.8m per day. Before casting the top domes, the permanent aggregate ballast was brought in and compacted in a carefully controlled operation. This was overlaid by a thin concrete slab to prevent mixing with the oil.

After casting and prestressing the top domes of the cells, the slipforming of the three towers continued. At Statfiord A, the rate of climb achieved 3m per day and the towers rose to a height of approximately 115m above sea level.

Upon completion and fitting out of the towers, the substructure was lowered in the water by increasing the internal water level so that the preassembled deck unit could be placed in one operation. To ensure structural integrity under the extreme hydrostatic pressures induced, compressed air was pumped into the 12 peripheral cells at about 4 atmospheres. The steel deck was floated over the semi-submerged substructure, supported between two barges. The substructure was then raised in the water to lift the deck.

After towing to the installation site and grounding on the sea bed, the required penetration depth of the steel skirt was achieved under the weight of the structure, aided by evacuating water from beneath the structure as necessary. Grout was then pumped inside the steel skirt to ensure an even bearing.

Prestressing:

The concrete skirt sections are prestressed horizontally. This is provided to ensure that the skirt is airtight to assist in float out from the dry-dock or in its subsequent removal at the end of its service life. The prestressing also helps contain lateral forces between the foundation skirts and the substructure.

The lower dome structures are prestressed by a web of tendons which are designed to resist locally reduced pressures from uplift on the structure due to wave loading. This prestressing steel is anchored in the lower ring beam, which is prestressed itself to maintain it in compression.

Horizontal prestressing is also used at the base of the upper dome structures, again forming a ring beam to resist any lateral forces from the domes.

The towers are vertically prestressed over their entire length, the tendons continuing into the cell wall below the tower where they are anchored. This ensures continuity between the towers and the foundation raft. The towers are subject to very high bending moments under wave action, and the prestressing reduces lateral sway at the deck due to these forces by stiffening the substructure and reducing vibrations.

Tensions at the base of the towers are transferred into the cell walls and also into neighbouring cell roof structures by a system of horizontal tendons.

2.5.2 Later Condeep designs

The design of the newer Condeep platforms has been described by Aldstedt et al.16

After the initial platforms, construction of the later Condeep type structures resumed at the beginning of the 1980's with the development of the North Sea oil fields into deeper water.
The later structures were larger than their predecessors, the base raft now incorporating 24 storage cells with a larger storage capacity. Additionally, the deck was now supported above 4 towers.

Improvements were also made to the deck structure to enable a higher degree of topside completion before installation on site. To this end, a new T-shaped layout was adopted for the Gullfaks platforms to facilitate the placement of modular topside units. This new design was reflected in an altered substructure layout.

The Condeep design concept was further adapted for the Draugen platform installed in 251m of water. Here, the three towers converge to form a large mono-tower structure.

<table>
<thead>
<tr>
<th>Platform Name</th>
<th>Client</th>
<th>Water Depth</th>
<th>Installation Date</th>
<th>Volume of Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Statfjord B</td>
<td>Mobil</td>
<td>145m</td>
<td>1981</td>
<td>140 000m³</td>
</tr>
<tr>
<td>Statfjord C</td>
<td>Mobil</td>
<td>145m</td>
<td>1984</td>
<td>135 000m³</td>
</tr>
<tr>
<td>Gullfaks A</td>
<td>Statoil</td>
<td>135m</td>
<td>1986</td>
<td>125 000m³</td>
</tr>
<tr>
<td>Gullfaks B</td>
<td>Statoil</td>
<td>141m</td>
<td>1987</td>
<td>100 000m³</td>
</tr>
<tr>
<td>Oseberg A</td>
<td>Norsk Hydro</td>
<td>109m</td>
<td>1988</td>
<td>120 000m³</td>
</tr>
<tr>
<td>Gullfaks C</td>
<td>Statoil</td>
<td>216m</td>
<td>1989</td>
<td>240 000m³</td>
</tr>
<tr>
<td>Sleipner A</td>
<td>Statoil</td>
<td>82m</td>
<td>1993</td>
<td>75 000m³</td>
</tr>
<tr>
<td>Draugen</td>
<td>Shell</td>
<td>251m</td>
<td>1993</td>
<td>85 000m³</td>
</tr>
</tbody>
</table>

2.6 MORE RECENT GRAVITY BASE STRUCTURES

2.6.1 Ravenspurn North Platform

The design and construction of the Ravenspurn platform is described in an article by Luedke et al17. The basic characteristics of the platform are compared to those of other recent gravity base platforms in Table 2.5. The platform consists of a cellular caisson base with three circular concrete towers. Two of the towers support the gas production deck and the third supports the gas compression module.

<table>
<thead>
<tr>
<th>Platform Name</th>
<th>Client</th>
<th>Water Depth</th>
<th>Installation Date</th>
<th>Volume of Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ravenspurn</td>
<td>Hamilton</td>
<td>42m</td>
<td>1989</td>
<td>10 000m³</td>
</tr>
<tr>
<td>F3</td>
<td>NAM</td>
<td>43m</td>
<td>1992</td>
<td>21 000m³</td>
</tr>
<tr>
<td>Harding</td>
<td>BP</td>
<td>109m</td>
<td>1995</td>
<td>36 000m³</td>
</tr>
</tbody>
</table>

The topside modules were installed after the installation of the structure using a semi-submersible crane barge. This allowed considerable savings in concrete and cost of the structure since traditional submersion of the substructure during mating with the deck has often been critical to the concrete design capacity. The reduced weight of the structure enabled construction to be completed entirely within a dry dock, with a float out draft of only 9.7m.
The base is 54m by 62m. The caisson is a stepped structure to give maximum clearance for the semi-submersible crane during deck installation. This design also allowed additional ballast to be placed after installation was complete. The foundation is ensured by a two metre deep skirt around the periphery of the base and under the internal walls.

The substructure has a total weight of 24347t, of which 2450t is conventional reinforcing steel and 450t is the prestressing steel. A steel transition piece is provided between the substructure and the deck to facilitate installation.

The substructure was installed by tilting it to 37° until one edge was in contact with the sea floor. This unusual technique was adopted to help prevent instability.

2.7 ALTERNATIVE STRUCTURE TYPES

2.7.1 Floating Structures

Since the installation of the Ekofisk gravity base tank in 1973, oil production facilities have been installed in increasing depths of water. This has required further developments in platform design since the well established gravity base concept was less economical for depths exceeding 300m.

The oil companies have begun to show a preference for floating structures as the exploitation of the oil fields spreads eastwards into deeper water. In general, the quantity of materials used in floating platforms is independent of the water depth. Therefore, the main parameter dictating the size of the structure is the weight of the deck, since sufficient buoyancy must be provided.

The viability of the concrete floating platform had been successfully demonstrated on a smaller scale by the Maureen Field articulated loading column, installed in 1982. During 1995, two floating concrete platforms were installed in the North Sea; the Troll Oil platform and the Heidrun. These two structures have been designed using different design concepts, the Troll Oil platform being of catenary moored semi-submersible type, the Heidrun a tension leg platform. These platforms are compared with a gravity base structure installed in a similar water depth in Table 2.6.

<table>
<thead>
<tr>
<th>Table 2.6</th>
<th>A Comparison of Two Floating Platforms with a Gravity Base Platform</th>
</tr>
</thead>
<tbody>
<tr>
<td>Platform Name</td>
<td>Heidrun</td>
</tr>
<tr>
<td>Client</td>
<td>Conoco</td>
</tr>
<tr>
<td>Water Depth</td>
<td>345m</td>
</tr>
<tr>
<td>Topside Weight</td>
<td>90 000t</td>
</tr>
<tr>
<td>Platform Type</td>
<td>Tension Leg</td>
</tr>
<tr>
<td>Duration of Construction</td>
<td>2 years</td>
</tr>
<tr>
<td>Installation Date</td>
<td>1995</td>
</tr>
<tr>
<td>Quantities of Materials:</td>
<td></td>
</tr>
<tr>
<td>Volume of Concrete</td>
<td>66 000m³</td>
</tr>
<tr>
<td>Weight of Reinforcement</td>
<td>27 000t</td>
</tr>
<tr>
<td>Weight of Prestressing</td>
<td>4 100t</td>
</tr>
</tbody>
</table>

2.7.2 The Maureen Articulated Loading Column
Geometry:

The Maureen loading column comprises an 89m long, 9m diameter column designed to float above a semi-buoyant concrete base unit seated on the sea floor. A steel chimney is stressed to the top of the column, above which is a 200t steel lattice structure supporting a helideck and the crude oil connection pipes for offloading to a tanker. The concrete column and base buoyancy units were constructed from 16m long slipformed ring sections with a 300mm wall thickness.

The base buoyancy units were built from 2 of the precast ring sections, joined by a 1.5m wide in situ stitch. These units were placed on the base frame, located between buttresses acting against corbels stressed to the sides of the ring units. The bearings between these two elements were completed by neoprene pads to allow for slight movements during installation. This assembly was then locked in place by the completion of the top of the buttresses, this enclosing the corbel upper face.

The main column is connected to the base via a steel Cardan articulated joint. This joint is attached to the base through a steel connection plate, and to the column through a steel seating stressed to the column foot.

A steel cofferdam is installed inside the column at the permanent sea level position, with all of the steel working decks, ballasting systems and access holes.

Construction:

The concrete used throughout construction was a C55 grade concrete using low C\text{3A} content aggregate. The ring segments were slipformed vertically before being turned on their sides for assembly in the dry dock.

The 1.5m in situ joints between the segments of the buoyancy units and main column are reinforced by steel bars lapped to starter bars protruding from the ends of the slipformed sections. To ensure watertightness, a 30mm square circumferential recess was provided in the outer and inner faces of the joint. This recess was finally sealed with an epoxy mortar.

Prestressing:

The principal prestressing in the column is provided by 38 tendons located in ducts running over the full height of the column. These were stressed and grouted while the column was in the horizontal position, and were dimensioned to provide adequate bending capacity. The maximum moments in the column were experienced during installation, when it was raised from its horizontal position to the vertical.

2.7.3 The Troll Oil Catenary Moored Platform

The Troll Oil catenary moored platform is described in an article by Sjetnam\textsuperscript{19}.

Geometry:

The Troll Oil platform comprises a deck supported on four towers, these joined under water by a pontoon. This configuration was adopted to improve the stability of the structure, although this type of structure cannot be used with rigid risers due to excessive movements, and flexible pipes have to be used.

The hull of the Troll Oil platform has the overall dimensions of 101x101x63m, and was originally designed to hold 100 000m\textsuperscript{3} of oil, although this capacity was finally not needed. Within the hull, oil is stored above a layer of water. Above this is an air pocket, regulated to maintain the correct buoyancy of the structure.

The pontoons are 15m high and 30m wide, and are strengthened periodically by internal ring beams and bulkhead walls. The slab and wall thicknesses vary between 0.4m and 0.9m. The towers are 30m in diameter, 50m tall, and are divided internally by concrete slabs. The subdivision of the interior of the structure was necessary to ensure the buoyancy of the platform should the structure suffer a ship impact.
The corner nodes between the towers and the pontoons required special attention during the design phase. As well as providing the intersection between the different structural elements, it is here that the mooring chains are attached. The prestressing force needed to counteract these forces amounts to almost 20,000t.

*Construction:*

The Troll Oil platform was built entirely within dry-dock facilities. This facility offered sufficient draft for the completion of the substructure including some of the mechanical installations.

Construction took a little over 14 months to complete.

*Prestressing:*

The requirement for the structure to be watertight meant that the levels of prestressing are far higher than for a typical gravity base structure. The entire shell of the substructure needed to be prestressed in 2 directions, and this was achieved by 6 sets of different sized tendons, each performing different functions. In total, the structure contains more than 7500 anchorages.

The ducts are of spirally wound steel, this being required to ensure adequate flexibility for the complicated prestressing layout.

The location of the prestressing tendons for the transition piece between the deck and the substructure needed to be very precise. The transition piece took the form of a steel corbel ring around the entire circumference of each of the towers. Each ring was held in place by 100 prestressing tendons anchored in the substructure, with a total prestressing force of 40,000t.

**2.7.4 The Heidrun Tension Leg Platform**

The design and construction of the Heidrun tension leg platform are described in articles by Kepp and Botros, and by De Oliveira and Fjeld.

The objective behind the tension leg concept is to reduce platform heave, pitch and roll using parallel mooring tendons kept in constant tension. This platform type can provide sufficient stability for rigid risers to be used. For the Heidrun platform, the calculated maximum horizontal displacement is 31m under severe loading conditions, equivalent to 10% of the water depth. Under such movements, the structure would lower in the water by 2m.

Care is needed both in the design and operation of a floating platform to ensure that sufficient buoyancy is maintained. In addition, the design must consider accidental loading conditions to ensure that the structure would survive storm loading after the hull has been punctured.

*Geometry:*

The Deck of the Heidrun is of a modular design, similar to the gravity base platforms, whereas the substructure comprises the hull, the tension legs and the foundation construction.

The hull has a total height of 110m, with a draft of 77m, and comprises four cylindrical towers at 80m centres which are connected at the base by a square pontoon frame. The deck is supported above two parallel support beams spanning between the towers at the top. Wall thicknesses are about 0.5m.

Internally, the pontoons are compartmentalised to allow them to be individually waterballasted to ensure that the structure maintains the correct attitude in the water. The towers have a double wall at the waterline to minimise any possible damage following collision with a ship and the consequent influx of water. These double walls form a separate compartment 2m deep and 30m tall, 20m of which lie below the waterline.

The tension legs are anchored in groups of 4 against the lower portion of the towers. These legs are made from steel pipes which remain empty in normal service and have almost neutral buoyancy. They are equipped with an anchoring system at both ends which allow a moment-
free load transmission even during the largest horizontal movements. Each leg has a breaking strength of about 50MN.

The foundations are each made up of 19 cells, the walls of which are extended vertically down to form a skirt foundation. The tension legs are anchored within an enormous frame, which spans the upper diameter of the cells. Each of the 4 foundations is designed so that it can be installed without the help of a crane.

Construction:

The construction of the Heidrun platform followed the same pattern developed for the earlier gravity base structures. The pontoon structure was completed in the dry dock before construction continued on the towers whilst afloat. Slipformed concrete was used extensively.

The Heidrun was the first large offshore platform built entirely from light-weight aggregate concrete. Use of concrete in the deck structure required special attention to the spalling behaviour of the concrete in hydro-carbon fuelled fires, where temperatures can reach 1000°C in a matter minutes. The spalling behaviour was improved by use of polypropylene fibres which melt at 120°C leaving a series of canals through which vapour pressures can disperse. These fibres did not appear to affect the strength of the concrete.

2.7.5 The Ekofisk Barrier

Also worth mentioning is the Ekofisk barrier, built in 1988 to protect the Ekofisk tank from the '100 year' design wave. This became necessary as the oil extraction in the area had lowered the sea floor by up to 3.5m.

The structure is 106m tall, the top 22m installed once the structure was in position. This top portion was built from prefabricated elements. The bottom part of the structure was built in two halves so that the structure could completely surround the Ekofisk Tank.
3. PRESTRESSING DESIGN AND CORROSION PROTECTION

3.1 PRESTRESSED CONCRETE IN OFFSHORE APPLICATIONS

The use of prestressing is fundamental to the design of concrete gravity based structures. Buoyancy conditions at float-out require the structure to be as light as possible, therefore precluding heavier reinforced concrete. Most importantly, the prestressing is required to prevent cracks in the concrete. This helps protect the steel reinforcement from corrosion and fatigue, and is also required if the structure is to remain water-tight.

Concrete offshore structures are typified by cylindrical or spherical element forms due to the inherent ability of concrete to resist compression. However, these configurations are ill-suited to cyclic shear forces under the action of wave loading since the reinforcement is only partially effective against diagonal shear cracks. Prestressing may be used to alter the inclination of this weak plane so that the conventional reinforcement is more effective.

In the early structures, drawdown, where the water level inside the structure is lower than the surrounding sea level, is used to help maintain the compression in the concrete. This drawdown, often of the order of 40m, was important to maintain structural integrity during storm loading conditions. In the case of the damage and subsequent repair to the Brent B platform, this was found to cause serious difficulty\textsuperscript{12}, and has more recently been avoided.

Large prestressing forces are required because of the scale of the structures. Therefore the tendon layouts can become congested. The main tendon configurations used are:

- curved horizontal tendons in the reservoir and breakwater walls, overlapping at the diaphragms
- straight horizontal tendons in the diaphragms (These tendons must be installed at the same rate as the slip-forming for reasons of access)
- vertical U-shaped tendons in the walls when the latter are not too high. (These tendons are stressed using a jack at each end)
- vertical J-shaped tendons in the walls when U-shaped tendons would be excessively long to install. (These tendons are stressed by a jack at the upper end only).

At the time of construction of the first prestressed concrete offshore platforms, the construction industry had little experience of such large scale use of prestressing tendons in a marine environment. However, land-based prestressing technology was already well established. In addition to this, invaluable experience had been gained from the use of prestressed and reinforced concrete in coastal structures, enabling Derrick to write that "the challenge to the industry... was mainly a matter of scale rather than of novelty."\textsuperscript{12}

3.2 DESIGN OF THE PRESTRESSING SYSTEMS

The prestressing systems have been supplied by the same manufacturer for most of the offshore platforms, and often this same company has been employed as the prestressing subcontractor. Therefore, there has been a continuity in the development of the special techniques required for offshore structures.

Standard spirally wound ducting has often been used for horizontal prestressing. The vertical deviation of horizontal ducts has tended to be very restricted where these have been installed in the slipformed walls. The slipform itself is supported on a frame passing just over the top of the concrete. At Frigg MP2, the clearance given was only 0.6m.

There has been widespread use of smooth bored thick-walled steel tubing for the vertical ducting. This has been chosen to help the threading of the prestressing tendons in the small radii bends often encountered, and so that the ducting is largely self supporting throughout the slipforming process. At tight bends in the ducting (up to 1.2m radius), the tube diameter has been increased.
The steel tubing is pre-bent to the appropriate radius and has been connected using a spigot and socket joint. There have been a few instances of these connections breaking under the stresses applied by the slipforming process, and so originally the joints were made 200mm long, although later this was successfully reduced to 150mm.

Vertical ducts must always be capped to prevent blockage by accumulating debris. Gerwick reports that blockage is more common on offshore structures due to the construction techniques adopted, requiring the ducts to remain open for longer than usual. Also, crowded prestressing design can increase the risk of interconnection between ducts, and therefore blockage by grout passing from one to the other.

It was commented by Long that up to 2% of ducts become blocked or damaged, preventing the threading of prestressing tendons. Gerwick suggests the provision of as much as 5% extra to overcome this.

Vents for the ducts have been found to be inconvenient since they interfered with the slipforming process. Therefore, they have often been placed further apart than the 15m required by the codes. In addition, during the construction of the BP Harding platform in 1994, vents were eliminated completely from the horizontal ducts in the base slab. This action was taken due to accessibility problems and following trials to demonstrate successful grouting without them.

Anchorage design has been the subject of much attention. The concentrated stresses in these areas have been known to promote cracking problems. In addition to this, the design detailing can be particularly difficult due to the requirement to limit any delay of the slipforming process.

Where the tendon is turned out from the wall into an anchorage beam, radial forces would tend to tear the protrusion from the wall. Therefore sufficient ties are required in an area already congested by the anchorages and other reinforcement.

Despite the requirements for heavy reinforcement in anchorage zones, the high axial loads from the prestressing also require the concrete in these zones to be well compacted and free from honeycombing. However, Gerwick reports that good compaction in this area can be difficult to achieve due to accessibility problems and the amount of reinforcement present.

3.3 INSTALLATION

All strand has received a coating of soluble oil at the manufacturing plant, this coating being refreshed periodically throughout storage. Due to the corrosive nature of the construction environment, more care than usual is needed in the storage of materials.

There was originally some concern about the ability to thread the prestressing strand in ducts over 100m in length, and incorporating sharp bends. Long reported in 1976 that, with a carefully made threading eye at the end of the tendon, few problems were encountered. The preferred method of tendon threading has been by pulling pre-cut tendon through the duct. This is in contrast to the more usual method of pushing the tendon from a complete drum, and is due to the restricted room and weight limits on the slipform platforms. In this way, the need to support a full coil of strand on the scaffolding is avoided.

Before threading, a plunger is blown through the duct to check for obstructions, attached to a light line. This line is then drawn back along the duct carrying the main pulling tendon. In turn, the prestressing strand is attached to this and drawn through.

Vertical tendons are generally threaded from the top down on a mechanised roller. This requires adequate braking since a 150m long tendon can weigh up to 2.5t.

In general, most horizontal tendons follow a simple profile and are threaded at low forces on a simple winch mechanism. However, problems have been encountered where these tendons follow sharply curving profiles, where greater threading forces have been required.

For reasons of access, all horizontal tendons in slipform walls need to be installed, stressed and grouted at the same rate that the slipform proceeds. Therefore special multi-storey
platforms are suspended from the slip-forms in the tendon anchorage zones. The ducts are installed from the slip-form deck just before concreting. The succeeding operations (threading and stressing of tendons and grouting) are carried out from the different floors of the platform which climbs at a rate of 1 to 2 metres a day.

On multi-legged structures, where the slipforming of the towers can achieve about 4m a day, horizontal prestressing has not been used. However, the anchorage positions of the vertical tendons have caused difficulty to the slipforming process and several different design solutions have been adopted to overcome this.

Stressing operations have not themselves presented any new technical problems, but have generally been found to be an exercise in management of the stressing equipment so that the concreting works are not held up.

3.4 GROUTING PROCEDURES

In terms of grouting, conditions in offshore construction are very different from those frequently encountered on land. Due to the construction methods used, it has often been difficult to locate the grout batching plant near to the anchorages, and so a central batching plant has generally been set up and grout pumped to the anchorages through large diameter delivery lines. In some cases the delivery lines have been up to 300m long.

In addition, for the earliest structures, the industry had little experience in the grouting of tall vertical ducts and much development work was necessary.

In the early structures grouts used were generally ordinary cement based grout, although retarders were often needed to compensate for the delivery time from the batching plant. Where tendons were closely spaced, it was also sometimes necessary to grout several ducts simultaneously due to possible interconnection between them.

Often, two distinct grout mixes have been used, distinguishing between vertical and horizontal ducts. These mix designs have undergone a significant evolution over the past two decades. Particularly for vertical ducts, many attempts have been made to improve grouting procedures to reduce the frequency of voids, and therefore to reduce the extent of re-injection operations.

3.4.1 Grouting of Horizontal Tendons

Grout mix design for horizontal tendons has generally been dictated by the need to keep the grout fluid for several hours to allow for extended delivery times, and by practical limitations concerning pumping pressures. In 1976, Long24 reported that grout recording a time of 12 to 30s in the CEN flow cone test was found to be satisfactory. Grouts outside of this range tended either to cause blockages in the pump or exhibit excessive bleeding and shrinkage in the ducts. However, some difficulties in complying to the bleed water reabsorption rules were reported, these being due to the quantities of retardant in the grout. Therefore this rule was relaxed to allow 48 hours for reabsorption to take place.

Recent years have seen the increased usage of admixtures to improve grout characteristics. For the Hibernia platform25, the grout used for horizontal tendons included a superplasticizer, a water reducing agent and a retarder. Using these, a water/cement ratio of 0.34 was achieved for a grout complying to the requirements in the Canadian codes. This gave a fluidity range between 12 and 25 seconds in the flow cone test, and a usable life of 3 to 4 hours. Despite this, most grout was injected within 1 hour of mixing. The batching of the grout was computer controlled to ensure correct mix quantities.

The practice of flushing out the duct prior to grouting was only adopted on the earliest structures. This was abandoned since the protective oil coating on the strand and any debris were found to be adequately removed by the initial head of grout. This is then expelled from the duct outlet.
3.4.2 Grouting of Vertical Tendons

Prior to the construction of the offshore platforms, there was little experience relating to the grouting of tall vertical ducts. Vertical ducts up 60m in length had been previously encountered in the construction of reactor containment vessels for nuclear power plants. However, difficulties had been encountered in the grouting of such tendons and adequate procedures had not yet been fully developed at the time the earliest platforms were constructed.

Problems arose due to the formation of large voids beneath the upper anchorage which needed additional inspection and topping up as necessary. These formed due to the grout being partially filtered through the interstices of the prestressing strand, and the water being driven to the top of the duct under the hydrostatic pressure. This is known as the 'wick' effect, and can account for a shrinkage of the grout by 3-4%. When the column of grout is over 100m in length, this can be a very significant amount. Ungrouted lengths of 3m were not uncommon.

Long\textsuperscript{20} reported that grouting trials were carried out on vertical ducts strapped to the walls of the platforms as they were built. The first trials resulted in large voids forming. The grouting method was therefore modified so that the strands were left protruding from the top anchorages to permit the bleed water to be removed.

In the earliest structures, grout would be re-injected at the top anchorage using a hand pump or by a gravity feed to fill up any voids that may have developed. Later, the grout mix was modified to reduce the wick effect. An expansive and a water-reducing admixture were added, with a subsequent reduction in the water/cement ratio from 0.42 to 0.34.

Aluminium-based expansive agents have also been employed. Early fears that the hydrogen gas evolved could help promote hydrogen embrittlement of the strand have largely been allayed.

In later trials on a 15m tall U-shaped duct, a technique was developed in which the grout was injected down one leg of the U and up the other. When the duct was filled, the direction of the grout flow was reversed by injecting it from the other anchorage. These trials showed that, on the 15m test piece, dropping the grout down one leg of the U was not detrimental, and that complete filling could be achieved.

This technique was adopted for the grouting of 70m tall U-shaped tendons on the Ninian platform in 1978.

Gerwick\textsuperscript{23} reports that, despite efforts to overcome the wick effect, small voids are still frequently found at the top of tall vertical ducts. Refilling these from the top is therefore required.

Additional care is also required, since, having vertical lengths in excess of 100m, high grouting pressures are required. In the construction of the Ekofisk barrier in 1988, this excessive pressure was blamed for the widespread laminar cracking that occurred in the walls. However, it is possible that thermal cracking from the hydration of the concrete was an additional factor.

More recently, thixotropic grouts have been used in vertical ducts to minimise problems of bleed and shrinkage.

A technique of vacuum-assisted grouting has been adopted on the Hibernia platform. The platform design included about 500 U-shaped ducts up to 104m tall. It was found to be most practical to grout these from the top. The duct was filled by gravity from a point 5m below one of the anchorages. In order to assist the process, the air in the duct was evacuated and the grout injected under pressure.

At the end of injection, a pressure of 0.4N/mm\textsuperscript{2} was maintained on the grout until the bleed water had stopped flowing from the ends of the strands and no more grout could be injected. These procedures were adopted after three separate trials to demonstrate each aspect of the process. The experience demonstrated the requirement for a fluid grout that needed to be as fresh as possible.
3.4.3 General Experience of Grouting

Based on experience up to 1976, the following observations were made:

- Maximum grouting pressures for vertical ducts could be as high as 3.5N/mm²
- Lower pressures were required to grout U tubes and, in consequence, there was less bleeding
- The procedure of holding the grout under pressure after filling the duct and before sealing seemed unnecessary. Later, ducts were grouted without this with no discernible effect
- Topping-up can be carried out within an hour of grouting. This can be done from a small header tank containing 10-15 litres of grout and positioned about 1m above the anchorage
- Trials should be carried out before grouting commences in order to prove the proposed technique. If re-injection is used, it should be carried out with a coloured grout and the duct cut up afterwards to determine the degree of penetration.

Increasingly, thixotropic grouts are used for the grouting of vertical tendons. However, there remains the possibility of voids forming at the upper anchorages and no standardised Quality Control methods exist for adequate inspection.

3.5 CORROSION PROTECTION

It is widely recognised that different corrosion zones exist for offshore structures, where the likelihood of corrosion depends on the local conditions. Three zones have been defined in the design codes (see section 4.2.2):

- in the splash zone, where the concrete is exposed intermittently to sea water, corrosion is considered to be at the highest risk. It is for this reason that a greater thickness of cover to reinforcement is specified in this zone. In addition, the concrete in the splash zone will be at the most risk from damage due to the erosive action of waves and floating debris, or from ship impacts
- in the submerged zone it is generally considered that the limited oxygen supply to the steel largely impedes the corrosion process. Experience from the inspection of the Tongue Sands Naval Fort (part of the Concrete in the Oceans programme) 23, suggests that corrosion is therefore not usually a problem.
- embedded steel in the atmospheric zone is generally considered to be at less risk than that in the splash zone. However, the atmospheric zone should still be considered as an aggressive environment by normal standards.

The importance of adequate corrosion protection systems for the embedded steel has always been well understood, although the details of corrosion mechanisms and protection may not have been as comprehensive as perhaps they are today. Therefore, the corrosion protection systems employed in the first offshore platforms are open to re-assessment in the light of more recent knowledge.

Corrosion protection has been provided on several levels, including simply providing a sufficient thickness of good quality cover concrete to prevent the ingress of chlorides to the steel. Therefore a full description of corrosion protection systems involves such factors as concrete mix design and specified cover to reinforcement.

In addition, there was early concern about stress corrosion and fatigue of the prestressing. This has been extensively covered in the report by Burdekin and Rothwell1 and in the 'Concrete in the Oceans' programme60.

3.5.1 Concrete Mix Design

In terms of corrosion protection, the objective of concrete mix design has been to provide a highly impermeable protective layer between the steel and the environment. This has implied the use of plasticizers and super-plasticizers to reduce the water/cement ratio, a high cement content and well graded aggregates.

Cement content has been in the region of 400-450kg/m³, and the water/cement ratio in the region of 0.40, although lower ratios have been achieved in more recent structures.
To achieve low permeability, surface cracks must also be limited, although these can result from the construction methods used and cannot be completely eliminated. Tension cracks have been controlled by the prestressing itself which maintains the concrete in compression.

Attention has also been given to limit thermal and shrinkage cracks due to the heat of hydration of the cement mix. This has potentially been a great problem in offshore platforms due to their size and wall thicknesses which are often in excess of 1m. Heat generation has been limited with the use of a cement replacement such as fly ash or silica fume. Additionally, for the Condeep structures, concrete surfaces have been sprayed with water to reduce the heat build up.

If compressive strengths may be taken as a measure of concrete mix design, development was particularly rapid during the first years of construction practice. For the first platforms, a 45N/mm² strength concrete was specified. By 1977, 50N/mm² was the norm, this rising to 60N/mm² by 1982. The most recent platforms have been built using 75N/mm² strength concrete.

A slight confusion over the behaviour of the C₃A content of cement in sea water was apparent in the years between 1973 and 1975, the time of construction of the first offshore platforms. There was originally concern that the C₃A would react with sulphates present in sea water. However, it is now known that C₃A combines with the chloride ion to form an insoluble compound that effectively reduces the concrete’s permeability. Therefore, a cement with a moderate C₃A content (typically about 8%) is now often specified. The cement supplied for the Sea Tank/McAlpine platforms had a C₃A content in the region of 12%, and the contractors tried to reduce this percentage by a 20% cement replacement with fly ash.

Cover to reinforcement has generally been of the order of 50mm, but 70mm in the splash zone, and cover to prestressing steel has generally been 100mm.

### 3.5.2 Cathodic Protection

In theory, steel properly covered by a good quality concrete has no need for cathodic protection. However embedded steel gains an electric potential of between 300 and 400mV with respect to exposed steel. Because of this, exposed steel that is electrically connected to embedded steel is more at risk from corrosion since the embedded steel could act as an extended cathode.

Exposed steel is typically protected by either an ‘impressed current’ or a ‘sacrificial anode’ cathodic protection system. However, the cathodic protection may interact with the embedded steel, and this needs to be considered at the design stage. Consumable anodes may be consumed more rapidly in these situations, necessitating early replacement.

In the past, two different approaches have been adopted. The first was to isolate the embedded steel from the exposed steel so that no interaction could take place, the second was to systematically interconnect the embedded and exposed steel and allow for current drain to the embedded steel in the design of the protection system.

The isolation technique implied that care was needed during construction so that attachments to exposed steel components never made contact with the reinforcement. This method offered the simplicity that interaction between the exposed and embedded steel was prevented, and was used on several of the earliest structures. However, this method is no longer used in the North Sea since, where isolation has been attempted, it has proved almost impossible to achieve. In theory, there is a possibility of localised corrosion where the insulation becomes impaired, although no cases have been reported.

The method of interconnection implied the provision of extra steel giving electrical continuity across all of the steel groups. Whilst generally being simpler to put in place, this system requires that the cathodic protection system is designed so that the entire structure is encompassed. This has now become the preferred method, and has been stipulated in the Health and Safety Executive Guidance Notes (see Section 4.2.2).
In land-based structures, there is a general nervousness about the use of cathodic protection for post-tensioned steel. There is a general lack of experience in the field and this is an area requiring more attention.

3.6 ADDITIONAL CORROSION PROTECTION

In addition to the above corrosion protection, the following measures have been adopted;

- the freshly slipformed concrete surface has been treated with epoxy resin, particularly in the splash-zone,
- particular care has been taken in the storage of tendons and metal ducting due to the aggressive marine environment, coating with soluble oil is in general usage,
- sherardized bars have been used to attach the skirt of the Ninian platform,

Most recently, individual greased strands in an HDPE sheath, and HDPE ducting has become available on the market. However, the use of unbonded strands is against the conventional wisdom for offshore structures. Bonded tendons have been preferred in the past since, should the tendon fail, the loss in prestress is limited to a 2 or 3m length rather than the entire strand. The replacement of tendons in offshore structures would not generally be feasible.

3.6.1 Corrosion Protection of Anchorages

The prestressing systems of prestressed concrete structures are generally most vulnerable at the anchorages. Particularly for vertical ducts, voids located immediately behind the anchorages have been a recurring problem. In offshore structures in general, anchorages are located in recesses which are filled after the tendons have been stressed. The discontinuity between the surrounding concrete and the fill can provide a line of weakness, allowing the penetration of chlorides.

The most common problem has been reported as the shrinkage of the filling material. This has led to the corrosion of some anchor plates, and the degradation of the grout fill due to expansive freeze-thaw action.

The most successful system has been to prime the recess surfaces with a suitable epoxy bonding compound, and to use a small aggregate concrete through a window box. An alternative has been to use latex modified concrete. Finally, an epoxy coating has been applied to the concrete surface around the fill.
4. DESIGN CODES FOR PRESTRESSED OFFSHORE STRUCTURES

4.1 INTRODUCTION

The design and construction of fixed offshore installations in the UK is currently governed by several documents. The sections of these documents relevant to the corrosion protection of prestressing tendons will be reviewed in this chapter.

The codes reviewed as part of this study may be listed as follows:

- Det Norske Veritas, Rules for the Classification of Fixed Offshore Installations, 1989
- Building Code Requirement for Reinforced Concrete, ACI 318M-96, United States of America
- Guide for the Design and Construction of Fixed Offshore Concrete Structures, ACI 357R-84, United States of America

The superseded codes of practice relevant to the UK sector and used for the early structures include:

- CP 110 Part 1, 1972. The Structural Use of Concrete, Code of Practice for Design and Construction
- Editions 1-3 of the Department of Energy Guidance Notes...
- Det Norske Veritas, Rules for the Design Construction and Inspection of Offshore Structures, 1977

4.2 CURRENT CODES OF PRACTICE IN THE UK SECTOR

The “Offshore Installations (Construction and Survey) Regulations 1974” (SI 1974/289) specified the framework by which offshore structures were to be inspected and assessed. In order to assist in the interpretation of these Regulations, the former Department of Energy published Guidance Notes; “Offshore Installations: Guidance on design, construction and certification”. These Guidance Notes, more recently maintained by the Health and Safety Executive, are relevant to this study since they provide the basis upon which Offshore Installations have been designed.

However, the “Offshore Installations (Construction and Survey) Regulations 1974” have been replaced by the “Offshore Installations and Wells (Design and Construction, etc.) Regulations 1996” subject to a transition period to June 1998. The Guidance Notes will be maintained through this period.

In the United Kingdom, concrete structures are normally designed in accordance with BS 8110, 1985: The Structural Use of Concrete. This Standard is intentionally broad in its
scope since it covers all forms of reinforced and prestressed concrete construction. However, it does not cover loading requirements, these being presented in other Standards such as BS 6399 for buildings, and bridge design, which is covered separately in BS 5400.

Design aspects and loading conditions specific to offshore structures were embodied in BS 6235, 1982: Code of Practice for Fixed Offshore Structures, but this was withdrawn in 1985 and has not been replaced.

Under the 1974 Regulations, six Certifying Authorities were appointed for the certification of fixed offshore installations. As a Certifying Authority, Det Norske Veritas have published their own set of rules concerning the design, construction and certification of offshore installations. The current rules are given in "Rules for the Classification of Fixed Offshore Installations", Det Norske Veritas, July 1989.

In addition to these documents, the Fédération Internationale de la Précontrainte (FIP) has published recommendations entitled the Design and Construction of Concrete Sea Structures39, currently in its fourth edition. These recommendations are internationally recognised as containing sound engineering practice.

To summarise, current practice for the design and construction of fixed offshore installations in the United Kingdom is governed by a range of different documents, each with a different purpose and therefore a different approach. A detailed comparison of these documents is therefore not possible, although a comparison of their content areas may be of some interest (see Table 4.1.).

The post-tensioning requirements as outlined in these documents will be described in the following sections.

4.2.1 British Standard BS 8110 Part 1, 1985

This Standard, covering most standard reinforced and prestressed concrete structures, is necessarily general in its scope. However, it has been argued that BS 8110 is not entirely suitable for the design of offshore structures, because it was written for smaller land-based structures30.

In addition, despite requirements for the design and corrosion protection of prestressing systems being included, these are couched in rather general terms to allow for the differences between construction types. No requirements are made for corrosion protection at anchorage zones.

Section 4 of BS 8110 covers the design and detailing of prestressed concrete following the limit state philosophy given in section 2. Cover to the prestressing tendons is given for a range of concrete grades and exposure conditions. However, the values given are not adequate for offshore platforms and have been modified in the Health and Safety Executive Guidance Notes.

Section 6 covers concrete mix design, testing and placement. The durability of the finished structure is highlighted as the basis for many of the rules given. These rules are directly applicable to offshore structures.

However, section 8 of BS 8110 is the most relevant to this review since it details procedures and specifications for prestressing systems. In this section rules are given for each of the various stages from the handling and storage of prestressing components, to installation, stressing and grouting.

The grouting requirements from section 8 are discussed in more detail in Chapter 5 where comparisons are made with specialised grouting codes. The requirements are summarised in Tables 5.1 to 5.3.
4.2.2 Health and Safety Executive (formerly Department of Energy), Offshore Installations: Guidance on Design, Construction and Certification

The Health and Safety Executive Guidance Notes refer to the Offshore Installations (Construction and Survey) Regulations 1974 (SI 1974/289). This has now been superseded by the Offshore Installations and Wells (Design and Construction, etc) Regulations 1996, guidance notes for which have been published. However, much of the information contained in the former Guidance Notes has not been replaced, and therefore it remains relevant to this study. The most relevant sections are 12 and 23.

Section 12 outlines corrosion protection related issues. In paragraph 12.2, corrosion zones are defined for concrete structures. These are the:

- **submerged zone**: the part of the structure below the splash zone
- **splash zone**: the part of the structure between the crest level of the 50-year (average) wave, superimposed on the highest astronomical tide, and the trough on the lowest astronomical tide
- **atmospheric zone**: the part of the structure above the splash zone.

These definitions comply with the FIP recommendations, and are used in the design process for the specification of minimum cover to reinforcement. These rules are given in section 23 discussed below.

Where cathodic protection is used for the protection of exposed steel parts, there is a requirement for the prestressing and the reinforcement to be electrically linked to the protected steel. This requirement is a more recent addition to the Guidance Notes, and has not always been adopted for the earliest structures (see Section 3.5.2).

Section 23 of the Notes outlines design aspects specific to the use of prestressed concrete in the construction of offshore platforms. This section relates primarily to fixed platforms, but the guidance should be followed as far as practical for floating structures. The requirements in this section are given in addition to the requirements of BS 8110.

Cover to reinforcement is given in Table 4.2 on page 35. Section 23, however allows a thinner cover to be used in the submerged zone at the discretion of the owner and where this is not detrimental to structural requirements.

The Notes do not specify increased cover for prestressing tendons since 'the resistance to corrosion protection of post-tensioned prestressing tendons is dependant on the achievement of complete grouting of the ducts...'. This is in contrast to the FIP Recommendations, where increased cover to tendons is given. It is also debatable whether the corrosion protection of tendons should be 'dependant on the complete grouting of ducts' since there are currently no adequate inspection methods to ensure that grouting has indeed been complete (see Chapter 6). This philosophy also stands in contrast to the view taken by the Concrete Society who specify a multi-level corrosion protection system.

Rigid or semi-rigid metal ducts with water-proof joints are specified in the Notes, in addition to the requirements in BS 8110.

The specification for grouting in the HSE Guidance Notes is identical to that contained in BS 8110, except that the re-absorption time of bleed water in retarded grouts is increased to 48 hours in the place of 24. In addition, there is a requirement for grouting trials for long vertical tendons and long horizontal ducts with extended intervals between vents.

No guidance is given for design detailing or corrosion protection at tendon anchorages.
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<td>Concrete mix specification</td>
<td>6.3</td>
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<td>2.3.1 C 100-500</td>
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<td>3.2.7 C 200</td>
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<td>8.3</td>
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<td>3.2.7 E 200</td>
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<td>3.2.7 E 200</td>
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<td>Post-Tensioning</td>
<td>8.7.5</td>
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<td>3.2.7 H 100</td>
</tr>
<tr>
<td>Protection of prestressing tendons</td>
<td>8.8</td>
<td></td>
<td></td>
<td>3.2.7 J 100</td>
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<tr>
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<td>3.2.7 H 200</td>
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</tr>
<tr>
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<tr>
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<td>8.9.3</td>
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</tr>
<tr>
<td>Frequency of tests</td>
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<td></td>
<td>3.2.7 C 400</td>
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<td>Grout composition</td>
<td>8.9.4</td>
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<td>2.3.1 D 200</td>
<td>3.2.8</td>
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<tr>
<td>Batching and mixing of grout</td>
<td>8.9.5</td>
<td></td>
<td>3.2.7 F 100</td>
<td></td>
</tr>
<tr>
<td>Grouting procedures and trials</td>
<td>8.9.6</td>
<td></td>
<td>23.4.11 c)</td>
<td></td>
</tr>
<tr>
<td>Blockages and Breakdown</td>
<td>8.9.7</td>
<td></td>
<td>3.2.7 H 200</td>
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<tr>
<td>Maintenance and safety</td>
<td>8.9.8</td>
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<td></td>
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<tr>
<td>Grouting during cold weather</td>
<td>8.9.9</td>
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<td></td>
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<tr>
<td>Precautions after grouting</td>
<td>8.9.10</td>
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<tr>
<td>Checking the effectiveness of grout</td>
<td>8.9.11</td>
<td></td>
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<td></td>
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<td>Cathodic Protection</td>
<td></td>
<td></td>
<td>12.3.4</td>
<td>4.7.4</td>
</tr>
</tbody>
</table>

Note: The guidance in the HSE document is in addition to that in BS 8110 Part 1 1985.
4.2.3 Det Norske Veritas, Rules for the Classification of Fixed Offshore Installations

The Rules for the Classification of Fixed Offshore Installations were written for application where Det Norske Veritas was the Certifying Authority for the structure. The Rules have been used in addition to the HSE Guidance notes, and are now generally considered as the international standard for the design of offshore platforms.

The current Rules are a complete revision of the 1977 Rules for the Design, Construction and Inspection of Offshore Structures, and include structural design aspects closely related to those of the Norwegian land-based concrete code NS 3473. A comparison of these rules with the British codes has been made elsewhere.

The sections of the Rules most relevant to this report are contained in Part 3: Structures, Chapter 2: Fabrication and Construction, Section 7: Construction Requirements, Concrete Structures. These sections are primarily concerned with the Quality Control and Quality Assurance methods to be applied throughout construction, and are summarised in the following paragraphs.

During construction, the properties of grout are to be tested at the following frequencies:

- strength: tested once per shift, or once per 100m³ or whenever the constituent materials are changed
- initial and final setting time: three times per shift, or once per strength sample
- expansion and bleeding: every three hours or with each strength sample
- viscosity: every three hours or with each strength sample
- density: every three hours or with each strength sample

Testing is to be performed on samples taken from the mixer and at duct outlets whenever this is possible. Records of tests are to be maintained. Grout production is to be surveyed to ensure that the approved mix design and mixing procedures are complied with, and that the above sampling and testing intervals are used. The approved mix designs and procedures are established before the start of the construction phase.

The grouting procedure is to contain as a minimum:

- requirements for the fresh and hardened grout properties
- batching and mixing requirements
- transportation methods of the fresh grout
- grouting pressure
- holding time
- the number and position of vents
- detailed procedures for difficult operations such as the grouting of tall vertical ducts
- grout sampling points
- contingency measures in case of equipment failure, or blockages, etc.

All of the documentation resulting from these procedures is compiled in the construction records, used during the in-service life of the structure as an aid to the inspection and maintenance procedures.

4.3 OTHER CODES

In respect to grouting procedures and corrosion protection of prestressing tendons, little additional guidance is offered by the other codes produced outside Europe. In particular, the American ACI 357R-84 suggests the use of 'specialist literature'.

The most information is offered by the Canadian offshore code, CSA/CAN S474-94, although it is still incomplete when compared to specialist land-based codes. However, of note is clause 11.19.3, relating to the protection of tendon anchorages:
'Anchorages shall be recessed so that the minimum cover to any tendon or anchor part, after tendons are cut, is 100mm. The concrete mortar used in recesses shall be non-shrink and mechanically bonded to the structure with the surface sealed against the entry of water.'

The cover to conventional reinforcement and prestressing steel as required by the various international codes is give in Tables 4.2 and 4.3.

<table>
<thead>
<tr>
<th>Corrosion Zone</th>
<th>HSE, Offshore Installations: Guidance...</th>
<th>ACI 357R-84 Guide for... Fixed Offshore Concrete Structures</th>
<th>CSA S474-94 Part IV, Concrete Structures</th>
<th>FIP Design and Construction of Concrete Sea Structures 4th Edition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atmospheric Zone</td>
<td>55mm</td>
<td>50mm</td>
<td>65mm</td>
<td>65mm</td>
</tr>
<tr>
<td>Splash Zone</td>
<td>70mm</td>
<td>65mm</td>
<td>65mm</td>
<td>65mm</td>
</tr>
<tr>
<td>Submerged Zone</td>
<td>45mm</td>
<td>50mm</td>
<td>55mm</td>
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</table>

<table>
<thead>
<tr>
<th>Corrosion Zone</th>
<th>HSE, Offshore Installations: Guidance...</th>
<th>ACI 357R-84 Guide for... Fixed Offshore Concrete Structures</th>
<th>CSA S474-94 Part IV, Concrete Structures</th>
<th>FIP Design and Construction of Concrete Sea Structures 4th Edition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atmospheric Zone</td>
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<td>75mm</td>
<td>90mm</td>
<td>90mm</td>
</tr>
<tr>
<td>Splash Zone</td>
<td>70mm</td>
<td>90mm</td>
<td>90mm</td>
<td>90mm</td>
</tr>
<tr>
<td>Submerged Zone</td>
<td>45mm</td>
<td>75mm</td>
<td>75mm</td>
<td>75mm</td>
</tr>
</tbody>
</table>
5. GROUTING SPECIFICATIONS

5.1 INTRODUCTION

Grout within prestressing ducts fills the dual role of:

- protecting the steel tendon from corrosion,
- bonding the tendon to the concrete member throughout its length to provide structural continuity.

These roles will only be adequately fulfilled using cement grouts of the required characteristics injected so that voids are not formed within the duct. Recent investigations of land-based prestressed concrete structures\textsuperscript{35,36} have shown that grout injection has not always been satisfactory. The reasons for this relate to variable grout quality, inadequate duct design and inadequate grout injection procedures.

In setting out the requirements for site grouting practices, national specifications play a major role in ensuring the correction of these faults. These requirements are summarised in the following sections.

5.2 GROUT COMPOSITION

Standard grouts normally consist of Portland cement (Pc), water and an admixture or compatible admixtures. The requirements of differing national specifications are outlined in Table 5.1.

5.2.1 Cement

Most of the cements specified in the codes are general purpose cements that meet national standards. BS 8110 specifies the use of Portland Cement that complies to BS 12. However, BS 12 permits a wide range of cement compositions, and it is not possible to maintain similar grout rheology for cements from different works. This situation is common experience throughout the world.

It has been shown by Jefferis and Forrester\textsuperscript{37} that the water requirement for a given fluidity increases significantly with the C\textsubscript{3}A content and the alkali content as well as the particle size distribution. In practice, this means that mix proportions of the grout need to be adjusted each time a different source of cement is used.

Several of the European grouting codes set limits on the chloride content of cement. Typically this is limited to 0.1% by weight. Some Specifications also set limits as to the age of the cement to be used.

5.2.2 Water

Generally, the water content should be high enough to obtain the necessary fluidity of the fresh grout, but low enough to limit bleeding and shrinkage problems. Typical water/cement ratios are specified between 0.33 and 0.45. The lower ratios are for special grouts in the Concrete Society\textsuperscript{4} and the OMTA specifications\textsuperscript{35}. Both of these grout mixes include the use of a superplasticising admixture to maintain the grout's plastic properties.

In addition to the water/cement ratio, the FIP Guide to Good Practice\textsuperscript{39} sets guidelines for water quality. Water should be free from organic products and the chloride ion content should not exceed 500mg/l. Several European codes currently impose similar limits, although drinking water is normally assumed to comply.
<table>
<thead>
<tr>
<th>Specification</th>
<th>Cement</th>
<th>Water</th>
<th>Admixture</th>
<th>Aluminium Powder</th>
<th>Silica Fume</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>European Standard EN 447:1996</td>
<td>Pc to ENV 197-1 Type CEM-I</td>
<td>w/c: 0.44 max</td>
<td>To EN 934-4, singly or in combination</td>
<td></td>
<td></td>
<td>Fly-ash permitted, max. 5% carbon and</td>
</tr>
<tr>
<td>FIP Guide, 1990</td>
<td>Pc Blast furnace cement permissible. 0.1% chlorides maximum</td>
<td>w/c: 0.40 max</td>
<td>Permitted.</td>
<td></td>
<td></td>
<td>3% sulphur trioxide. Fine sand for large</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&lt;500mg/l Cl⁻</td>
<td>Limited use of glycol or methanol as anti-freeze permissible</td>
<td></td>
<td></td>
<td>ducts. Total aggregates &lt;30-40% wt of cement</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No organic substances</td>
<td></td>
<td></td>
<td></td>
<td>Sand and fly ash permitted for ducts &gt;150mm diam. Max 30% wt. of cement</td>
</tr>
<tr>
<td>British Standard BS 8110: 1985, Part 1</td>
<td>Pc to BS12</td>
<td>w/c: 0.44 max</td>
<td>Permitted.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Chlorides, nitrates and sulphates not permitted</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Society Technical Report no.47, 1996, UK</td>
<td>Pc to BS12 Class 42.5N 0.1% chlorides maximum</td>
<td>common grout: w/c: 0.40 special grout w/c: 0.35</td>
<td>Permitted. Total admixture ≤5% wt cement. Thiocyanate, nitrates, formates, chlorides and sulphides not permitted</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DIN 4227, Part 5 December 1979, Germany</td>
<td>Pc to DIN 1164, Part 1. Not more than three weeks old</td>
<td>w/c: 0.44 max</td>
<td>Permitted by approval To DIN 4227 part 1.</td>
<td></td>
<td>Not permitted</td>
<td>Aggregates permitted by approval, to DIN 4226 Part 1.</td>
</tr>
<tr>
<td>Specification</td>
<td>Cement</td>
<td>Water</td>
<td>Admixture</td>
<td>Aluminium Powder</td>
<td>Silica Fume</td>
<td>Other</td>
</tr>
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<td>---------------------------------------------------------------------------</td>
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<td>----------------------------------------------------------------------</td>
</tr>
<tr>
<td>Fascicle 65-A</td>
<td>Pc, CPA class 45 or 55 to NFP 15-301. 0.05% chlorides maximum</td>
<td>&lt;500mg/l Cl&lt;sup&gt;-&lt;/sup&gt; &lt;400mg/l SO&lt;sub&gt;4&lt;/sub&gt;²&lt;sup&gt;-&lt;/sup&gt; No detergent</td>
<td>Total admixture ≤3% wt cement. &lt;0.1% wt chlorides &lt;0.2% wt sulphates Thiocyanate, nitrates, formates and sulphides not permitted</td>
<td>Maximum 8% replacement of cement</td>
<td>Total mix: ≤0.1% wt Cl&lt;sup&gt;-&lt;/sup&gt; ≤0.1% wt SO&lt;sub&gt;4&lt;/sub&gt;²&lt;sup&gt;-&lt;/sup&gt; traces of S&lt;sup&gt;-&lt;/sup&gt;²</td>
<td></td>
</tr>
<tr>
<td>NEN 6722 Netherlands</td>
<td>Pc to NEN 3550</td>
<td>To NEN 5995</td>
<td>Permitted by approval No chlorides, nitrates or sulphates.</td>
<td></td>
<td></td>
<td>Fine sand permitted for ducts &gt;150mm diam. Total aggregates &lt;2.5% wt cement. Fly ash not recommended.</td>
</tr>
<tr>
<td>PTI Draft Spec November 1994, USA</td>
<td>Type Ii Pc</td>
<td>w/c: 0.35-0.45 Established by tests</td>
<td>Permitted (inorganic)</td>
<td></td>
<td>15-25% replacement wt cement</td>
<td>Pre-mixed dry 25kg bags</td>
</tr>
<tr>
<td>OMTC Draft June 1993, Canada</td>
<td>Type 10 OPSS 1301</td>
<td>w/c: 0.33</td>
<td>To OPSS 1303. Naphthalene sulphonate superplasticiser 1.5% wt cement</td>
<td>ASTM-D-962-81 Type 1, Class C 0.0067% wt cement 2.5-3.0% stearic acid coating</td>
<td>CAN 3-A23.5-M86 6% replacement by wt cement</td>
<td></td>
</tr>
<tr>
<td>CIA Recommendations, 993, Australia</td>
<td>Pc to AS 1315. Max: 21 days old.</td>
<td>0.4 to 0.45</td>
<td>To AS 1478</td>
<td></td>
<td></td>
<td>Chlorides ≤750mg/l of grout. Fly ash and fine sand not recommended.</td>
</tr>
</tbody>
</table>
5.2.3 Admixtures

Although traditionally, grouts have been simple cement-water pastes, admixtures are increasingly used in grout composition to improve the performance characteristics of the mix. Admixtures commonly used in grouts are plasticisers and retarding agents. They are used to facilitate the grouting process by:

- improving grout flow for a given w/c ratio
- reducing the amount of bleed water
- preventing segregation of the grout under pressure
- delaying the setting of the grout
- reducing the grout shrinkage.

Many standards require that admixtures conform to national codes or the use of tests to verify their effectiveness. The FIP also suggests the limited use of methanol as an anti-freeze in low temperatures, although this reduces the strength of the hardened grout.

Portland cement grout shrinks as it cures and ages; this shrinkage increases with the water/cement ratio. Cracks can occur when shrinkage induced stress exceeds the tensile strength of the grout. Expansive admixtures may be used to help eliminate this shrinkage and reduce the formation of voids in ducts resulting from grout segregation and settlement.

Small quantities of aluminium powder are commonly used as an expansive admixture, since this reacts with the alkalis in the cement, producing hydrogen gas. The American codes allow the use of aluminium powder in sufficient quantities to produce a 5% unrestrained expansion of the grout. However the older German code DIN 4227 1980 and the CIA recommendations (June 1982) do not permit its use due to the fear that the liberated hydrogen could encourage hydrogen embrittlement of the tendons.

An addendum to the CIA recommendations dated January 1993 admits that 'it is now widely accepted that aluminium powder... does not [promote hydrogen embrittlement] under normal circumstances'. However, pending further tests, the CIA still does not recommend its use.

The OMTC prescribes the use of 0.0067% aluminium powder by weight of cement, enough to produce up to 8% unconfined expansion of the grout. The powder is to be coated in stearic acid which would have the effect of delaying the evolution of the hydrogen gas. This is the only code to specify a commercial product.

Expansive admixtures may also be used for air-entrainment to improve the freeze resistance of the grout during the setting period.

5.2.4 Aggregates and Cement Replacements

Generally the use of aggregates in grout is limited to special projects where the duct internal diameter is very large. Fine sand has sometimes been used, and where mentioned in the specifications, its use is limited to ducts above 150mm in diameter. The CIA do not recommend its use.

The use of silica fume in grouts as a partial replacement for cement has been the subject of much research. Silica fume is a waste by-product from the production of silicon, and is therefore not a well-defined material. Consequently, it is important to document the particular properties of the fume from a given source. It is now generally recognised that the high surface area of the fume and its high water absorption reduce the amount of bleed water from fresh grout. In addition, its rounded particle shape improves grout fluidity when used in conjunction with superplasticizers. However, without plasticizers, the grout displays increased viscosities.

These advantages are carried through to the hardened grout since the fume reacts with the calcium hydroxide content of the cement producing a dense, strong grout. However, there has been concern that this reaction could reduce the alkali content of the paste, hence removing its natural buffer to tendon corrosion.
The FIP cautiously recommends the use of silica fume pending further research or suitability tests. In the United States, the PTI prescribes a 15% to 25% replacement of cement by silica fume, although research carried out by the OMTC\(^4\) found 6% replacement to be optimal.

It has also been noted that the use of silica fume necessitates a more careful selection of superplasticizer and tight batching controls.

Fly ash may be used in a similar manner to silica fume but generally with less positive results. It develops strength more slowly than pure Portland cement, and there is also concern as to the impurities it may contain. Its use is disallowed in several countries, but it is not prohibited by BS 8110.

### 5.2.5 Pre-bagged Pre-mixed Grout

It has become recognised that the performance of grout can be variable for two reasons:

- batching on site is a difficult task to carry out accurately,
- different supplies of cement, even from the same manufacturer can give significantly different properties to the grout.

In consequence of this, there is a move towards using grout that is pre-mixed and pre-bagged in the factory where better quality control can be exerted and properties of the resulting grout are guaranteed. In the Concrete Society Specification\(^6\), grout is classified as common (mixed on site) or special (pre-mixed, pre-bagged). It is pointed out that, not only are cement properties variable, but it is permitted under British Standards for the weight of a bag to vary by up to 6% from its nominal weight.

Development of a pre-mixed ‘new generation’ grout for the Ontario Ministry of Transportation (OMTC), is described by Ip and Manning\(^5\). In the past, the OMTC required the use of a standard grout but there were concerns that it offered limited corrosion protection with a lack of quality control during on-site mixing. After various trials, a grout was developed which was composed of Type 1 cement, silica fume (6% cement replacement), superplasticizer (1.5% non-retarded naphthalene sulphonate by weight of cement) and aluminium powder (coated, 0.0067% by weight of cement), with a water/cement ratio of 0.33. In trials it was found that compressive strengths were 50 to 58N/mm\(^2\) compared to 12 to 49N/mm\(^2\) for grout made to the old standard. Expansion was 6% falling to 5% for grout made from pre-mixed material stored for three months. There was zero bleed. It was recommended that the pre-mixed pre-bagged product should not be used after a storage life of one month.

A pre-mixed pre-bagged grout has also been developed in the UK as part of a joint industry Department of Transport LINK project as described by Tilly and Woodward\(^6\). The formulation was developed from a series of laboratory and field trials. During this work, it was found that the presence of aluminium powder (to generate hydrogen and expand the grout) caused the formation of ‘eyelet’-shaped voids. As these constituted stress concentrations that could lead to the development of cracking and reduced corrosion protection, Azo chemicals were substituted for the aluminium powder. The composition of the grout was Pe 42.5 cement, micro-silica and other admixtures to control sedimentation and bleed (approximately 4% by mass) superplasticizer, antifoam, retarder and Azo (0.02% by mass) with an equivalent water/cement ratio of 0.35. It was found that the compressive strength was 52N/mm\(^2\), expansion was 0.3% and there was zero bleed. The expansive gases (ammonia) generated by Azo, caused the development of finely dispersed small voids that were more acceptable than the larger eyelet developed by aluminium powder. In addition to the common grout tests, sedimentation was measured and values confirmed to be acceptable according to the Concrete Society Specification\(^4\).
5.3 GROUT PERFORMANCE CRITERIA

Grout performance criteria relate to the physical properties of the grout in both its fresh and hardened state. Observed characteristics of the liquid grout relate to its fluidity, expansion and bleeding or sedimentation, while the characteristics important to the hardened grout are strength and permeability. The performance criteria demanded by the differing national standards and specifications are outlined in Table 5.2.

The DnV Rules require that the properties of the proposed grout are documented sufficiently in advance of the grouting works to enable the composition to be changed. Often the properties of the grout may be well known from previous experience, otherwise tests are universally prescribed. Tests are also required for the fresh grout taken from the grout holding tank and issuing from the duct outlet.

5.3.1 Compressive strength

Compressive strengths are specified in all codes, usually at 7 days and 28 days. The 7 day values range from 20N/mm² in the FIP guide to 27N/mm² in the Concrete Society Specification. However, the values given in Table 5.2 relate to the type of test (50mm cube, 100mm cube, cylinder) and correction should be made to make due allowance for the different conditions. The values specified usually relate to the average value of three tested samples.

Although compressive strengths are required routinely, there is a general lack of understanding of their purpose. It is suggested that this is two-fold

- to test the variability of the grout (few codes specify an accepted range of values),
- to ensure that a minimum strength is achieved, compatible with requirements of transference of load from tendon to the concrete member.

In addition to compressive strength tests, the French Specifications require that the bending resistance of the hardened grout is also measured at 28 days. The minimum average value for three samples is 4N/mm².

5.3.2 Flow

Grout fluidity measurements are generally required. The FIP calls for tests but provides no guidance as to expected test values. The codes specify either the flow cone method (the Marsh cone or variants) or the German Immersion Test.

However, these tests are not suitable for thixotropic grouts that can sometimes give no useful results. It is therefore suggested by the CIA that tests like the flow cone do not give absolute results as to the grout's suitability, but provide a useful comparison between different mixes.

A considerable amount of work has been carried out to develop flow tests that describe the actual flow behaviour of grouts within a duct. To be meaningful flow tests should be representative of the flow conditions of grout in the duct. Parameters which may influence grouting are injection rate, duct and tendon diameters, grout shear rate, the formation of lubricating layers at solid surfaces, and the rheological properties of the grout (Reynolds number).

The flow of grout in a duct is a complex process and has been described by Jeffries and Forrester. The flow may be horizontal, vertical, or inclined upward or downward. The space to be grouted is unlikely to be axisymmetric; in general the tendons will be eccentric and this eccentricity will vary along the duct. The grout also has to penetrate into the tendon bundle. The duct diameter and the proportion of the cross-section occupied by tendons will vary between ducts. Furthermore, contractors may have their own particular mixing, agitation and injection equipment and the choice of flow rate may be left to the site operator (if it is not fixed by the equipment).

Most of the specifications studied in this chapter require that the grout advance rate in the duct should be in the range of 5 to 15m/min. In practice grouting rates actually achieved on site are more variable. Values of 13 to 22m/min were recorded at a construction site by
Woodward and 1.4 to 2.7m/min in UK grouting trials (Tilly). The latter relate to the time from starting injection to closing off the pump, so that the actual speed of the grout front was probably faster if allowance is made for interruptions due to venting operations.

At any solid boundary, the local liquid content of a grout must be higher than in the bulk suspension since particles will be forced away from the boundary (the centre of a particle of diameter d, cannot approach closer than d/2 to a wall). In addition to this it appears that some segregation of the liquid and solid phases of a grout occurs in the wall region. The effect of this is to produce a more fluid layer, perhaps comparable to the lattece which can develop when a concrete surface is trowelled. In trials, when the duct has been cut away from the hardened grout, it has been observed that there is a 'dusting' of white powder on the grout surface. This was gypsum separated from the body of the grout.

The flow of grout suspensions appears to be strongly influenced by this wall layer. The situation is further complicated by the fact that in practice, a new wall layer is created continuously as the grout front advances along the duct, whereas in test equipment (especially rotational viscometers), the full wall layer may be established before any results are recorded.

Another factor influencing flow is shear rate in the suspension. For a Newtonian fluid flowing in a circular pipe, the average shear rate is given by 8v/d where v is the average velocity and d is the pipe diameter. For an annulus the equivalent shear rate may be approximated by 12v/(d_i - d_o) where (d_i - d_o) is the annular gap.

If the grout velocity in the duct is to be in the range 6 to 22 m/minute then, for empty ducts ranging in diameter from 75mm to 150mm, the extreme range of shear rates will be 5 to 40 s⁻¹. For a similar range of ducts with 50% of the cross-section occupied by tendons the range of shear rates will be 18 to 200 s⁻¹. Thus the shear rate during continuous injection into a duct might be in the range 5 s⁻¹ (injection into an effectively empty 150mm duct) to 200 s⁻¹ (injection into a 75mm duct with 50% of the area occupied by tendons). These are very much guideline figures and take no account of the non-Newtonian nature of the grout.

In practice, rates may be rather different, but would be expected to be within the same order of magnitude. The above analysis takes no account of tendon eccentricity or grout penetration into interstices of the bundle of strands. In both these situations, the grout may have to penetrate narrow spaces and thus, for a constant flow velocity (i.e. uniform advance along the duct) a higher shear rate would be developed. This may occur but it should be remembered that a higher shear rate will require a higher pressure gradient (shear stress) and this may not be available within the flow regime.

The flow test used for a particular grout must take into account the rheological properties of that grout, since there are at least three different categories of grout depending on fluidity:

- grouts that remain fluid throughout the specified time period,
- grouts that are thixotropic,
- grouts that are initially pourable but which display false set (see the test method in Section 5.5.4).

A wide range of equipment is available for testing the flow of grouts but, at present, no one test is entirely adapted to all of the categories of grout.

5.3.3 Bleed

Limits on grout bleed are specified in all codes. Sedimentation and bleeding can lead to the risk of voids in the duct as the solid particle content of the grout settles. With the hydration of the cement, most of this bleed water will be re-absorbed back into the grout. After 3 hours of hydration, the grout will have almost achieved its final volume change due to bleed. Therefore, most of the specifications limit 3 hour bleed values to a maximum of 2%. So that full hydration of the cement may be accomplished, the bleed water should be re-absorbed within 24 hours. This has been made a requirement by almost all codes, but it is not clear how this originated nor by what mechanism re-absorption takes place. It has been suggested that re-absorption may take place via micro-cracks in the grout.
<table>
<thead>
<tr>
<th>Specification</th>
<th>Compressive Strength</th>
<th>Flow</th>
<th>Bleed</th>
<th>Volume Change</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>European Standards EN 445 and 447: 1996</td>
<td>Specimen: 40x40x160 prism or: 100 diam. x 80 cylinder or: up to 100 mm cube</td>
<td>Min: 3 tests/8 hrs Immersion test: 30 secs min. after mixing 80 secs max. after 30 mins 30 secs min. at outlet</td>
<td>Min: 2 tests/day 2% max after 3 hrs</td>
<td>Min: 1 test/day -1 to 5% or 0 to 5% when expansive admixtures used</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7 day: 27 N/mm² 28 day: 30 N/mm²</td>
<td>CEN flow cone test 25 secs max. after mixing 25 secs max. after 30 mins 10 secs min. at outlet</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FIP Guide, 1990</td>
<td>Specimen: 50 to 100 mm cubes or: 50 to 100 mm cylinders where diam. = height</td>
<td>To be tested (no figures given)</td>
<td>2% max after 3 hrs Reabsorbed in 24 hrs</td>
<td></td>
<td>-2 to +5%</td>
</tr>
<tr>
<td></td>
<td>7 day: 20 N/mm² minimum 28 day: 30 N/mm² minimum</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>British Standard BS 8110: 1985</td>
<td>100 mm cubes test to BS1881</td>
<td>CEN flow cone test: 12-25 secs after mixing and at outlet</td>
<td>2% max after 3 hrs 4% max overall. Reabsorbed in 24 hrs</td>
<td></td>
<td>10% max unconfined expansion</td>
</tr>
<tr>
<td></td>
<td>28 day: 30 N/mm² minimum</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Society Report 47, 1996, UK (a) common grout</td>
<td>100 mm cube test: BS 1881 Min: 2 tests/day 7 day: 27 N/mm² minimum</td>
<td>Min: 2 tests/duct CEN flow cone test: 25 secs max. at mixing, at end of injection and after 30 mins 10 secs min. at outlet</td>
<td>Min: 2 tests/day</td>
<td>Min: 2 tests/day</td>
<td>(a) -1 to 5%</td>
</tr>
<tr>
<td></td>
<td>(b) special grout</td>
<td>(a) 2% maximum. Average value for 4 tests &lt;1% (b) 0</td>
<td></td>
<td>(a) 0 (b) 0 to 5%</td>
<td></td>
</tr>
<tr>
<td>Specification</td>
<td>Compressive Strength</td>
<td>Flow</td>
<td>Bleed</td>
<td>Volume Change</td>
<td>Other</td>
</tr>
<tr>
<td>------------------------</td>
<td>---------------------------------------------------------------------------------------</td>
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<td>--------------------------------------------</td>
<td>----------------------------------------</td>
<td>----------------------------------------------------------------------</td>
</tr>
<tr>
<td>DIN 4227, Part 5,</td>
<td>Specimen: 99mm diam x 80mm cylinders 28 day: 30N/mm² min No single value below</td>
<td>Immersion test: 30 secs</td>
<td>-2% maximum contraction</td>
<td>No water to remain on grout sample</td>
<td>28 day bending resistance: 4N/mm² minimum</td>
</tr>
<tr>
<td>December 1979,</td>
<td>27N/mm²</td>
<td>min. after mixing 80 secs</td>
<td>max. after 30 mins</td>
<td>after 28 days.</td>
<td>(40x40x160mm sample) Capillary absorption: &lt;1g/cm² 24 hrs maximum setting time</td>
</tr>
<tr>
<td>Germany</td>
<td>28 day: 30N/mm² minimum</td>
<td>max. after 30 mins</td>
<td>max. after 3 hrs reabsorbed in 24 hrs</td>
<td>36 hrs maximum setting time at 2°C</td>
<td>3 to 12 hours setting time Permeability test: 3000 Coulombs maximum</td>
</tr>
<tr>
<td>Fascicule 65-A,</td>
<td>Specimen: 40x40x80mm prism (half sample from bending tests) 28 day: 30N/mm² minimum</td>
<td>ASTM flow cone test: 13</td>
<td>10% maximum unconfined expansion</td>
<td>3 per 100 bags of grout to be inspected by Ministry representative for evidence of age, dampness or hard lapses</td>
<td></td>
</tr>
<tr>
<td>January 1992, France</td>
<td>7 day: 20N/mm² minimum</td>
<td>secs min. at mixing 25</td>
<td>max. after mixing 2% max after 3 hrs</td>
<td>6% expansion ± 2%</td>
<td>Relative density 1.85 to 2.0 Settlement: 10mm maximum</td>
</tr>
<tr>
<td>NEN 6722, Netherlands</td>
<td>70mm cubes, hardened 7 days at 20°C 7 day: 20N/mm² minimum</td>
<td>secs max. after 30 mins</td>
<td>reabsorbed in 24 hrs</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PTI Draft Specification</td>
<td>Specimen: 50mm cube 1 day: 10.3N/mm² min 7 day: 20.7N/mm² min 28 day: 27.6N/mm² min</td>
<td>ASTM C939 flow cone 20-30</td>
<td>0 to +5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>November 1994, USA</td>
<td>7 day: 20.7N/mm² min 28 day: 27.6N/mm² min</td>
<td>secs Pressurised flow test</td>
<td>max. after mixing 1% max. reabsorbed in 24 hrs pH of bleed water &gt;11.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OMTC Draft June 1993,</td>
<td>50 mm cube tested at 7 day and 28 days. (values not specified but nominally 35N/mm²</td>
<td>ASTM flow cone test 10-16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Canada</td>
<td>minimum at 28 days)</td>
<td>secs after mixing and 30 mins later Test to CAN3-A23</td>
<td>max. after mixing 6% expansion ± 2%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CIA</td>
<td>Specimen: 100mm cubes 7 day: 25N/mm² minimum 28 day: 40N/mm² minimum</td>
<td>Flow cone test recommended, values not given</td>
<td>1% max at 3 hrs</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Recommendations 1993,</td>
<td></td>
<td></td>
<td>Total ≤ 2% reabsorbed in 24 hrs</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Australia</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>10% maximum unconfined expansion</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5.3.4 Volume Change

The CIA, the Netherlands and BS 8110 specify an upper limit of 10% unrestrained expansion with the use of admixtures. However, most standards limit this to 5%. The largest allowable contraction specified is 2% in the FIP Guidance notes.

5.3.5 Set Time

The French and Dutch codes specify a set time for the grout. The lower limit of 3 hours for common grout is to ensure that the grouting procedure may be completed successfully in high temperatures (30°C) without the risk of blockages. The upper limits of 24 and 36 hours are to ensure that the grout does not risk freezing while still in the fresh state at low temperatures (at 5°C and 2°C respectively).

5.3.6 Density

The CIA recommends that the relative density of the grout is also measured. These measurements directly relate to the water/cement ratio of the grout. The recommended limits of 1.85 to 2.0 for the relative density correspond to water/cement ratios of 0.4 to 0.45.

5.3.7 Segregation

The Concrete Society Specification calls for a segregation test. The difference in density between the top and bottom quartile sections should not exceed 10%.

5.4 GROUTING PROCEDURES

5.4.1 Basic Procedures

Grout composition is controlled by adding a fixed number of bags of cement (or pre-bagged mix) to a measured quantity of water. The volume of water is usually controlled using a tank with an overflow pipe which is adjusted to the required height prior to grouting. Admixtures, when used, are normally added by volumetric measure.

The water is added to the mixer, followed by the cement and admixtures, which may either be added as a whole or in part in sequence until the total quantities are added. Maximum mixing times are typically specified as 2 minutes for a high-speed colloidal mixer and 4 minutes for a vane mixer. Longer mixing times increase the risk of trapping air in the grout.

After mixing, the grout is transferred to a holding pan where it is slowly stirred. Samples are usually taken from here and tested for fluidity before the injection of the ducts begins. It is often specified that the grout is to be retained in the holding pan until a sufficient quantity is available to fill the duct to be injected. This requirement reduces the risk of the holding pan running low so that air is pumped into the duct.

Tests to confirm the performance of the grout are usually required a few days before the grouting operation and during the operation. The former tests are carried out under laboratory conditions, although it is often required that the grout mixing take place on site using the actual equipment and personnel to be used for the main grouting operation. This is a condition of the European Standard EN 446:1996.

Although grouting procedures are similar in most countries, they differ in detail. These include restrictions on temperature, batching accuracy, mixing order, injection pressures, pumping rates and procedures for post-injection of grout as shown in Table 5.3.
Prior to grouting, all of the duct vents, the inlet and outlet should be closed to prevent the ingress of water. However, it is still a common practice to fill ducts with an anti-freeze mix in the case of cold weather to protect the inside of the structure from the formation of ice. An alternative is to blow out the duct with dry, oil-free compressed air.

The pre-wetting of lined ducts is generally unnecessary and is not a recommended practice. On the other hand, several specifications require the duct to be thoroughly cleaned up to a week before grouting begins. This would usually be by flushing it out with water, followed by a blast of compressed air to clear out any that remains.

In addition, the French specifications (F65-A) allows for the ducts to be pressure tested with water at 0.3N/mm² to test for leaks. This is carried out where specified in the main contract. The Concrete Society specification requires that ducts are pressure tested with air at 0.1N/mm², the pressure not dropping by more than 50% over 5 minutes.

All of the duct vents, the inlet and outlet should be opened immediately before grouting is to begin. Grout injection then begins from the lower end of the duct towards the other. As the grout issues from the duct vents, it is collected until it has the same consistency as the injected grout. The vent is then closed. In the case of intermediate crests, the vents immediately downstream are closed immediately before the crest vent. This procedure is continued in one operation until the duct is filled and the grout at the outlet visibly has the correct flow characteristics.

In the case of the Concrete Society Specification, a further 5 litres of grout should then be pumped through each vent in the direction of grout flow.

The Concrete Society further specifies that the duct then be locked off under 0.5N/mm² pressure for one minute. Under EN 446, this would be optional in order to prevent the loss of grout from the duct.

Injection pressures are not usually specified in European codes, although it is always specified that the pump be capable of maintaining a constant pressure of up to 1N/mm², with safety valves to prevent pressures in excess of 2N/mm². The Euro Norme remarks that limiting the injection pressure serves to:

- prevent blow-outs of hoses, inlets and outlets
- prevent spalling of the concrete near the duct
- protect the equipment and operators
- prevent segregation of the grout
- control the grout flow.

Instead of specifying the injection pressure, it is more common to specify the injection rate. These are all encompassed by the limits of 5 to 15 metres per minute for normal duct sections. All of the standards allow these limits to be exceeded in special circumstances, the CIA suggesting an absolute upper limit of 40 metres per minute. In practice it is difficult to control rates and this is rarely attempted.

In the case of a duct blockage during the grout procedure, the duct should be quickly washed out and the operation restarted. However, the washing out of grout is surprisingly difficult.

Several standards, including EN 446, specify that the grouting operation should be complete within 30 minutes from the grout mixing. Some also specify a time limit between the placement of the prestressing tendons and grouting, typically of 28 days. This is to prevent the onset of tendon corrosion.

Full-scale trials are required by the Concrete Society Specification where demanded by the individual contract. EN 446 requires trial grouts on representative duct configurations in certain circumstances, i.e. when there is doubt about the ability to grout a particular duct successfully.

### 5.4.2 Complementary Grouting Procedures

EN 446 defines two complementary grouting procedures post injection and post grouting. Both have the aim of eradicating air or water pockets found in the duct after the main
<table>
<thead>
<tr>
<th>Specification</th>
<th>Testing</th>
<th>Temperature</th>
<th>Batching</th>
<th>Injection Pressure</th>
<th>Pump Rate</th>
<th>Post Injection</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>European Standard EN 446 and 447: 1996</td>
<td>Full scale trials on special projects. On site assessment of personnel, grout and procedures.</td>
<td>Air: 5-30°C Grout: 10-25°C Structure: 5-25°C</td>
<td>±2% cement and admixtures ±1% water</td>
<td>1N/mm² max (2N/mm² cut off) 0.5N/mm² for 1 min after grouting</td>
<td>5-15m/min</td>
<td>As required, up to 1N/mm² pressure or post-grouting (vacuum method optional)</td>
<td>Grout within 30 mins of mixing. Maximum grouting length: 50m</td>
</tr>
<tr>
<td>FIP Guide, 1990</td>
<td>Full scale trials on special projects. On site assessment of grout.</td>
<td>Grout: 10-40°C Structure: 5°C min for 3 days (use anti-freeze)</td>
<td>±2% cement</td>
<td>1N/mm² max (2N/mm² cut off)</td>
<td>5-10m/min</td>
<td>As required, 10-20 min after grouting. At 0.4-1.0N/mm²</td>
<td>Mixing: 2/4 mins max. Grout within 30 mins of mixing.</td>
</tr>
<tr>
<td>British Standard BS 8110: 1985</td>
<td>Full scale trials as required.</td>
<td>Structure: 5°C min for 2 days after grouting</td>
<td>2N/mm² max 0.5N/mm² for 5 min after grouting</td>
<td>6-12/min 2-3/min for vertical ducts</td>
<td>5-10/min</td>
<td></td>
<td>Use grout within 30 mins if no retarder is used. Grout tendons within 21 days of placing. Maximum grouting length: 50m Grout tendons within 28 days of placing.</td>
</tr>
<tr>
<td>Concrete Society Technical Report no.47, 1996, UK</td>
<td>Full scale trials as required. Pressure test ducts to 0.1N/mm² ≤50% pressure loss in 5 mins</td>
<td>Grout: 5-25°C Structure: 5°C min for 2 days after grouting</td>
<td>±2% cement and admixtures ±1% water</td>
<td>1N/mm² max (1N/mm² cut off). 0.5N/mm² for 1 min after grouting</td>
<td>5-10/min</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DIN 4227, Part 5 December 1979, Germany</td>
<td>Grout: 35°C max Structure: 5°C min for 5 days after grouting</td>
<td>±2% cement and admixtures</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mixing: 4 mins max. Grout within 30 mins of mixing.</td>
</tr>
<tr>
<td>Specification</td>
<td>Testing</td>
<td>Temperature</td>
<td>Batching</td>
<td>Injection Pressure</td>
<td>Pump Rate</td>
<td>Post Injection</td>
<td>Other</td>
</tr>
<tr>
<td>------------------------</td>
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<td>-------------------------------------------------</td>
</tr>
<tr>
<td>Fascicule 65-A</td>
<td>Pressure test ducts, water to 0.3N/mm^2,</td>
<td>When structure &lt;5°C or &gt;25°C use special grout,</td>
<td>±3% cement ±2% admixture ±1% water</td>
<td>1.5N/mm^2 max</td>
<td>0.5 N/mm^2</td>
<td>1 min after grouting</td>
<td>mixing: 4 mins max. (normally 1-2 mins)</td>
</tr>
<tr>
<td>January 1992, France</td>
<td>where required.</td>
<td>when &lt;0°C no grouting</td>
<td></td>
<td>0.5N/mm^2 for 1 min</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NEN 6722</td>
<td>Grout: 2-20°C</td>
<td>1N/mm^2 max (2N/mm^2 cut off)</td>
<td></td>
<td>6-12m/min</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Netherlands</td>
<td>Structure: 2°C min for 2 days after grouting</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PTI Draft</td>
<td>Test ducts for leaks</td>
<td>Grout: 32°C max and admixtures ±2% cement ±1% water</td>
<td></td>
<td>0.5 - 1.0N/mm^2</td>
<td>5-10m/min</td>
<td>As required, 10-20 min after grouting.</td>
<td>Grout tendons within 28 days of placing. Mixing: 2/4 mins max. Grout within 30 mins of mixing. After grouting, vents remain shut for 24 hr</td>
</tr>
<tr>
<td>Specification November</td>
<td>Structure: 2°C min</td>
<td></td>
<td></td>
<td>0.7N/mm^2 for 1 min</td>
<td></td>
<td>At 0.4 to 1N/mm^2</td>
<td></td>
</tr>
<tr>
<td>PTI Draft Specification</td>
<td>for 3 days (use anti-freeze)</td>
<td>After grouting</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>November 1994, USA</td>
<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>June 1993, Canada</td>
<td></td>
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<td></td>
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<td></td>
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<tr>
<td>CIA</td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Recommendations 1993,</td>
<td></td>
<td>Air: 5-32°C</td>
<td></td>
<td>0.75N/mm^2 max</td>
<td>6-40m/min</td>
<td></td>
<td>mixing: 2 mins min. Grout within 30 mins of mixing.</td>
</tr>
<tr>
<td>Australia</td>
<td></td>
<td>Grout: 5-32°C</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Structure: 5°C min.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Protect against freezing for 5 days</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>
grouting operation, post injection being performed before the stiffening of the original grout and post grouting afterwards.

Post injection should normally be carried out using a fresh batch of grout. In the case of prestressing strands, this grout is pumped in at the inlet under gradually increasing pressure up to 1N/mm². The ends of the strands are left protruding from the anchorages so that water collecting in the interstices between the strands may escape.

Post grouting may also be carried out, and often this is achieved using a hand-pump. Vacuum grouting has also been used.

5.4.3 Placing Thixotropic Grout

Many grouts currently used in post tensioned ducts are thixotropic in nature, that is, undisturbed they will thicken, but upon agitation, they will return to a fluid state. The PTI draft specification demands different injection procedures when these grouts are used. The thixotropic grout is to display zero bleed and mild expansion, the grout pump is to be driven by exacting pressure control equipment and the duct vent pipes at the tendon high-points are to be transparent with dual shut-off mechanisms.

Before grouting, the ducts are to be flushed with water. The grout is then mixed in a high shear colloidal mixer and is not required to pass through a screen before entering the grout hose. The injection pressure is held at 0.5N/mm² but then reduced to 0.35N/mm² when the grout reaches the vents and the duct outlet. When each vent is closed, the injection pressure is maintained at 0.35N/mm² for a further 15 seconds, then increased to 0.5N/mm². At the duct outlet, the injection pressure is slowly reduced before the duct is temporarily closed off.

When the grouting procedure is complete, the vents are again opened in turn. The inner valve is opened first, and then the outer so that a sudden pressure jump in the grout is avoided. A jump in pressure would risk forming a trapped air pocket within the duct.

5.4.4 Grouting of Vertical Ducts

The specifications summarised in the preceding sections have been developed mainly from experience and research on horizontal ducts. Vertical grouting presents somewhat different problems and special measures are specified in the PTI code. In the case of prestressing strands, the hydrostatic pressures in the column of grout can force the water into the interstices of the tendon. This water would then be forced up to the top of the column, known as the 'wick' effect.

PTI requires that, in lieu of a positive shut-off, vertical and near vertical ducts should terminate in reservoirs at the highest point. The reservoir must have sufficient capacity to store all bleed water to enable it to be reabsorbed into the grout. This reservoir should be maintained until the grout has set and the bleed water reabsorbed.

The CIA advises the addition of methyl cellulose dosed at 0.2% of the weight of cement to give a viscosity of 4,000 centipoise to the grout. This water retentive admixture would reduce the amount of bleeding and add thixotropic qualities to the grout.

When the vertical duct is very long, it may be necessary to grout in several stages to avoid excessive pumping pressures and to help reduce the effects of grout bleed. Several grout inlets would have to be provided. The PTI also specifies a lower injection rate of 5 metres per minute.

5.5 GROUT TESTING METHODS

There is a wide range of available tests to quantify the characteristics of grout. These are typically used on site during the grouting procedure, or earlier in laboratories for the pre-assessment of a grout mix's suitability. The specifications generally define the upper and lower limits to results, giving boundaries by which grouts are either accepted or rejected. Typical values have been given in Table 5.2.
Factors which must be considered when deciding on a choice of test for site purposes include:

- the time required to carry out the test
- the operator skill required
- the range of grouts for which the test will work
- the robustness of the equipment
- the ease of use and cleaning after use
- the suitability of the tests to site conditions.

These conditions automatically preclude the use of some of the more sophisticated research methods. The tests detailed in the grouting codes are described in the following sections.

### 5.5.1 Strength Tests

In America, compressive strength tests are according to the ASTM code C942-86. 51mm (2 in) cube moulds are half filled with grout. This is shaken down to remove air bubbles, then the mould is filled and shaken down again. The top surface of the grout is then smoothed with a trowel. If the grout contained an expansive admixture, the expansion is contained by covering the mould with an 18.2kg weight on a steel plate. The prepared samples are then cured in humid conditions for 24 hours or until the grout has set. They are then placed in lime water and cured for 7 or 28 days before testing.

In France, the test samples have the dimensions 40mm x 40mm x 160mm and undergo both bending resistance and compressive strength tests. Details of the test are described in NFP 18-360.

The samples are prepared at 20°C ± 2°C and in a relative humidity of at least 65%. The oiled moulds are filled in two stages, each time shaken down by receiving 60 shocks on a vibrating table. The samples to be tested at 28 days are then covered and stored at constant temperature and at 95% humidity until 20 minutes before testing. The other samples are removed from the moulds after 24 hours and placed in water to be cured.

The bending resistance test is then carried out on the samples until rupture. The resulting six half samples are subsequently used to measure the compressive strength.

During the compressive strength test, a load is applied to the 40mm square faces of the sample at a rate of 2 to 4N/mm² per second until rupture. If the results for these two tests differ by more than 10% for a given batch, then the test is be re-started.

EN 445 allows the use of any of the above methods, with no mention of any incompatibility of results.

### 5.5.2 Flow Tests

The flow cone test is the most common test for the measurement of grout flow characteristics (see Table 5.2). In this test, the time required for a specified volume of grout to flow through a cone is measured. There are a variety of cones in common use and the one recommended in the most recent European specifications is the CEN cone which has a volume of 1.7 litres and an internal throat diameter of 10mm. This size of throat corresponds to an average velocity of 75m/minute and a shear of 1000s⁻¹ for a flow time of the order of ten seconds.

The CEN cone is used in conjunction with a 3mm sieve to remove any grains or unmixed cement from the grout as it is poured in. Any material caught in the sieve is to be recorded.

Flow cones have the advantage of being cheap, robust and easily portable and are well suited for use on site. A disadvantage is that for fluid grouts the flow may be very fast so that accuracy is poor, whereas slightly thicker grouts may show much longer times or even not flow at all. Tests on a range of funnels have shown that, for each funnel geometry, there is a narrow range of viscosities and gel strengths for which the resolution is satisfactory.\(^{17}\)
Additionally, the flow cone test is not suitable for thixotropic grouts. Schupack\textsuperscript{46} has suggested an alternative method for thixotropic grouts based on the ASTM C939 Marsh Cone. The cone would be filled to the top, not to the stop mark, and the time taken for a litre of grout to pass taken. This method provides a driving head for the thixotropic material.

An alternative to the flow cone test is the immersion test. This is the standard flow test used in Germany for the pre-assessment of grout suitability. The test instrument consists of a 62mm diameter stainless steel tube which is filled with grout. A record is made of the time taken for a 58mm diameter bullet shaped plunger to fall 0.5m through the grout. The instrument is specified in the DIN Standard 4227 Part 5 and also the European Standard EN 445.

Immediately before the test is performed, the cylinder of the immersion test apparatus and immersion body (plunger) is slightly moistened with a clean damp cloth. The cylinder is then filled with about 1.9 litres of grout to 260mm below the edge, so that the immersion body will be just immersed when the stop on its guide rod is in contact with a spacer placed on the top end of the tube. The spacer is then pulled away and the immersion body sinks until the stop comes into contact with the tube. Next, the immersion body is raised to its initial position, the spacer is re-inserted, pulled away again, and the time that elapses until the stop comes into contact with the tube is measured.

The test is performed three times in succession with the same grout filling. The average of the immersion times measured during the second and third immersion is taken as the determinative value (the time measured during the first immersion is generally longer).

The immersion time of the grout should be measured immediately after completion of mixing and then again 30 minutes later.

The test apparatus should be calibrated at least once a year or after any damage to it. This involves altering the weight of the plunger until the average time for three drops through glycerol at 20°C is 34 seconds.

The immersion time specified in the EN Standard for grouts is 30 to 80 seconds, which correspond to velocities in the annulus of 7.0 to 2.6 m/minute and shear rates of 350 to 130s\textsuperscript{-1} for a Newtonian fluid. Thus the grout velocity is close to the normal range of specified injection velocities and the shear rate is higher than the upper limit during injection.

However, Taylor\textsuperscript{47} has reported that the test is difficult to use and has measured times of more than 120 seconds for German specified grouts with 0.39 water/cement ratios. Jefferis and Forrester concluded that the instrument is suitable only for high water/cement ratio grouts or grouts with a superplasticizer. Therefore the test is generally not suitable for thixotropic grouts.

5.5.3 Volume Change and Bleed Tests

The volume change of a grout sample is often measured at the same time as bleed, since both may be measured directly.

The sample is usually placed in a glass or plastic cylinder and covered to prevent evaporation. Although the tests in the specifications considered are the same in principle, there are several differences in their details, normally relating to the cylinder dimensions. Examples of these are given in Table 5.4.

The European Standard EN 445 suggests a further variant for the volume change test. This consists of using three 120mm high, 100mm diameter cans with water-tight lids. These cans are filled to 100mm and the distance between the top of the can and the grout surface is measured. This measurement is then repeated after 24 hours.

The results of these tests are given as simple percentages against initial volumes. In the case of the FHWA research method, it was noted that for the test cylinder with the inserted tendon, the average bleed results were 3% whereas for the conventional test, the grout
showed no bleed. This demonstrated that the filtration effect of wire strand tendons is apparent, even over small vertical distances.

<table>
<thead>
<tr>
<th>Specification</th>
<th>Cylinder Diameter</th>
<th>Cylinder Height</th>
<th>Cylinder Volume</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Euro. Standard EN 445:1996</td>
<td>Test as for UK’s Concrete Society or according to the French standard NFP 18-359</td>
<td></td>
<td></td>
<td>test 150mm of grout measure at 3hrs and 24hrs</td>
</tr>
<tr>
<td>Concrete Society, UK</td>
<td>50mm</td>
<td>200mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>French Standard NFP 18-359</td>
<td>25mm</td>
<td>250mm</td>
<td>100ml</td>
<td>test 90-100ml of grout measure at 3hrs and 24hrs</td>
</tr>
<tr>
<td>(Bleed test only)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>American code ASTM 940-87</td>
<td></td>
<td></td>
<td>1000ml</td>
<td>test 800ml of grout measure every 15 mins for 1hr, then once per hour until no further change</td>
</tr>
<tr>
<td>FHWA Research Method</td>
<td>51mm</td>
<td>1000mm</td>
<td></td>
<td>2 samples tested, one with insertion of 1305mm 7-wire strand to verify grout bleed and strand corrosion.</td>
</tr>
<tr>
<td>CIA Resc. 1993, Australia</td>
<td>40mm</td>
<td>600mm</td>
<td></td>
<td>Rod sample 25 times with 2.5mm rod. Measure at 3hrs and 24hrs</td>
</tr>
</tbody>
</table>

### 5.5.4 Set Time Tests

The Vicat test apparatus is used to determine the initial and final set time of grouts. The grout setting time gives an indication of the time that a grout remains workable and may be pumped into a post-tensioning duct after mixing. This test is described in the French code NFP 18-362 and the American ASTM C953-87.

The Vicat apparatus consists of a cylindrical steel needle, 50mm long and of 1.13mm diameter. This is attached to a guide so that it may drop vertically without friction. The moving parts have a total mass of 300g.

The test is carried out at 5°C, 20°C or 30°C, depending on the expected conditions on site. The grout components are kept at the chosen temperature for 12 hours preceding the test. The freshly mixed grout sample is poured into a 30mm high conical ring mould placed on a lightly greased glass sheet. The grout is not compacted, but excess grout is removed, leaving a smooth surface. The prepared sample is then kept at constant temperature in 90% humidity for 3 hours before being tested.

The Vicat apparatus is calibrated to zero when the needle is resting on the glass surface. The needle is then lifted and the sample placed under it. The needle is then carefully lowered so that it just touches the upper surface of the grout and held for a few seconds. It is then released and the scale reading taken after 3 seconds. This gives the distance of the end of the needle above the glass base-plate.

This procedure is repeated every 10 minutes, where the penetration holes are kept 10mm apart and until the test reading is 4mm. The time since filling the mould is recorded as being the initial set time.

The time between tests is now increased to about 30 minutes. The final setting time is recorded when the needle penetration is no more than 0.5mm.

Results are given as the average reading for three samples from a given batch of grout.
The Tusschenbroeck test runs in parallel to the Vicat test as described above. It is designed to measure the false setting of a grout, where the grout thickens prematurely. The test is described in the French code NFP 18-363.

The sample is prepared as for the Vicat test. The penetration of the needle is measured every 5 minutes after mixing, for the first 15 minutes. If the needle does not completely penetrate the sample, giving a zero reading on the scale, the test is repeated 6 times. False setting is said to have occurred when, for the 6 tests, the needle does not completely penetrate the sample 5 times or more.

5.5.5 Shrinkage Test for Hardened Grout

The French codes specify a test to measure the linear shrinkage properties of hardened grout. This test is described in the code NFP 18-361. Three samples are prepared using the same three 40mm x 40mm x 160mm moulds as in the strength tests. The specimens are prepared in the same manner, except that they are cured at 70±5% relative humidity. The lengths of the samples are recorded after removal from the moulds at 24 hours, after 3 days, 7 days and 28 days. The results are expressed using the equation:

\[ \text{shrinkage} = \frac{\Delta L}{L} \times 10^6 \text{(microstrain)} \]

where:
\( \Delta L \) is the change in length (mm).
\( L \) is the original length (160mm)

This test is supplementary to the shrinkage test for fresh grout described in Section 5.5.3.

5.5.6 Capillary Absorption Test

The current French codes specify that the capillary absorption characteristics of the hardening grout are measured. The test is described in the code NFP 18-364.

Three moulds are filled with grout as for the strength tests described in Section 5.5.1. These 40mm x 40mm x 160mm samples are cured at 20°C and at a relative humidity of 50%. After 24 hours, the samples are removed from the moulds, weighed and placed horizontally on a 20mm thick bed of sand just covered with water. The entire test is then sealed in a container and protected from evaporation.

The samples are weighed again after 24 hours, 48 hours, 7 days and 14 days from the beginning of the test. The samples are dried on a damp sponge immediately before each measurement, and then returned to the bed of sand. The capillary absorption, \( X_j \), is given by the expression:

\[ \frac{M_j - M_o}{64} = X_j \]

where:
\( M_o \) and \( M_j \) are the mass in grammes of the sample before the test and after \( j \) days of absorption. The results are given in g/cm².

5.5.7 Grout Stability Test

Both grout bleed and sedimentation results are required to understand the particle stability of a grout under working conditions. Sedimentation, even in horizontal ducts, can lead to the formation of low density, high permeability grout in the top of the duct. This process can therefore reduce the corrosion protection to the tendon, and may be characteristic of grouts showing little or no bleed in conventional tests as described above.

The distinction between bleed and sedimentation is not always fully appreciated. In reality they are different phenomena free fluid (bleeding) can be formed with minimal sedimentation and sedimentation can take place without free fluid being formed.
Sedimentation is defined as the differential vertical distribution of particle sizes due to settlement within a sample.

In dilute suspensions, the sedimentation of solid particles is described by Stokes' Law, individual particles settling at a rate determined by their size and density. In concentrated suspensions such as simple cementitious grouts, sedimentation is generally hindered and all of the particles, regardless of size, settle at the same rate. When cement settles in this manner, a layer of free water can develop at the top of the column. The more free water produced, the less stable the grout.

For grouts containing additives such as retarders, dispersants and water retentive admixtures, cohesive forces between the particles can be reduced. As a result, the different sized particles no longer settle together, allowing the coarse particles to settle at a faster rate. This results in higher concentrations of coarse particles at the bottom of the sample and fine particles at the top. When this occurs, grout at the top of the sample is of a lower density than at the bottom, although no bleed water may be apparent. Therefore, current bleed tests would not give an indication of the grout's instability.

A test for grout stability has been included in the Concrete Society Specification. In this test the grout is left for 24 hours in a transparent cylinder 40mm to 60mm in diameter and over 175mm in height. The cylinder is sealed to prevent evaporation and is not disturbed for this time. The sample is then removed from the cylinder and divided into four equal sections. The density of each section is then measured and the results reported as a percentage variation over the height of the sample.

5.5.8 Permeability Test

A test for chloride solution permeability of hardened grouts may be performed in accordance with the AASHTO T277-831 designation, "Standard Method of Test for Rapid Determination of the Chloride Permeability of Concrete". The test is based on the principle that negatively charged chloride ions are attracted to a positive electrode.

A hardened grout core, 51mm thick and 102mm in diameter, is saturated with water under vacuum prior to testing. The sample is then placed in a test cell so that there is a sodium chloride solution on one face and an alkali hydroxide solution on the other. There must be no direct communication between these two solutions. An electrical current is then passed so that chloride ions are attracted through the sample to the positive electrode. The current passing through the sample after 6 hours at 60V is taken as being directly proportional to its permeability since the resistivity of the test sample will be greatly reduced by the ingress of the chloride ions.

The relationship between the charge passed in 6 hours and the concrete permeability is given in Table 5.5. Most currently used grouts would lie in the moderate to low categories.

<table>
<thead>
<tr>
<th>Charge passed, Coulombs</th>
<th>Chloride Permeability</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 4000</td>
<td>High</td>
</tr>
<tr>
<td>2000-4000</td>
<td>Moderate</td>
</tr>
<tr>
<td>1000-2000</td>
<td>Low</td>
</tr>
<tr>
<td>100-1000</td>
<td>Very Low</td>
</tr>
<tr>
<td>&lt; 100</td>
<td>Negligible</td>
</tr>
</tbody>
</table>
6. INSPECTION

6.1 INTRODUCTION

The inspection of prestressed concrete structures is rather difficult because the vulnerable components, reinforcement and prestressing tendons, are beneath the concrete surface so that they cannot be seen directly unless the concrete has been broken off for one reason or another, in which case there is a self-evident problem. Moreover serious corrosion of grouted tendons can occur without exhibiting any external evidence.

In prestressing systems the most common type of construction defect is the presence of voids which can act as a container for water or leave the tendons exposed within the duct. Voids can occur due to a variety of causes; poor grouting techniques, unstable grout which exhibits separation and bleeding, and leakage from the duct during the grouting operations. In general voids most commonly occur at changes of curvature and at anchorages in horizontal ducts, and at upper anchorages in vertical ducts. The different causes of voids are as follows:

i) Grout can fall back from anchorages leaving as much as several metres of unprotected tendon. This is significant because water and chloride can enter the space via the end of the anchorage which is a vulnerable location. In land-based structures serious corrosion has been known to occur under these circumstances.

ii) Bleed water can collect on upper surfaces at changes of curvature and form voids having widths which are typically between 25% and 50% of the duct diameter. For the latter the prestressing steel is likely to be exposed. This type of void is not uncommon but rarely leads to serious corrosion.

iii) Small ‘worm’ shaped voids of several mm diameter and typically 0.5m long can occur on upper surfaces of the grout. These do not normally expose the steel and are not a serious defect. The presence of ‘worms’ is not uncommon.

iv) The grout can fail to enter the interstices formed between strands and the duct wall at locations where they are pulled together and against the duct wall by the prestressing action. This is a potentially more serious condition because the steel is exposed.

v) The duct may be completely empty due either to the grouting operation having been overlooked or the presence of a major leak so that the grout is pumped into and out of the duct. Under the latter circumstances a cementitious coating may be left on the steel so that protection is provided for a limited time. In land-based structures empty ducts have been discovered on a number of occasions and have resulted in serious corrosion to the extent that it has been considered necessary to replace the entire structure.

vi) In vertical ducts and in vent pipes fitted to horizontal ducts, separation of the grout can occur so that bleed water collects at high points. The bleed water may be reabsorbed in which case a void is left. In the high vertical ducts common to offshore structures the voids may be several metres long. Alternatively, segregation may occur so that the grout at the top of the duct dries out as a friable powder having no strength and no protective properties. It should however be added that with the thixotropic grouts developed in recent years, bleeding and segregation are less likely to occur.

It follows from the above that the most serious types of void are at the anchorages particularly at the top of vertical ducts, changes in curvature (in the vicinity of crests and troughs of horizontal ducts) and locations where the strands are pulled against the duct wall (also at crests and troughs). It goes without saying that wholly empty ducts present the greatest risk.

Inspection programs should include vulnerable locations such as joints between precast segments, joints and construction joints. These present a plane of weakness between two stages of concrete and it is possible for them to permit water and chlorides to travel to tendons crossing between segments (normally not protected) and ducts crossing construction joints. In practice the lightweight corrugated steel ducts commonly used to date can corrode and fail relatively quickly. Interestingly, empty ducts permit the contaminants to distribute over the length of the steel causing general corrosion whereas with fully grouted ducts the corrosion is concentrated in the vicinity of the joint and is much more serious as discovered in the Ynys-y-Gwas Bridge.

Attention has been given to defining tolerable sizes of void. In the Concrete Society Specification, type (ii) voids have been considered and it has been recommended that in new construction the depth of the void should not be greater than 5% of the duct's internal
diameter. For this size of void there should normally be sufficient grout cover above the strands to give reasonable protection.

The above discussion has been concerned with problems that can be caused by corrosion of the prestressing strands. Problems can also be caused by water entering the voids and freezing during cold spells. Cumming et al. considered this in relation to inspection of offshore structures in the Arctic environment. Critical void sizes were calculated according to two criteria: the minimum size of water-filled void that will cause cracking of the concrete when it freezes, and the minimum size of water-filled void that will cause yielding of the reinforcement when it freezes. It was calculated that voids 100mm long and 250mm from the concrete surface or 200mm long and 500mm from the surface would cause cracking. The critical size increased with distance from the concrete surface to the point where at a depth of 750mm a void at least 500mm long is required to initiate cracking with lengths greater than 1000mm to yield the reinforcement. Although not specified in detail the authors imply that the voids being considered are lengths of duct that are completely empty of grout; this is an unlikely geometry in practice. Also, the thicknesses of concrete cover that were considered were almost an order of magnitude greater than in normal prestressed concrete.

It follows that it is difficult if not impossible to specify a critical size of void even if it was possible to carry out inspections to the implied level of discrimination. The size of void and likelihood of corrosion vary according to the location in the structure, the design of structure and the environment.

In offshore structures, inspection of prestressing systems has received comparatively little attention due to other actions requiring higher priority. As a consequence, there is very little available information about the condition of the strands. However, during discussions with contractors and operators, limited unpublished experiences were mentioned:

- the cover was removed from a lower anchorage and a quantity of discoloured water escaped. The discoloration may have been due to corrosion products, but the water was not tested;
- ducts have been observed to be weeping. Samples of the water were collected and analysed, but no corrosion products were found. The ducts were re-injected with grout, but some continued to weep;
- on one occasion, when samples were removed from the concrete for strength tests, a duct was accidentally cored. It was found that the duct was completely filled with hard, well cured grout;
- in possibly the only instance when prestressing ducts have been inspected, examination was by invasive coring (see Section 6.3.4). The inspections were at four different locations and a total of 39 cores were removed. It was found that 36 of the cores were of well grouted ducts, one core failed to locate the duct and two had water filled voids.

Considerable research has been carried out on methods of detecting defects in post-tensioning ducts and the following section briefly outlines the available techniques and procedures.

### 6.2 ORGANISATION AND PLANNING OF INSPECTIONS

There are a number of Codes and Advisory Documents providing general guidance on the inspection of concrete structures, for example the reports of the Concrete Society, the Institution of Structural Engineers and the FIP. These are mostly generalised to cover all types of concrete structure and are not specific to offshore structures. One of the most recent and specific documents is the Highways Agency document BA 50/93 for inspection of post-tensioned concrete bridges. Although written specifically for bridges, much of this is relevant to offshore structures and provides the most useful available information.

Experience obtained during three years operation of BA 50/93, including complete inspection of 104 post-tensioned concrete bridges, has been summarised by Cullington et al.
There are three phases in the Special Inspections of post-tensioned concrete to BA 50/93; desk study, preliminary inspection, and detailed inspection. The following sections outline those principles of BA 50/93 relevant to offshore inspections.

6.2.1 Desk Study

The desk study is concerned with assessing the as-built design drawings, the construction records and the inspection and maintenance records. The basic design details need to be assessed in order to identify the locations and dispositions of the post-tensioning ducts and the positions where problems are most likely to be encountered. Construction records should be reviewed to identify whether any difficulties were encountered particularly with respect to the grouting operations. Problems that have been reported during the construction of offshore structures include blocked ducts that have subsequently either been cleared or abandoned altogether, bleeding and fall-back of the grout level at the top of vertical ducts, displacement of the ducts and cracked concrete cover due to the high pressure of grouting operations. The original material specification should provide information about the grout, its water/cement ratio, and its cement and admixture content. These are useful in assessing its expected performance both during construction and in service.

It is relevant to note that comparatively little has been reported about construction operations in relation to incidents, problems and trials. During the preparation of this review, discussions were held with designers, contractors and operators. It was pointed out on several occasions that information about construction was often not available because it was held informally by engineers who subsequently moved on to other projects. Indeed there has been a tendency for engineers to move around more frequently than their opposite numbers in land-based work.

Large scale grouting trials and material tests, although invariably carried out, were apparently not reported in detail and it has not been possible to identify any reports, confidential or otherwise. In consequence of this, it would appear that there has been little transfer of information relevant to grouting operations between the construction teams or contractors and maintenance managers. This situation would be in contrast to land-based structures where the maintenance engineers usually have full records that can be used in support of inspections.

When the available information is insufficiently detailed it may be necessary to carry out extra investigation work during the preliminary site inspection.

It is considered very important to examine all previous inspection and maintenance records to determine whether there has been any history of problems. Nil returns are relevant because they indicate the extent to which the preliminary inspection should be carried out.

The report on the desk study should identify tasks to be undertaken in the preliminary inspection of the structure.

6.2.2 Preliminary Inspection

The objectives of the preliminary inspection are to confirm the design details of the structure, (for example the locations of ducts and anchorages, and the methods of protection from sea water and the environment), to identify any deterioration of the structure with special emphasis on the locations in the post-tensioning system known to be vulnerable to corrosion, and to recommend whether it will be necessary to proceed to a detailed site inspection. Indications of problems are typically cracking, rust staining of the concrete surface and leakage of water. The preliminary inspection can involve the use of equipment such as a cover-meter to aid the location of steel reinforcement. The anchorages can be uncovered if this involves no more than unbolting an end-cap. The preliminary inspection does not normally involve the removal of samples from the concrete or the use of sophisticated equipment.

It should be noted that preliminary inspections have sometimes identified problems unrelated to the post-tensioning system.
A recommendation to carry out a detailed inspection of the structure should not be made lightly as tasks such as the removal of concrete cover and invasive inspection of the ducts are difficult to make good. It is not possible to fill in the holes after an invasive inspection and achieve the same resistance to chloride ingress as for the original as-cast concrete cover.

If it is considered that a detailed site inspection is required, the report on the preliminary inspection should provide a technical plan and objective.

### 6.2.3 Detailed Inspection

Objectives of the detailed inspection are to investigate the locations identified as being likely to have corrosion or other damage.

The detailed inspection can involve a range of special techniques. Those that are totally non-destructive tend to be indirect and provide data that are indicative rather than definitive. With the exception of radiography, the direct methods are invasive and have the implications outlined in Section 6.2.2. The available inspection methods are summarised in the Section 6.3.

The report on the detailed inspection should provide a full account of the work and the data collected. If defects are found, the implications should be assessed and appropriate actions recommended.

### 6.3 METHODS OF INSPECTION

In Section 4 of BA 50/93, methods are outlined including endoscopy, pressure-vacuum testing, radiography, surface penetrating radar, impact-echo and reflectometry. There is a section on internal examination carried out after voids and potential corrosion have been identified. Methods outlined for exposing the tendons, to enable the internal examination to be carried out are percussion (impact breakers), diamond saw drilling, high pressure water jetting and grit blasted holes. It is also mentioned that during the internal examination, samples of the grout should be tested to determine whether any carbonation has occurred or whether chlorides are present.

Section 4 of BA 50/93 was drafted before the Highways Agency's programme of special inspections had started and could usefully be revised in the light of subsequent experience in the field. Nevertheless it remains the most practical and useful guide to inspection of post-tensioning tendons. Moreover, the principles are equally relevant to offshore structures.

There have been numerous research programmes to investigate methods of inspecting post-tensioned concrete, for example CANMET, NCHRP and FHWA in North America and TRRL in the UK.

The different techniques for inspection are summarised in the following sections, and are considered in relation to their applicability to offshore structures.

### 6.3.1 Non-Destructive Testing at the Concrete Surface

In this section, nine methods of detecting damage are considered. They are classed as direct methods because a positive result would normally indicate broken or severely corroded wires. A summary is given in Table 6.1.
<table>
<thead>
<tr>
<th>Method</th>
<th>Comments</th>
<th>Experience in the field</th>
<th>Applicable offshore</th>
</tr>
</thead>
<tbody>
<tr>
<td>Radiography, Radioscopy and Gammagraphy</td>
<td>Direct method, interpretation fairly easy.</td>
<td>Considerable</td>
<td>Possible</td>
</tr>
<tr>
<td>Radar</td>
<td>Interpretation requires experience.</td>
<td>Limited</td>
<td>Possible</td>
</tr>
<tr>
<td>Impact-echo</td>
<td>Interpretation requires experience. Results tend to be conservative.</td>
<td>Limited</td>
<td>Possible</td>
</tr>
<tr>
<td>Ultrasonic methods</td>
<td>Sophisticated equipment. Interpretation requires experience.</td>
<td>Limited</td>
<td>Doubtful</td>
</tr>
<tr>
<td>Tomography</td>
<td>Applied in other fields but limited work on concrete.</td>
<td>None</td>
<td>Unlikely</td>
</tr>
<tr>
<td>Thermography</td>
<td>Applied in other fields but limited work on concrete.</td>
<td>None</td>
<td>Unlikely</td>
</tr>
<tr>
<td>Electrical continuity</td>
<td>Not practical</td>
<td>None</td>
<td>Unlikely</td>
</tr>
<tr>
<td>Reflectometry</td>
<td>Interpretation requires experience</td>
<td>None</td>
<td>Possible</td>
</tr>
<tr>
<td>Magnetic flux</td>
<td>Not yet fully developed</td>
<td>None</td>
<td>Doubtful</td>
</tr>
</tbody>
</table>

**Gammagraphy, radiography and radioscopy**

Gammagraphy, radiography and radioscopy involve the attenuation of radioactive radiation through the structure.

Table 6.2 gives a comparison between the three methods.

<table>
<thead>
<tr>
<th>Method</th>
<th>Rays</th>
<th>Result</th>
<th>Maximum thickness of concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gammagraphy</td>
<td>$\gamma$</td>
<td>film</td>
<td>600mm</td>
</tr>
<tr>
<td>Radiography</td>
<td>X</td>
<td>film</td>
<td>1.3m</td>
</tr>
<tr>
<td>Radioscopy</td>
<td>X</td>
<td>video image</td>
<td>1.3m</td>
</tr>
</tbody>
</table>

**Application**

BS 1881 provides recommendations for radiography of concrete. A particular radiographic source that has been used extensively is the Scorpion developed by LCPC in France\(^\text{7}\). The average energy of the Scorpion is 4MeV. However, the Scorpion is a bulky and heavy instrument. There are safety implications and it is necessary to clear people away from the vicinity of the Scorpion to avoid radiation. Another radiographic source which is often used is the betatron. Some betatrons are relatively small and portable. Betatrons have been used successfully for inspections of highway structures in relatively close proximity to people.

Notwithstanding the disadvantages, gammagraphy has been used reasonably successfully on some 60 structures in the field.
Limitations

Costs are high. The surface area that can be covered by one measurement is limited to 1.2m x 1.2m. High safety precautions are necessary.

Radar

Pulses of low power radio frequency are transmitted into the structure. The reflected signal is detected by a receiver and processed for display and analysis. At a subsurface interface between materials of different dielectric properties the signal is partly reflected and partly transmitted.

Application

It is claimed that this method can be used to locate reinforcing bars and differences between corroded and uncorroded bars but the latter is unproved. It has been used for locating ducts but has been judged effective in only half the cases. Radar is strongly supported by some investigators.

Limitations

It cannot detect voids and corrosion in metallic ducts because the sheathing reflects the radar.

Impact-echo

A mechanical impact creates a transient stress pulse of compression (P-wave). The stress pulse is reflected at the other side of the structure and at possible irregularities such as cracks, voids, steel bars, etc. The arrival of these reflected waves at the surface, where the impact was generated, produces signals which are measured by a receiving transducer.

Impact-echo is essentially an automated version of the old fashioned method of tapping a structure and listening for a hollow sound which would denote the presence of a defect.

Application

There have been several evaluations of impact-echo using beams having known defects. Result have varied but on all occasions some of the defects have been identified. The method has been tested in the field and commercial units developed. However, it has not yet been generally adopted for field use.

Limitations

A mechanical pulse does not always create P-waves but also shear waves (S-waves) and Raleigh waves (R-waves). In consequence the response signal contains a combination of P, S and R waves and the signal analysis can be very complicated. Inspecting a long duct with this system would be time consuming. The system could be used to check local spots. The method is generally applied to the whole concrete section. Anomalies in the concrete complicate the signal analysis of the reflections coming from the duct.

Ultrasonic pulse transmission

The time taken for a pulse to travel between two transducers positioned on the concrete surface is recorded. If a defect is present along the path, the transmission time will be increased as the pulse has to pass around the void.

Application

Attempts to detect voids in ducts using ultrasonic pulse transmission have met with limited success as the change in transmission time is small and masked by other effects such as the presence of reinforcement and variations in the quality of the concrete. However, recent work has indicated that it might be possible to detect defects for ducts containing voids although it is not possible to determine the size of void. Commercial equipment is available that is cheap and easy to use.
Limitations

The method requires access to both sides of the concrete element. The effect is small and difficult to detect.

Ultrasonic pulse along tendon

An alternative approach to ultrasonic transmission is to propagate pulses along the prestressing tendons rather than the concrete. Much higher frequencies can be used as the attenuation is less in the steel than in concrete and has the advantage that the pulses are more directional.

Application

The technique has been investigated under laboratory conditions and shown to work in principle. However, commercial equipment designed specifically for the purpose is not available and the method has not been properly tested in the field.

Limitations

The technique has the disadvantage that access is required to the anchorages and detection of fractures is limited to a short distance from the ends because a considerable amount of energy is lost in the grout. However, this is compensated by the fact that this is a vulnerable area where corrosion and failure of tendons can occur.

Tomography

Tomography is based on measurement of the velocity or the travel time of a pulse (impact echo, radar, ultrasonic) in the concrete. Instead of using one receiver several receivers are installed around the concrete. After a first measurement of the travel time of the signal to each receiver, the source is placed in another point of the same cross-section and a new measurement is made.

This method enables the spatial distribution of the velocities through the concrete to be obtained.

The result can be presented as an image, eg a dark pixel when a low velocity of the signal is found for the segment.

Tomography can also be used in combination with gammagraphy and radiometry. In this case the attenuation of the radioactivity in different locations is measured.

Applications

This method has been used in the field for other applications with apparent success but is unlikely to be practical for detecting voids in post-tensioning ducts.

Limitations

This method is rather laborious but gives an indication of the location and volume of air voids, cracks, reinforcement bars and ducts. The application of this system together with the installation of the measurement equipment is complex in practice.

Thermography

Two methods have been reported in the literature:

The surface of the tested object is submitted to fast heating. The heat front propagates into the object. When a thermal inhomogeneity is met (eg a delamination or an air void) this will be reflected back to the surface. An infrared camera is used to detect the temperature difference and indicate the defect.

The tendons of the structure are heated some 10°C above the ambient temperature so that they can then be located using thermography.
Application

This method is laborious because, after the heating process, the structure has to be scanned with a camera. Naturally occurring thermal gradients have been used to detect delaminations caused by reinforcement corrosion. There are no reports of using this method to detect defects in grouting. It is doubtful whether it would work.

Limitations

There is a risk of introducing high thermal stresses and cracking the concrete.

Electrical continuity

This is a simple method of detecting fractures in tendons which are electrically isolated from each other and is often recommended for investigation.

Application

This method is applied to ground anchors and has been considered for detection of broken strands in post-tensioned concrete.

Limitations

In practice it is unlikely to be successful as the tendons are usually in contact with each other through the anchorages, duct sheathing and secondary reinforcement.

Reflectometry

RIMT (Reflectometric Impulse Measurement Test) consists of applying a series of electrical pulses to one end of the tendon. These electrical pulses travel along the cable and will be affected by the variations in impedance of the conductor.

Application

RIMT was applied to the ‘Katerveer II’ bridge in Holland. A working party has been set up in Holland to investigate RIMT.

Limitations

In the UK trials were carried out on a beam with known defects. RIMT was only partially successful and was not recommended.

It is necessary to have access to the end of the tendon.

Magnetic flux exclusion

A magnetic field is applied to the concrete and the presence of flaws in the tendon produces a disturbance in the field which can be detected by a Hall effect probe on the surface of the concrete.

Application

The method has been used in laboratory tests to detect fractures in a 13mm diameter strand at depths of 50 to 75mm from the surface.

Limitations

Distinguishing between damaged tendons and other causes of disturbance such as pieces of tie wire embedded in the concrete presents problems.
6.3.2 Corrosion activity

In this section, five methods of detecting the likelihood of corrosion are considered. These are classed as indirect methods because positive results indicate that corrosion may be occurring. It should be emphasised that the measurements relate to activity in the reinforced concrete and possibly the steel duct. However, indicated activity in these locations suggests that further investigation of the tendons should be carried out.

The methods are compared in Table 6.3.

<table>
<thead>
<tr>
<th>Method</th>
<th>Comments</th>
<th>Experience in the field</th>
<th>Applicable offshore</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chloride content</td>
<td>Requires collection of concrete samples. Easy to interpret.</td>
<td>Considerable</td>
<td>Yes</td>
</tr>
<tr>
<td>Carbonation</td>
<td>Requires exposure of fresh concrete surfaces. Easy to interpret.</td>
<td>Considerable</td>
<td>Yes</td>
</tr>
<tr>
<td>Resistivity</td>
<td>A practical method that provides useful information</td>
<td>Some</td>
<td>Possible</td>
</tr>
<tr>
<td>Electrode potential</td>
<td>Commonly used, fairly easy to interpret.</td>
<td>Considerable</td>
<td>Yes</td>
</tr>
<tr>
<td>Polarisation</td>
<td>Laboratory technique, not yet used in the field</td>
<td>Limited</td>
<td>Unlikely</td>
</tr>
</tbody>
</table>

Chloride content

Chlorides progressively penetrate the concrete during service. When they reach the steel, corrosion can occur. There are a number of methods of measuring chloride content, the most common requiring the removal of concrete powder extracted at different depths from the surface. The samples of powder are subsequently analysed in the laboratory and results are most commonly expressed as percentage chloride per weight of cement. There are numerous other methods of measuring chloride content but mostly less accurate.

There have been attempts to establish a threshold value of chloride content at which corrosion will occur. However, it is best to consider values in relation to carbonation depth and electrode potential so that an overall judgement can be made.

Application

Chloride contents have been measured on many occasions and on many types of structure so that there is considerable experience. The test is practical and useful.

Limitations

A full set of measurements on a large structure would be expensive

Carbonation

During the life of concrete, carbon dioxide and other acidic gases in the atmosphere react with the alkaline constituents at the surface of the concrete. As a result the normal protection provided against corrosion is lost as the pH value falls. This is known as carbonation. With time the carbonation becomes deeper until eventually all the cover concrete is affected. On site the best method of measuring the depth of carbonation is by exposing a fresh concrete surface then spraying with a 2% solution of phenolphthalein in ethanol. The resulting magenta areas represent the unaffected concrete and the colourless areas represent the carbonated concrete.
Application

Measurement of carbonation depth is a useful and practical test that can be carried out in the field. It indicates the protective quality of the concrete cover and has been used on many occasions.

Limitations

Measurement of carbonation depth is an indirect method and provides no information about conditions inside a steel duct.

Resistivity

Measurement of resistivity of the concrete provides useful information about the likelihood of corrosion. If resistivity is high, the corrosion current will be so small that no significant corrosion will occur. Resistivity measurements are related to the moisture content of the concrete and can be used to predict the probability of corrosion. A common way of measuring resistivity is by the Wenner method. Four contact points are placed on the concrete in a straight line and spaced equal distances apart. An alternating current is made to flow between the outer contacts and the resulting difference in potential between the inner contacts is measured. The resistivity of a unit volume of concrete can be calculated from these measurements.

Application

Measurements of resistivity provide an indication of the likelihood of corrosion. Commercial equipment is available and there is considerable experience of its use.

Limitations

Measurement of resistivity is an indirect method and provides no information about conditions inside a steel duct.

Electrode potential

Electrode potential measurements are commonly used to assess the risk of corrosion in reinforced concrete. The electrode potential in the concrete environment is measured by comparison with the known potential of a reference electrode (half-cell), usually saturated copper/copper sulphate and sometimes calomel. An electrical connection is made to the steel at a convenient position enabling electrode potentials to be measured at any desired location by moving the half-cell over the concrete surface. A grid of suitable dimensions is usually marked out on the concrete surface so that electrode potential can be measured in an orderly manner and results plotted as equi-potential contours. The level of the electrode potential provides an indication of the presence of corrosion activity. Information is also provided by the shapes of the potential contours.

Application

There is considerable experience in the use of electrode potential measurements to investigate corrosion of reinforcement. It is not used so often in relation to tendons but can provide an indication of the likelihood of problems. It is a practical method that can be used in the field. Equipment is available to enable measurements to be automated and plotted on-line.

Limitation

Values of electrode potential are influenced by the moisture content of the concrete. When the tendons are in steel ducts they are shielded so that direct measurement cannot be made. Nevertheless, results indicating likelihood of corrosion in the reinforcement point to the need for further investigation of the tendons.
Polarisation

In this method the electric potential is measured between a reference cell and the tendon. A current is sent into the object. The current I and the corresponding voltage V are measured. The slope of the V-log I diagram is a function of the corrosion rate of the tendon.

Application

The measurement equipment comprises a ring which transfers the current around the reference half-cell. Sometimes an outer ring is present which concentrates the current to the cross-section under investigation and avoids the spreading of the measurement current (from the inner ring).

This technique has been used for corrosion measurements in the laboratory but has not been used successfully on post-tensioned concrete in service.

Limitations

For use in the field, polarisation has no compelling advantages over the established method of electrode potential.

6.3.3 Mechanical activity

In this section, three methods of detecting the likelihood of corrosion are considered. These are classed as indirect methods because positive results indicate the presence of mechanical activity that could be due to a variety of causes, one of which may be damaged tendons.

The three methods are compared in Table 6.4.

Acoustic emission

The energy which is liberated when a crack is formed generates an acoustic signal. The principle of the acoustic emission technique is to record these acoustic signals by putting sensors into the concrete or by attaching sensors to the element. The localisation of the sources of acoustic emission (cracks) is done by using the first arrival times of the P-waves of the emitted signal.

Application

Acoustic emission works well in a laboratory environment but less well in the field. The technique has often been considered for monitoring structures but there are no reports of it being used successfully for post-tensioned concrete.

Limitations

For post-tensioned bridges, there is likely to be quite a lot of acoustic noise during normal operation so that if there is also emission due to spurs of crack growth in the concrete it is likely to be masked. The method is more suitable for long term monitoring.

Vibration characteristics

The natural frequencies of a structure changes when a crack grows. In consequence measurement of the natural frequencies enable the condition of the structure to be assessed. This could be done by carrying out a one-off test and comparing results with theory or by periodic measurements and looking for changes. Dynamic characteristics can be obtained by measuring response of the structure to impact, mechanical activities (operation of equipment), wave loading or wind.

Application

There have been ambitious claims for vibration measurements and the method has been tested on steel offshore structures. It has not been used successfully on concrete structures.
Limitations

The method measures stiffness not strength. Loss of prestress due to corrosion or fractured tendons could cause serious reduction in strength but little change to the vibration characteristics. Also, the small changes that do occur can be masked by other effects such as temperature.

<table>
<thead>
<tr>
<th>Method</th>
<th>Comments</th>
<th>Experience in the field</th>
<th>Applicable offshore</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acoustic emission</td>
<td>A sophisticated technique</td>
<td>Limited</td>
<td>Unlikely</td>
</tr>
<tr>
<td>Vibration monitoring</td>
<td>Considerable research but not able to discriminate to required level.</td>
<td>Limited</td>
<td>Unlikely</td>
</tr>
<tr>
<td>Stress-relief coring</td>
<td>Measurement of in situ stress. A well established and practical method.</td>
<td>Considerable</td>
<td>Yes</td>
</tr>
</tbody>
</table>

In Situ Stress

In situ stress in prestressed concrete can be measured by the instrumented coring method. At the location where measurements are to be made, an array of demec gauges is fixed at positions on the area to be cored and across the core hole. Eight vibrating wire strain gauges are fixed at locations outside the core. Appropriate gauge lengths and spacings are selected for standard core sizes of 75mm and 150mm. The core is removed by precision coring. Changes in the strains, as measured after the coring, are noted. The relevant value of elastic modulus of the concrete can be obtained by laboratory measurements on the core, or by use of a specially developed hydraulic jack which can be inserted in the core hole. Loads are applied to the jack to establish a load-strain relationship in four directions. Using these data, values of the elastic modulus of the material and the principal stresses can be calculated to an accuracy of ± 1/Nm². The in situ jacking method has the advantage that it measures the in-plane modulus. The method of stress-relief coring is fully described by Mehrkar-Asl.

Application

Instrumented coring is a specialist technique which has been in use since 1987. It has been applied successfully to land-based structures in several countries. In the UK some 40 structures have been investigated, involving 250 instrumental cores. The technique is well established and applicable to offshore structures.

Limitations

A high level of skill is required for both the coring and the interpretation of the data. As with other techniques involving drilling or coring, care has to be taken to ensure that reinforcement bars or tendons are not cut. It is difficult to make good the hole to provide the same level of protection as before from the cover concrete because the joint between the original concrete and infill forms a plane of potential weakness. In cases where ducts have been well grouted but there is local corrosion and failure, failures will not be detected if the measurements are taken at locations beyond the rebound length from the point of failure.

6.3.4 Direct observation

In this section, three direct methods of detecting voids and corroded or broken strands are considered. These are compared in Table 6.5.
Invasive drilling

This is an invasive method of exposing the tendons. With wet drilling the process can be controlled so that the reduced electrical resistance which is obtained shortly before a metallic duct is reached can be used to trigger a switch-off device acting on the drilling machine. In the case of dry drilling automatic switching off can be obtained upon the first contact between the drill bit and the duct. In practice, experienced operators can drill to the depth of the duct without requiring automatic aids. Subsequently the ducting can be manually opened. The different methods of exposing the tendons are mentioned in at the beginning of the Section.

Application

Invasive drilling into a duct has become a routine procedure that has been used on numerous occasions in the field and has provided valuable data.

Limitations

Inexperienced operators could cause damage to the tendons. It is difficult to make good the drilled hole to provide the same level of protection as before from the cover concrete because the joint between the original concrete and infill forms a plane of weakness. If there is a delay in making good, contaminants may enter the duct and be impossible to clean off.

<table>
<thead>
<tr>
<th>Method</th>
<th>Comments</th>
<th>Experience in the field</th>
<th>Applicable offshore</th>
</tr>
</thead>
<tbody>
<tr>
<td>Invasive drilling</td>
<td>The most positive available method. Practical. Drawback is that it is difficult to make good and completely restore cover if tendons found to be in good condition.</td>
<td>Considerable</td>
<td>Possible</td>
</tr>
<tr>
<td>Endoscopy</td>
<td>Follows after smart drilling. Practical and provides direct evidence.</td>
<td>Considerable</td>
<td>Possible</td>
</tr>
<tr>
<td>Pressure-vacuum leakage</td>
<td>Follows after smart drilling and endoscopy. Provides extra information.</td>
<td>Limited</td>
<td>Possible</td>
</tr>
</tbody>
</table>

Endoscopy

As part of the invasive drilling investigation, an endoscope can be inserted into an exposed duct and used to check for voids and the state of the prestressing wires. Photographs of existing cavities and condition of the tendons can be made by attaching a camera to the endoscope.

Application

Endoscopy has been successfully used in the field on many occasions.

Limitations

If endoscopy causes delays and extra time when the duct is open to the air, there is increased likelihood of contaminants settling on otherwise undamaged tendons.
Pressure-vacuum testing

This method is applicable after a void has been detected and assessed. It consists of calculating the volume by application of a pressure to the void, using Boyle's Law.

Application

After drilling a hole to access the void, the pressure-vacuum test can be used to determine the volume of the void. This method is occasionally used in the field.

Limitations

If pressure-vacuum testing causes delays and extra time when the duct is open to the air, there is increased likelihood of contaminants settling on otherwise undamaged tendons.
7. CONSEQUENCES OF FAILED TENDONS

There is a wide variety of concrete offshore platforms and they have been provided with a range of prestressing. The consequences of failure of the prestress will vary greatly, depending largely on the nature of the design loading. Some of the prestress may be provided purely for temporary conditions that arise during erection and float-out. Long term failures of such prestress may have no serious consequences. In contrast, other prestress may be vital to the safety of the platform. Since it is not possible to generalize on the consequences of failure, it is necessary to consider the types of prestress separately.

7.1 VERTICAL PRESTRESS IN LEGS

Vertical prestress in the legs is the only form that virtually all the platforms have. It is therefore convenient to consider this form first.

The legs of a concrete offshore production platform serve primarily to hold up the deck and all the equipment mounted on it. They are subjected to permanent compressive loads of thousands of tonnes. This is not, however, a design criterion for the prestress in the legs. The prestress is designed primarily for bending from the design wave. The permanent load is approximately axial with only very small moments. The vertical dead load in the legs acts as additional prestress and increases the moment capacity of the concrete sections and so reduces the prestress required. The stress in the concrete due to prestress is therefore usually relatively low. It is typically greater than the tensile strength of the concrete but of a similar order. The result is that the legs would be able to resist a significant percentage of the design live load with no prestress at all. This is in complete contrast to most other major prestressed concrete structures, from bridges to nuclear pressure vessels and water tanks. For these, the dominant loading is permanent and much of the prestress is required to resist the permanent load. This has significant implications for the structural consequences of failures in the prestressing systems.

Provided the corrosion is not very localised and concentrated at one section, as in the well known case of the Ynys-y-Gwas bridge, prestressed structures are normally ductile. The moment required to cause failure is much greater than that required to produce the first visible sign of distress. A number of major bridges have experienced cracking or excessive deflection that was considered severe enough to require remedial action even though analysis showed them to be safe. Analysis of these bridges indicates that the prestress required to make them safe is, typically, half that required to prevent visible cracking. The permanent load in these relatively long span concrete bridges is of the order of 75% of the total. The extent of prestress loss required to make them unsafe under full design live load is therefore much greater than required to show visible signs of distress under permanent load. The result is that it is practically impossible for such bridges to become unsafe due to loss of prestress without warning.

The prestressed legs of concrete production platforms are also ductile in the technical sense and would show severe cracking under much lower moments than would be required to make them unsafe. However, unless the moment is very near the failure value, removing the moment would cause the cracks to close up again leaving the element apparently undamaged. Also, the tensile strength of concrete aided by the permanent compression means that the legs would not crack until a very substantial moment was applied. This, combined with the non-linear relationship between wave height and wave force and the absence of any significant permanent moment in the legs, means that a prestressed platform that is seriously deficient in strength due to loss of prestress may show no obvious sign of damage. The legs would therefore remain apparently undamaged until a design, or near design, storm occurred. Serious damage could then result which could lead to complete failure.

Another difference between these elements and most other prestressed elements, including others in offshore structures, is that ultimate strength is often critical in design. This arises because of the use of class two design criteria, allowing some tension, in combination with relatively light prestress. This has the effect that a significant part of the service moment can be resisted by the tensile strength of concrete. The significance of this to the consequences of prestress failure is that it means that any loss of prestress will lead to a shortfall of ultimate capacity.
The above may appear alarming. It suggests that any loss of prestress leads to the elements becoming unsafe and that this lack of safety will not be apparent. However, a marginal loss of prestress only makes the structure unsafe in the sense that it is does not comply with normal design criteria. It would only actually lead to failure if the structure was subjected to the design (factored) wave force and if the structure had only its design strength. In practice, because of the relatively low level of prestress, the critical element in the strength of the structure is the prestress and, if there are significant amounts of it, the secondary reinforcement. The design strength of both prestress and reinforcement in the British codes to which platforms were designed (CP10 and BS 8110) was taken as 87% of the characteristic strength. This is based on obtaining an acceptably low possibility of an individual bar or strand having less than the design strength. The strength of the very large elements considered here is not, however, dependent on the strength of individual bars, strands or tendons. It is dependent on the total strength of all the steel. This suggests there would have to be a loss of over 10% of tendon area before the platform had less than target reliability. Recent work suggests the BS 8110 safety factor for reinforcement and prestress is excessive even for smaller elements. An amendment changing it from 1.15 to 1.05 has been accepted.

The load and material safety factors mean that for the platform to fail under the unfactored design wave load, it would have to lose something of the order of 40% of the prestressing steel in the critical area. Because of the ability of tendons to re-anchor over comparatively short lengths (typically 1 to 2 meters) the tendons would also have to become ineffective in approximately the same sections. This is improbable by random chance. It is only likely to arise if there is something so radically wrong with the corrosion protection that it would become obvious. However, problems could arise if the corrosion was concentrated at specific sections so that the distribution was not random.

The joints of segmental structures are potentially vulnerable because corrosion damage is likely to concentrate there. This was the problem with the Ynyw-y-Gwas bridge. However, platform legs are normally slipformed so that construction joints are avoided as much as possible. The Maureen articulated loading column is of segmental construction however, being formed of precast segments stressed together with conventional bonded prestress. Unlike the legs of conventional gravity structures, the critical load case for the design of this prestress arises in construction when the column is raised from horizontal to vertical. As the prestress is designed to avoid any tension under this critical condition, the loss of prestress required to make it unsafe under the less severe loads it subsequently experiences would be very great.

Ninian is the only other North Sea structure that has been identified as containing a significant proportion of segmental construction. The break-water wall of this has both vertical and horizontal joints. The vertical joints are relatively wide and filled with in situ concrete. They are therefore essentially the same as normal construction joints in situ concrete construction. The horizontal joints are thinner, having been cast. Considerable effort was taken to ensure that the ducts were sealed across the joint. However, these joints must still be considered a relatively vulnerable area and at least those in the splash zone should be inspected regularly.

A potential problem could arise in conventional structures at the transition section between steel deck and concrete legs where the connection is made by anchoring the tendons in the steel structure. The transition is a potential weakness in terms of tendon corrosion, particularly in view of the bleeding problems that can arise in the top of tall vertical tendons. Many platforms were designed with the steel deck as a load bearing member under wave load. However, this may not be as critical as it appears. In the Beryl Alpha platform, for example, the design moment at the transition is only some fifteen percent of that at the base of the legs. This implies that, even with a total failure of the moment capacity at the transition, the structure would not necessarily fail.

Complete failure of a structure due to loss of prestress is very unlikely. However, it is theoretically possible for it to arise without any obvious signs of distress becoming apparent in advance. Such warning cannot, therefore, be relied on. Because of this, structures should be monitored and inspected to ensure that any evidence of corrosion problems is picked up as soon as possible. The inspection should concentrate on vulnerable and structurally significant areas such as steel-to-concrete transitions.

As well as having to resist wave loadings, offshore platforms have to be designed to resist local impact damage. The most probable cause of this is ship collision, so the splash zone
is most vulnerable. This means that such damage is most likely to affect the legs of platforms although it could also affect other parts.

It might be thought that resistance to local impact damage would be particularly sensitive to the loss of a small number of tendons if they happened to be concentrated in the critical area. However, in practice, this is not the case. The local strength of a concrete slab type element is affected by restraint from the surrounding structure and prestress and not just by the strength of the tendons in the critical area. Tests have shown that, because of this, the ultimate resistance of slab elements to impact is much greater than is conventionally assumed. This means that, even with significant loss of prestress, concrete structures are still likely to have greater strength than was assumed in design.

Some of the earlier structures were designed so that they were dependent on draw-down for their resistance. That is, they were designed to operate with a lower pressure inside than out and their calculated resistance to other loads was dependent on the resultant circumferential compression. This means that local failures that would normally only lead to leakage, could lead to loss of the pressure differential and hence make the structures unsafe. For this reason, designing structures to rely on draw-down is now considered bad practice.

Loss of draw-down may not be as serious to the integrity of these structures as their design calculations imply. Resistance to local impact damage, for example, is likely to be greater than the designers assumed. However, it is prudent to assume that loss of draw-down would be serious and therefore be particularly careful to monitor these structures for leakage.

7.2 OTHER PRESTRESS

The various types of platform vary far more in the other prestress they are provided with so it is difficult to make any general comments on the consequences of failure. The prestress may be designed to resist pressure load either in float-out, under wave load or due to the pressure differential resulting from storage of oil. Some is designed to ensure the structural integrity of base slabs distributing the weight of the platforms over the sea bed. The consequences of failure will vary greatly: some tendons are not needed at all in the permanent condition, others are vital to the integrity of the platforms, either all the time, in particular conditions of oil storage, in storm conditions or under accidental loading such as ship impact.

Frequently, the most critical load condition arises when the hydrostatic head is greatest which is normally when the concrete structure is lowered in the water to install the deck. The only recorded loss of a concrete North Sea platform, Sleipner A, arose in trials for this stage. In terms of the reliability of completed structures, critical construction cases are advantageous as they make failure under subsequent, less onerous load cases less likely.

Nearly all the prestressed elements of a platform are likely to be ductile in the technical sense: they would crack and/or show excessive deflection long before they failed. However, the location of many of the elements under water means that damage that would normally be considered highly visible may go undetected. There is therefore, as with the prestress in the legs, the possibility of failure with little warning. An exception to this is where the prestress is designed to ensure that concrete elements remain water or oil tight. A deficiency in this prestress could lead to cracking which would result in leakage long before the elements were unsafe. As long as the surface the liquid was seeping to was visible and inspected, this would give warning of deficiency. In general, though, it is clearly unwise to assume that loss of prestress would lead to visible signs of distress before it became serious. However, the sheer size of the elements, and the number of tendons they have, means that a large number of tendons would have to fail before the structure became unsafe.
8. DISCUSSION

8.1 DESIGNS OF PRESTRESSED STRUCTURES

The designs of offshore concrete structures have been listed and described under six groups in Chapter 2. It is evident that as the state-of-art has evolved, higher strength concrete has been used and designs have become more efficient and economic.

Of the 28 structures that were considered, 12 were installed between 1973 and 1977 and 16 between 1981 and 1995.

Variations between designs have given rise to differences in the degree to which the platforms have relied upon prestressing. However, the gravity base structures have typically been prestressed both horizontally and vertically in the caisson structures, with the towers prestressed by long vertical tendons. The connection between the concrete substructure and the deck has typically been made through a steel transition piece stressed to the concrete.

The design of offshore platforms has benefited from the reducing cost of computer analysis. However, the design of the Beryl A platform, one of the earliest gravity base platforms, has been found to be generally acceptable when recently re-assessed using modern codes and finite element tools. However, the increased use of computers and automatic code checking has led to problems, notably with the loss of the Sleipner A platform during trials.

The slipform construction technique has been used extensively for the concrete platforms and has been progressively improved to allow more complex construction forms. Segmental construction has also been used in offshore platforms, most extensively for the Ninian platform and the Maureen loading column. The Maureen loading column is a semi-buoyant structure, and has successfully demonstrated the ability to maintain a water-tight concrete structure over long periods of time. However, with the use of segmental construction, the Maureen loading column may present a unique opportunity for a study of the various prestress corrosion protection measures adopted in its construction.

8.2 PRESTRESSING DESIGN AND CORROSION PROTECTION

It is evident that considerable care and attention has been given to the processes involved in the grouting of the post-tensioning ducts. The construction operations are generally more onerous than in land-based structures due to the extreme lengths of the ducts, the lengths of delivery line from the pump to entry point in the duct and, consequently, the higher pumping pressures. Quality assurance has been maintained by carrying out fully representative grouting trials beforehand and cutting open the trial ducts to confirm that they are completely filled with grout. However, there is no check that the final grouting has been satisfactory.

Improvements to the grouting procedures and materials have been progressively introduced by the specialist grouting contractors. For example, part-vacuum grouting and HDPE ducting have been used in the most recent structures. Trials were carried out beforehand to confirm that the techniques were practical and effective.

Due to the size and difficulties of access, grouting procedures sometimes have to be used which would be considered unsound in land-based structures. There have been occasions when it has been impossible to install vents in long horizontal ducts. However, specifications require vents to be at intervals not exceeding 15m and experiences in land-based structures have confirmed that there is good reason for this requirement. The use of vents has been found to be helpful in indicating the presence of potential problems such as inclusion of air bubbles, residual water present in the ducts prior to the grouting operation, and partial blockages. It is also considered that venting operations are helpful in avoiding the development of voids at changes in inclination of the duct at high and low points.
Vertical U-ducts are sometimes grouted from the top of one leg so that the grout flows downhill to the low point and is pushed up the other leg. This method has been demonstrated in full-scale trials but would generally be avoided in land-based structures as it is felt that there is a lack of control in downhill grouting which could lead to the formation of voids.

The generally higher pressures required in offshore grouting are challenging as there is more likelihood of segregation occurring. This was identified at an early stage and led to the development of the more stable thixotropic grouts. Also a 'topping up' procedure was developed whereby bleed water which collects at the top of the duct can be re-absorbed and any shortfall in the grouting level made up with fresh grout.

Corrosion protection of the tendons is provided by three lines of defence; the concrete cover, the duct wall and grout. Good quality concrete has been used having cover over the prestressing system of 100mm or more. It is of course important for the concrete to have a low permeability so that chloride ions cannot get to the ducting. Although this was fairly well understood prior to the 1980's, it was then believed that concrete quality was indicated by cube strength. More durable concrete mixes and recognition of the need to measure permeability did not come until the later structures were designed. For some structures the concrete surface was coated with an epoxy material to provide added protection. Thicker steel ducts were used, particularly in the vicinity of the splash zone. The grout is usually considered to be the most important defence but it is generally overlooked that, in locations where the tendons are pulled against the internal wall of the duct, the grout is unable to provide any protection. In fact there may be unfilled interstices between individual wires and between wires and the duct wall.

Cathodic protection is applied to external steel. In early structures, the embedded steel was isolated from the external steel, albeit this was difficult to achieve in practice. In later structures the HSE Guidance Notes stated that the prestressing systems, including the ducts and anchorages, should form part of an equipotential network. There is a dearth of information about the cathodic protection of prestressed concrete structures. In relation to land-based structures, there is a general nervousness about possible side effects.

### 8.3 DESIGN CODES

Land-based concrete structures are generally designed to BS 8110 in conjunction with specialist loading codes, and bridges are designed separately to BS 5400. Offshore structures in the UK have also been designed using BS 8110 (or its predecessor CP 110) in conjunction with the Health and Safety Executive's Guidance Notes. However, in recent years, there has been a tendency towards the use of the DnV Rules for the Certification of Offshore Structures since these have been written explicitly for application offshore.

Additionally, the Health and Safety Executive Guidance Notes are no longer being maintained, the intention being that the industry should play a more significant role in the development of its own codes of practice. It would therefore appear that the DnV Rules are set for a more widespread adoption in the design of offshore platforms.

In relation to the installation of the prestressing components and the grouting of ducts, the DnV Rules appear to be rather incomplete when compared to specifications prepared for use on land-based structures. The emphasis of the Rules is towards ensuring the compliance of the components to the manufacturer's specifications and of the grout mix with that specified before the start of construction. For example, limits are set on the frequency of grout testing, although no mention is made of the test methods to be adopted, nor the limits by which a grout may be considered acceptable. It therefore seems to be the intention that the DnV Rules are used with more specific codes prepared for the prestressing and grouting procedures.

When using information from different Design Codes it is important to ensure that the respective clauses are compatible. This means that their philosophy and derivation has to be completely understood, albeit the necessary background information is not always available.

In relation to the grouting specifications, the most recent developments have been concerned with land-based structures. In consequence, the specification developed by the
Concrete Society provides the most relevant and up-to-date advice on the grouting process, the materials and acceptance tests.

8.4 GROUTING SPECIFICATIONS

In the time since construction of the first offshore structures in the North Sea, there has been considerable development and improvement of the constituent materials used in grout. Early problems of bleeding and segregation have been reduced by the introduction of thixotropic grouts. More recently, problems encountered in land-based structures have highlighted the variability of Portland cement. Different supplies of cement have been found to contribute significantly different performances to an otherwise identical grout mixture. The cement manufacturers have felt unable to provide material having a composition guaranteed to give a consistent performance. Moreover, bags of cement are supplied having weights which can vary by up to 6% of the nominal weight. In consequence users of cement in land-based structures are turning to pre-mixed pre-bagged grout so that the uncertainties of the cement properties and site mixing are eliminated. Development work has successfully been carried out in Ontario and in the UK. The Concrete Society Specification makes allowance for use of ordinary grout and special grout, the latter being pre-mixed and pre-bagged.

The pre-mixed pre-bagged grouts have improved specification in relation to volume change, flow, set time, bleed and stability. Volume change is generally specified as zero to positive (expansion) so that there can be no shrinkage from the internal duct wall. Flow properties have to be retained for at least 60 minutes to enable the longest ducts to be grouted. Allowance has also to be made for long supply lines between the pump and point of entry into the duct.

The importance of stability has only recently been recognised. Segregation can lead to upper sections of the grout forming a friable powder having no mechanical strength and providing no protection from corrosion. A test procedure and required performance for segregation are given in the Concrete Society Specification.

8.5 INSPECTION

The prestressing system is rated as being a critical element in offshore concrete structures and it is therefore necessary to be assured that it is in a satisfactory condition. In the past, inspections have been considered too difficult due to problems of access, the sheer size of the structures and the intrinsic problems associated with tendons shielded within the steel ducts. However, experience of corroded tendons in land-based structures where similar views were prevalent, have led to the development of a new approach based on a stage-by-stage investigation. This was driven by the realisation that serious corrosion can occur without any external evidence. The stages (described in Chapter 6) are desk study, preliminary inspection and, if necessary, detailed inspection and investigation. This strategy leads to the identification and targeting of the most vulnerable locations in the post-tensioning system. These locations are at positions where voids are most likely to occur in the grouted ducts at:

- changes in curvature in horizontal ducts,
- anchorages in horizontal ducts,
- at upper anchorages in vertical ducts,
- at steel-to-concrete transitions.

and at positions where the corrosive environment is most hostile:

- in the splash zone where the concrete is subjected to cycles of wetting and drying,
- immediately below the splash zone where there is a limited supply of oxygen so that intense anodes and local corrosion can occur,
- at construction joints.

Local corrosion is the term used when the corrosion is limited to a small area so that the steel can be corroded across most or all of its section. It can also occur when the supply of
oxygen is limited as in the area beneath the splash zone. By targeting the vulnerable locations, and using appropriate techniques, inspection becomes more manageable.

There is now considerable experience of inspecting tendons in land-based structures and several techniques have been shown to be practical and useful. Others are still the subject of research and development. However, many of the available techniques are either unsuitable or require development to be used underwater. To some extent this could be overcome by working from the inside of dry shafts. Additionally, inspections well below the splash zone may be considered less important since the lack of free oxygen would impede the corrosion process.

The most useful non-destructive testing methods are;

- radiography - very expensive and of limited use for very thick members
- radar - requires skilled interpretation,
- impact echo - occasionally used but still under development.

An indication of corrosion activity in the reinforced concrete can be obtained from;

- chloride profiles,
- depth of carbonation,
- electrode potentials.

These are semi-destructive in that it is necessary to take samples for the chloride and carbonation measurements and to make an electrical connection to the reinforcement for electrode potential measurements.

Direct mechanical methods that are well developed and reliable are;

- in situ measurements by stress-relief coring,
- invasive hole drilling into the ducts, supported by endoscopy and pressure-vacuum testing.

In situ stress can be measured by the technique of stress-relief coring. This is a powerful method specific to prestressed concrete and provides information about remnant prestress and hence the losses that have occurred due to factors such as creep and, if present, corroded or otherwise damaged tendons.

The most direct and unambiguous method of inspecting tendons is by invasive hole drilling which enables the tendons to be exposed and inspected. If voids are present, this can be supported by endoscopy and vacuum-pressure testing to obtain additional information. Exposure of the tendons should only be carried out after other tests have indicated that corroded tendons are likely. The sequence of tests should be tailored to suit the local requirements. In general, it is appropriate to carry out the simplest and least destructive test first. It is important when invasive tests and stress-relief coring are carried out, to ensure that the holes are left open for the least possible time and are properly filled in and sealed afterwards so that protection provided by the cover concrete is fully restored.

8.6 CONSEQUENCES OF FAILED TENDONS

In Section 7 a theoretical study was carried out of the consequences of failed tendons in the vertical legs and in other components.

It is shown that there would have to be a loss of 10% of the prestress to cause reliability to fall below the target value, and 40% before a leg could fall under loading for an unfactored design wave. Moreover, it would be necessary for the tendons to all fail in the same section which, on the face of it, is an unlikely event for fully grouted ducts. Experience in land-based structures, however, is that there have been occasions when failures have occurred in the same section. The most common location is at anchorages where grout levels have fallen back leaving the tendons unprotected. In offshore structures the anchorages of vertical tendons are more susceptible to inadequate grouting than for the horizontal post-tensioned beams used in bridge decks. Failures can also be concentrated in
the vicinity of construction joints such as those between precast segments as was dramatically evidenced in the collapsed Ynys-Y-Gwas Bridge.

Another factor that has to be considered is that it is theoretically possible for failure to occur without any obvious signs of distress becoming apparent in advance.

The most vulnerable area in a structural sense is the steel-to-concrete transition between the deck and platform legs. It follows that inspections should be made of this location.

8.7 PRESENT POSITION

The present position in concrete offshore structures has similarities to that of land-based structures in the late 1970's when the oldest were about 25 years old and there was a general belief that tendons were well protected and safe from problems. At this time, investigations indicated that about 50% of the ducts had voids present but the tendons were generally covered in a cement coating and appeared to be sound. However, a number of bridges were subsequently found to have seriously corroded tendons and, in 1985, Ynys-Y-Gwas bridge collapsed under its own weight.

Offshore structures are less dependent on prestress and it is difficult to envisage failure modes as sudden and catastrophic as Ynys-Y-Gwas. On the other hand, the marine environment is more corrosive. Recent designs have also tended to be more efficient so that the tendons are more critical to the performance. This may be partly offset by the better corrosion protection and efficiency imparted by the improvements in grout materials and grouting specifications.

From early experiences and reports of construction it is evident that there are almost certainly defects and voids present in grouted ducts of offshore structures but, in the absence of evidence to the contrary, it is being assumed that numbers are too low to be significant. As structures reach their original design life and may be required to be operated for a further period of time, it will be necessary to assess their structural condition. It follows that the post-tensioning systems should be inspected so that they can be confirmed as being in a satisfactory condition. There may be structures which are taken out of service in which case it would be possible to carry out invasive investigations of the tendons. This would provide invaluable data and a good indication of the likelihood of there being any problems in other structures.

Offshore platforms provide accommodation for several hundred people and structural failure could be catastrophic. Experience of land-based structures indicate that it is necessary to take steps to have total assurance of the integrity of the post-tensioning steel.
9. CONCLUSIONS AND RECOMMENDATIONS

9.1 CONCLUSIONS

In this review the design and performance of post-tensioned concrete offshore structures have been considered in relation to the durability of the steel tendons. Recent experience in land-based structures has emphasised the importance of having the ducts properly grouted so that the tendons are well protected. On occasions when grouting has been imperfect tendons had sometimes corroded and fractured, often without any external evidence of damage. The situation in offshore structures is rather different and the following conclusions are drawn.

1 In the first tranche of offshore structures, constructed up to 1977, considerable attention was given to the design and construction operations because of their sheer size and importance. Nevertheless, problems were experienced, for example, grouting of long vertical ducts required high pressures and bleed water collected at the tops. Blockages sometimes occurred in the ducts which were difficult if not impossible to free. It has been suggested that allowance should be made for up to 5% of the vertical ducts having blockages or being inadequately grouted. It is concluded that earlier structures now 20 to 24 years old are more vulnerable to corrosion.

2 As a result of the early problems, considerable efforts were made to improve grouting materials and procedures. Thixotropic grouts were developed having good bleeding and shrinkage characteristics. Capping was recommended for the tops of vertical ducts to prevent things being dropped into the ducts during construction. It is concluded that in later structures tendons should be better protected giving better durability performances.

3 Information about the service history of the structures has been rather varied. On the one hand, it is generally felt that performances have been very good; there have been few instances of poor grouting and little evidence of corroded tendons. On the other, there is a feeling that there is inadequate continuity of knowledge, contractors have handed over structures to the operators without contact with maintenance engineers and people have moved from project to project without being able to pass on their experience. It has also to be considered that from the experience of land-based structures corrosion can occur without any external evidence.

It is concluded that although post-tensioned structures appear to be in good condition, full inspections have not been carried out. Experience in land-based structures indicates a need to obtain added assurance that post-tensioning tendons are free of corrosion.

4 There is a dearth of information about the actual performance of cathodically protected concrete structures in the field. In land-based structures there is a general nervousness about the possible side-effects of cathodic protection on the prestressing steel.

It is concluded that there is a need to review the use of cathodic protection of offshore concrete structures and the implications of any relevant research.

5 From consideration of designs of structures it is evident that locations in the post-tensioning systems most at risk from corrosion are in the vertical ducts in the region of the splash zone, at construction joints (if present), upper anchorages and transitions between steel-to-concrete (the connection between deck and legs).

6 There is as yet no easy fully non-destructive method of inspecting tendons. Most recent developments in land-based structures are to undertake the inspection in stages, concentrating on the locations identified as being most at risk from corrosion and using a variety of methods. The final stage is an invasive inspection but this should only be undertaken with care because it is necessary to ensure that contaminants do not get into the exposed duct during the inspection and that the opening is properly filled and sealed afterwards.
It is concluded that, where there is reasonable access, it is now possible to carry out inspections of ducts and tendons but it must be done by people having adequate experience. This does not apply to locations under water, but it is usually considered that corrosion is less likely in these locations due to a lack of available oxygen.

The method of stress relief coring can be used to measure the level of prestress. This is a well researched technique that has been used on some 40 land based structures. It can measure in situ stresses to an accuracy of \pm 1 \text{ N/mm}^2 and is a very useful tool for assessing prestressed concrete.

Offshore construction practice differs from land based in a number of respects:

- in land-based construction it is considered important to have vents at relatively close spacing (up to 15m) but this is impractical in offshore structures,
- grout sometimes has to be pumped downhill into vertical ducts,
- long pumping lines and very high vertical ducts requiring high pumping pressures present special problems not encountered in land based structures,
- the sheer size of offshore structures attracts greater attention to detail in both the design and construction operations. In consequence there is a feeling that errors are less likely to occur than in the smaller and less significant land based structures.

Offshore practice has similarities with land based:

- best practice is generally given in documents developed primarily from land based experience,
- grouting is one of the less glamorous operations and generally tends to attract less attention and supervision.

In recent years the problems with land based structures have led to research activity and production of an improved specification for grouting. In contrast, there has been no evidence of similar problems in offshore structures and consequently there has been less development of the state-of-the-art.

It is theoretically possible for the prestressed legs of concrete gravity platforms to become unsafe due to loss of prestress, without showing any signs of distress which would be visible under normal loading. A failure of the structure due to this seems unlikely due to the large number of tendons which would have to fail before the structure as a whole failed. However, it does suggest that vulnerable positions, such as steel-to-concrete transitions, should be monitored and inspected so that evidence of tendon corrosion is found as early as possible. The consequences of structural failure could be very serious and involve fatalities.

9.2 RECOMMENDATIONS

1. It is recommended that if a suitable opportunity arises such as assessment for extended life or decommissioning, the condition of the post-tensioning system of an offshore platform should be investigated. Such an opportunity would enable the verification of current inspection methods. The concrete cover could then be removed so that the state of the grout and tendons could be examined directly. This would provide added assurance about the continued serviceability and strength of other post-tensioned concrete structures.

2. It is recommended that the effects of cathodic protection on prestressed concrete should be reviewed. This should include systems having the steel isolated and systems having the steel as part of an equipotential network. Information is needed on actual performance of cathodically protected offshore concrete structures. Also, and of particular importance, information is needed about possible side effects on the prestressing systems.
It is recommended that best design and construction practice, currently available in several documents and aimed primarily at land based structures, should be collated and published in a single document for use in the design and construction of future concrete offshore structures. The vehicle for such a publication could be a research report or a professional institution report such as The Concrete Society Report R47.

In the absence of any documents relevant to offshore structures, a guidance document on inspection of post-tensioned concrete should be prepared. This would be based on the stage-by-stage method of inspection leading up to invasive examination of the duct and tendons. It is very important that such a document should be properly focused on methods that are practical in an offshore environment and that shortcomings are highlighted. Many of the land-based techniques cannot be used underwater, but corrosion is less likely here due to the lack of oxygen at depths well below the splash zone.

Measurement of remnant prestress by stress-relief coring is recommended. This is an invasive method that requires special care. The core hole must be filled immediately after the measurements have been taken.

Although there have been numerous offshore conferences, there have been surprisingly few reports or papers that have addressed the durability of post-tensioning systems. It is recommended that encouragement be given to designers, contractors and operators to publish information about these topics.
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