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Review
The use of steel and concrete in the construction of North Sea oil production platforms

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The main areas of uncertainty in the materials field associated with the use of steel and concrete for the construction of North Sea oil production platforms are reviewed. The effects of corrosion from the seawater and corrosion fatigue from wave action are given particular attention. The long-term performance of steel and concrete in this environment is considered against current design and certification requirements and the need to undertake regular inspection and maintenance of platforms during their lifetime.

1. Introduction

Until the last decade, offshore oil and gas exploration was confined to relatively calm areas of water. The Gulf of Mexico has been one of the major areas of exploration and several thousand steel structures have been erected in this region where water depth is rarely more than 60 m. Most of these platforms have been welded tubular steel space-frame structures piled to the seabed.

The discovery of natural gas in the shallow southern part of the North Sea in 1965 was followed several years later by its subsequent extraction using similar steel platforms. Experience has shown that some of these were under-designed to cope with the more hostile environment in the North Sea and several have since been strengthened. This experience has been of great value in the exploitation of oil from the northern part of the North Sea which began in the early 1970s.

The first oil production platforms for the North Sea needed to be designed for deeper water, higher wind speeds and larger waves than had ever before been encountered by offshore structures. One of the first major North Sea fields to be discovered and developed is Forties, lying 180 km off the east coast of Scotland. Forties is a major North Sea oilfield with estimated recoverable reserves of 240 million tonnes and a peak production of 24 million tonnes per year [1]. Four steel piled production platforms have since been installed on this field (Fig. 1) and in mid-1978, the field is currently producing about 400 000 barrels a day worth up to £2m. Typical dimensions and weight of a Forties platform are given in Table I. It can be seen that these structures are very large compared to existing land structures (e.g. taller than the London Post Office tower). Each Forties platform contains about 25 000 tonnes of structural steel which, including the costs of construction, amounts to a total cost in the region of £50m. Current Forties production satisfies about a fifth of UK current requirements for oil. Thus, the long-term structural integrity of the four production platforms, which

TABLE I Dimensions and weights of typical offshore platforms

Steel (Forties FC) [6]
Wave depth 127 m
Production capacity 125 000 barrels/day oil
Dimensions of jacket on seabed 6640 m^2
Height of jacket section 145 m
Diameter of steel tubulars 0.5–4.3 m
Weight (less deck) 29 500 tonnes
Concrete (Brent C) [7]
Water depth 141 m
Production capacity 150 000 barrels/day oil
Storage capacity 550 000 barrels
Caisson base area 10 340 m^2
Caisson height 57 m
Weight of substructure in air 283 000 tonnes
Total height of platform 168 m

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Figure 1 Production platform FA in the Forties oilfield in the North Sea (by courtesy of British Petroleum).

depends on the quality of their materials and construction is of vital importance to both the operators' commercial interests and from the national viewpoint.

A new type of production platform built of reinforced and pre-stressed concrete has also been constructed and installed in the North Sea. These structures have not previously been used for oil production but reinforced concrete has a long history of successful use in a marine environment. The first concrete North Sea structure was the one million barrel oil storage tank for the Ekofisk field, installed in 1973. Since then a further thirteen concrete platforms have been installed or are under construction for use in the North Sea. Most of these consist of a large cellular caisson base for oil storage, supporting up to four cylindrical columns or legs to which the steel deck is attached (Fig. 2). Typical dimensions and weights of a concrete platform of this type are shown in Table I.

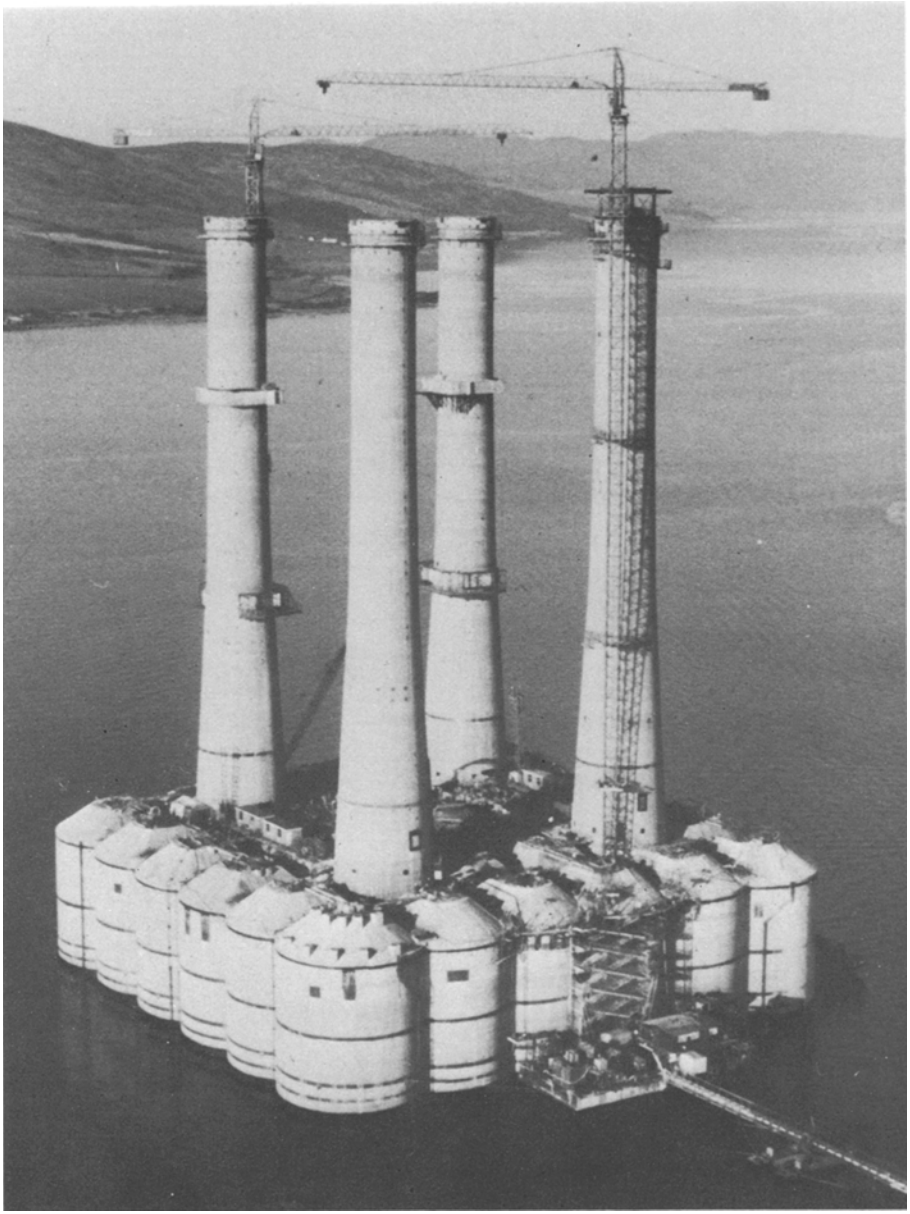

Figure 2 Concrete platform (Brent C) for the Brent field under construction at Ardyne Point in Scotland (by courtesy of Sir Robert McAlpine & Sons Ltd).

These gravity structures rest on the seabed and are not piled as a steel platform. They need consequently to be much heavier and weigh as much as ten times the weight of a steel platform and are therefore some of the largest structures ever built. It is interesting to note that a concrete platform contains about as much steel (as reinforcement and pre-stressing tendons) as an equivalent steel platform although in a much simpler form. Exploitation of the oil in the Brent field which has estimated recoverable reserves of 220 million tonnes [1] (only slightly less than the Forties field) will be by three concrete and one steel platforms.

Now that substantial quantities of oil are beginning to flow from the North Sea, attention is concentrating on the inspection and maintenance of these platforms to ensure their continuing operation for the lifetime of the field (up to 30 years). Techniques are being developed for inspection and repair of both steel and concrete platforms. Many of these involve divers or underwater vehicles and can be very costly compared to land-based operations. It is a challenging area for development and one in which the UK is well placed to take commercial advantage of any new technology.

This review will concentrate on both steel and concrete and their limitations for use in an offshore environment. The main materials factors of obvious interest are corrosion and corrosion fatigue and these two areas are given special attention. Emphasis will be on the materials and their properties in a marine environment: at least as important are the design, engineering and constructional aspects of offshore platforms but these have already received attention elsewhere [2–5].

Although this paper indicates several areas where there are gaps in understanding at present, it is important to note that present offshore structures have been designed conservatively, based on considerable experience and generally taking into account these uncertainties. Improved knowledge from continuing research and development should enable designs for offshore structures to be improved, leading to more efficient and cheaper recovery of oil. Exploration of deeper waters may well require more efficient designs, as present structures will become too cumbersome and costly when extended for such operations.

2. Environment

The North Sea is one of the most continuously hostile marine environments in the world, with winds reaching speeds up to 50 m sec^{-1} (~ 110 mph) and with waves of up to 30 m in height. In addition seawater, with its high concentration of salts, is one of the most corrosive of natural environments; its biological activity also has important consequences for structural loading and materials performance.

In the more northerly parts of the North Sea (i.e. north of latitude 56° N) where oil production is concentrated, water depths of up to 200 m (650 ft) occur. At present, the deepest water platform is on the Thistle field in 530 ft of water. At these depths, water pressure is about 17 atm. (1.72 N mm^{-2}).

The major ionic constituents of seawater are shown in Table II. The composition of seawater (particularly dissolved gases) and its temperature have marked effects on materials performance. Of particular interest are chloride ions concerning corrosion and corrosion fatigue and sulphate ions for deterioration of concrete. The waters of the North Sea have typical salinities (3.5%) for ocean water with little variation with depth and season (Fig. 3) [9]. An important factor is that at all depths because of constant mixing of surface and deeper water the water is fully saturated with oxygen (about 6.5 ml $litre^{-1}$) (Fig. 3). In other seas (e.g. parts of the Atlantic Ocean) oxygen content reduces with depth [10] with consequent reduction of corrosion rates. The amount of carbon dioxide in seawater, usually present as bicarbonate or carbonate ions, can also affect corrosion rates particularly in the presence of cathodic protection since carbonate scales are formed both on the surface and within cracks [8]. In the latter case, the plugging of the crack can slow down crack growth rates substantially (see Section 5).

TABLE II Major constituents of seawater [8] (parts per 1000)

Sodium	10.77
Chloride	19.37
Magnesium	1.30
Sulphate (as SO_4)	2.71
Calcium	0.41
Potassium	0.34
Other salts	0.20
(Total salts)	(35.1) termed salinity

Temperatures in the North Sea vary from about 16° C for surface regions in the late summer to 6° C in the winter (lowest temperatures are reached in March or April) (Fig. 3). In deeper water (below about 50 m) the temperature variation is much smaller (in the range 6 to 9° C) for all seasons.

Offshore structures are exposed to a range of environments varying from fully oxygenated cold seawater near the base, warmer summer conditions near the surface, splash zone conditions where the structure is continually wetted and dried and finally to the atmospheric zone which is exposed to salt-laden air. The parts of the structure in both the splash and atmospheric zone are particularly vulnerable to corrosion because of the plentiful supply of oxygen. Additionally, these regions cannot be protected by normal cathodic protection systems (see Section 4) and special precautions are required.

The parts of the structure carrying hot oil (e.g. risers, pipes) or adjacent to these regions can experience much higher temperatures which can enhance corrosion rates. The temperature of the oil can be 80° C or even higher when it is extracted from the seabed. There have already been instances of rapid corrosion reported for hot oil risers and in

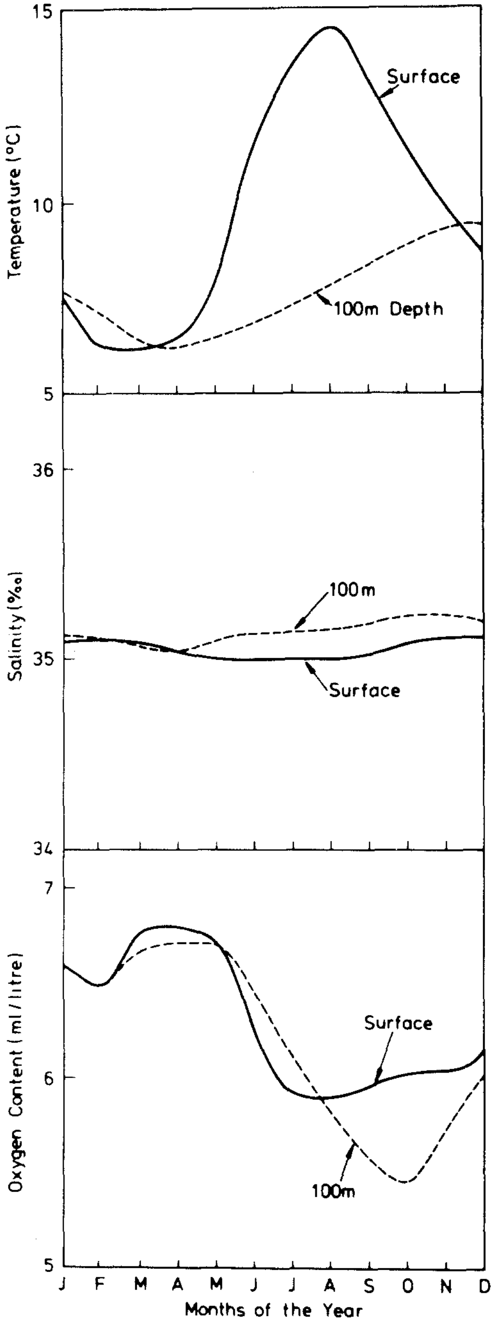

Figure 3 Variations in temperature, salinity and oxygen content of the North Sea (approximately 56° N, 1° E) through the year, both at the surface and at a depth of 100 m [9].

one case, this enhanced corrosion led to a failure and oil spillage [11]. Corrosion and the precautions taken to minimize its effects in the North Sea are discussed in more detail in Sections 4 and 6.

The wave loading on offshore platforms is of particular interest in designing against fatigue. Typical waves in the North Sea have periods in the range 9 to 18 sec, corresponding to a frequency of about 0.1 Hz. Over a 50-year life in the North Sea, the wave loading is produced by a range of wave heights up to a maximum H_L (Fig. 4) which is the design wave height for 50-year operation [12]. For the northern parts of the North Sea, H_L corresponds to a wave height of about 30 m. The number of encounters at each wave height increases with decreasing height and for small waves totals about 2×10^8 over a 50-year period. The structure is therefore subject to a spectrum of wave loading ranging from small waves up to winter storms. These need to be accounted for in fatigue life assessment (see Section 5).

3. Materials selection and certification

Rules for the design and construction of offshore structures are laid down by Government. In the UK, Section 3 of the Mineral Workings (Offshore Installations) Act of 1971 empowers the Department of Energy through its six Certification Authorities to approve the design and construction of offshore platforms for the British sector of the North Sea, and once the structures are in place, the same rules apply to issuing the Certificate of Fitness, required for continued use of the platform for oil production. The Department of Energy issues Guidance on the Design and Construction of Offshore Installations [12], which is continually updated (either by new issues or by additional notes) to take account of operating experience and

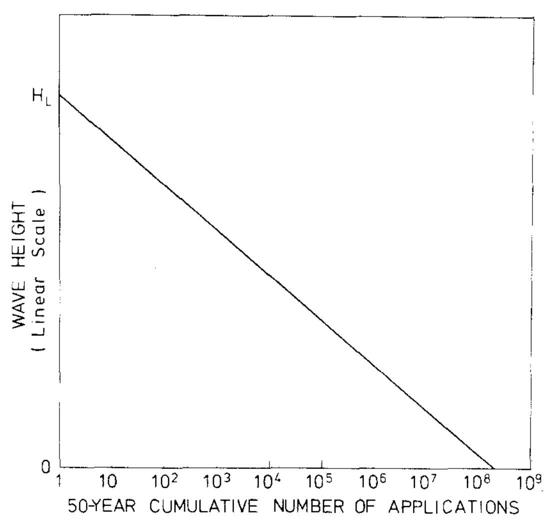

Figure 4 Schematic curve showing the number of encounters at different wave heights over a 50-year period in the North Sea.

TABLE III Specification for steels to BS 4360: Grade 50D [16]

Mechanical properties			
Yield stress (min)	Ultimate tensile stress	Elongation (min)	Charpy impact values
345 N mm^{-2} (for sizes up to 40 mm)	490 N mm^{-2}	18%	41 J at −20° C or 27 J at −30° C

Chemical analysis (%)							
	Carbon (max)	Silicon	Manganese (max)	Niobium	Vanadium	Sulphur (max)	Phosphorus (max)
(Ladle)	0.18	0.1/0.5	1.5	0.003/0.1	0.003/0.1	0.04	0.04
(Product)	0.22	0.1/0.55	1.6	0.003/0.1	0.003/0.1	0.05	0.05

the results of relevant research and development programmes. In addition, the Certification Authorities also issue their own rules [13, 14]; there is also a British Standard under development for these structures and a Draft for Development has recently been published for comment [15].

The Guidance Notes published by the Department of Energy [12] have separate sections dealing with design, materials, construction and inspection, all of which relate to the type of material for offshore structures and its service performance. The criteria relating to both steel and concrete will now be examined separately.

3.1. Steel

The main requirements in the selection of steel for primary members of offshore production platforms are medium yield strength, good weldability, resistance to brittle fracture and fatigue, which are satisfied by the requirements of BS 4360 (weldable structural steels) [16]. This standard includes a number of grades with yield strengths from 210 to 450 N mm^{-2}; most of the steel used in offshore platforms conforms to Grade 50D which has a minimum yield strength of 340 N mm^{-2} and a Charpy V notch of 27 J at − 30° C. Such steels are carbon–manganese steels, silicon-killed, grain-refined by niobium and with controlled carbon levels to achieve good weldability. The specified composition and mechanical properties are shown in Table III.

The production of large diameter tubular members from steel plate for jacket structures involves hot forming and welding. Substantial thicknesses of plate are used in local areas at node connections, with thicknesses up to 100 to 125 mm in some cases. These thicknesses are much greater than commonly used in most land-based structures and careful attention is necessary to materials properties and fabrication procedures to achieve good performance in the final welded member. Welded joints in thicknesses above 38 mm in earlier constructions, now relaxed to thicknesses above 50 mm, are given a post-weld stress relief heat treatment.

There are three main problems associated with the welded joints themselves. Firstly, good weldability is achieved by using steels with a controlled maximum carbon equivalent value* (often limited to a maximum of 0.43%) and by careful control of the welding procedures to BS 5135 [17]. This specification gives guidance on the control of preheat, weld arc energy heat input, material thickness and type of welding electrodes necessary to avoid hydrogen-induced cracking in the weld heat-affected zone.

Secondly, the problem of lamellar tearing [18, 19] at welded joints and the need for good through-thickness properties in thick members subject to stress (e.g. at complex node connections) has been substantially reduced by the use of low-sulphur steels (below 0.01% S). This problem is largely due to the presence of non-metallic inclusions aligned parallel to the plate surface during hot rolling and reduction of the sulphur content reduces the number and size of unfavourable inclusions.

The third problem associated with welded joints is the achievement of satisfactory toughness and notch ductility in the weld metal. Brittle fractures have already been experienced in the Gulf of Mexico [20] and careful consideration of this problem is necessary for North Sea applications

*Carbon equivalent = C + Mn/6 + (Cr + Mo + V)/5 + (Ni + Cu)/15 where the levels of each constituent (C, Mn, Cr, Mo, V, Ni and Cu) are in per cent.

because of the use of thicker sections and the lower air and sea temperatures. The parent material resistance to brittle fracture is dealt with at the design stage by selecting steels with good notch toughness at low temperatures [21]. The problem remains of ensuring that the weld metal and heat-affected zone at the welded joints also have satisfactory notch toughness. For quality control purposes, notch toughness in both parent material and welded joint is usually measured by the energy absorption in the Charpy V-notch impact test at particular test temperatures. Some criticisms of the Charpy-V notch test and its use for selection of appropriate materials have been made because of its failure to reproduce the conditions applied in service and its lack of correlation with other fracture toughness tests [18, 22]. In many cases, Certification Authorities and oil companies now use fracture mechanics methods and crack opening displacement tests to specify minimum fracture toughness and minimum allowable defect sizes.

The choice of materials for resistance to corrosion fatigue presents difficulties as the fatigue strength of welded high tensile steels is little better than that of welded mild steels [23], mainly because of the effects of welding. This is one of the main reasons limiting the use of steels of higher strength than Grade 50 D for offshore construction which would allow a useful reduction in thickness and overall weight. Studies of corrosion fatigue of welded joints are under way (see Section 5) and when these are completed, a better understanding of this behaviour may allow further use of higher strength materials.

Steels currently in use for construction of offshore structures are unlikely to be susceptible to stress corrosion cracking in seawater environments [22]. This problem is only significant for steels with higher yield strengths ($> 900\,\mathrm{N\,mm^{-2}}$).

3.2. Concrete

For the design and construction of reinforced and pre-stressed concrete offshore structures, the Department of Energy Guidance Notes lay down recommendations of the British Standard CP110 (Structural Use of Concrete) [24] supplemented by the recommendations for the Design and Construction of Concrete Sea Structures, issued by the Fédération Internationale de la Précontrainte [25]. The main requirement is for concrete with good durability in seawater and to provide adequate protection to the steel reinforcement or pre-stressing tendons. For concrete in and above the splash zone and for submerged zones, the characteristic strength (measured by standard cube tests) should not be less than $40\,\mathrm{N\,mm^{-2}}$, although in practice, most structures have been built with higher strength (at least $50\,\mathrm{N\,mm^{-2}}$) concrete [26]. The construction of such large structures with high strength concrete has been a new step for the construction industry. The constituents of concrete are also controlled. Several types of cement are allowed but in practice, ordinary Portland cement has been used because of the very large quantities required for offshore structures. Aggregates too, are controlled and the demand for very large quantities has meant use of local aggregates which have generally been granite-based [26]. Seawater as a constituent of concrete is not allowed because of the adverse effects of the chloride ions. A minimum cement content of $400\,\mathrm{kg\,m^{-3}}$ is specified for concrete in the splash zone as well as a water–cement ratio in the range 0.4 to 0.45. Both of these factors generally ensure good low-permeability concrete of high strength. The second factor in achieving good quality concrete is maintenance of construction standards and in particular, ensuring good compaction, as recommended in CP110. Additives are also used in offshore construction. For example, retarders are essential because of the very long distances to pump concrete, particularly when the structure is partially completed and floating offshore. Good corrosion resistance is dependent on adequate concrete cover. The Department's Guidance Notes [12] recommend a minimum cover to the reinforcement of 60 mm, increasing to 75 mm for the splash and atmospheric zones. For prestressing tendons, minimum cover is 75 mm for submerged regions and 100 mm for the more vulnerable areas in and above the splash zone.

Several types of reinforcing bar are allowed in CP110 but most offshore construction has tended to use cold-worked steel bars, covered by British Standard BS 4461 [27]. Fairly large-diameter bars (25 or 32 mm) have been used.

The standards for reinforcing steel are much less detailed than those for structural steels (e.g. BS 4360), the requirement being more performance oriented. Limits are given for minimum yield stress and for the ultimate stress to be not less than a percentage of the yield stress above this. Bend test and re-bend test requirements are included and there are limited restrictions on chemical composition. No recommendations exist at pre-

TABLE IV Specification for steels to BS 4461 [27] (cold-worked steel reinforcement) and to BS 3617 [28] (seven-wire steel strand) for pre-stressed concrete

	Mechanical properties (0.2% proof stress)				Minimum elongation (%)
Reinforcement*	460 N mm^{-2} (up to 16 mm diam)				12
	425 N mm^{-2} (over 16 mm diam)				14
Pre-stressing† steel	Values specified for total load which depends on tendon size. Approximate 0.2% proof stress is 1500 N mm^{-2}				3.5
	Chemical composition				
	Carbon (max)	Sulphur (max)	Phosphorus (max)	Silicon (range)	Manganese (range)
Reinforcement*					
(ladle)	0.25	0.06	0.06	ns	ns
(product)	0.28	0.068	0.068	ns	ns
Pre-stressing† tendon	0.6–0.90	0.05	0.05	0.1–0.35	0.5–0.9

*BS 4461: 1969 [27]
†BS 3617: 1971 [28]
ns: Not specified

sent on fatigue performance of reinforcement although some efforts are being made to introduce this in other demanding applications such as bridges. The specified composition and mechanical properties are given in Table IV.

Table IV shows a very large difference between the properties of steel used for reinforcement and that used for pre-stressing or post-tensioning cables. The reinforcing steel of the cold-worked type is normally a low carbon steel with a maximum carbon content of 0.25% (ladle). The increase in the characteristic strength is achieved by twisting the hot rolled mild steel bar to introduce cold work. Some increase in strength in these bars may also occur by strain ageing effects when, after the cold working, solute atoms (mainly carbon and on occasion nitrogen) diffuse to dislocations to cause pinning [29]. The strength levels for high yield hot-rolled reinforcement are obtained by some suppliers by the use of higher carbon contents. Such steels present additional problems as they may have poor fracture toughness and may be susceptible to brittle fracture on impact loading or reverse bending. Welding of such bars also presents problems and is not normally permitted.

Pre-stressing tendons and post-tensioning tendons have much higher characteristic strengths ($\sim 1500\,\mathrm{N\,mm^{-2}}$) and are usually made of a high carbon steel (0.6 to 0.9%) with a combination of controlled heat treatment and cold work during drawing. The individual wire sizes are kept relatively small compared to reinforcing bar material, but the wires may be assembled into multi-wire strands. With the much higher strength level, it is not possible to maintain a high level of ductility. No welding of these materials can be carried out after manufacture because of the chemical analysis, heat treatment and manufacturing processes necessary to achieve the strength level. Because pre-stressing and post-tensioning tendons can be subject to stress corrosion cracking, strict control of their storage before construction is necessary. This includes avoidance of mechanical damage, corrosion and pitting and contamination from other operations (e.g. welding) in the area (see also Section 6.4).

4. Corrosion of steel offshore platforms

The corrosion rate in oxygenated seawater of unprotected carbon manganese steels as used for structural components in offshore platforms is relatively rapid (up to 0.4 mm per annum on average, with even higher rates or more rapid local pitting under certain conditions) [8]. On immersion in seawater, such steels establish a potential of about -0.65 volts (with respect to a reference electrode, silver/silver chloride) known as the free corrosion potential. Corrosion rates differ in the different exposure zones as indicated in Fig. 5. As expected, steel in the splash zone suffers the highest rate of corrosion (increased by a factor of about four on the corrosion rate of submerged steel) because of the plentiful

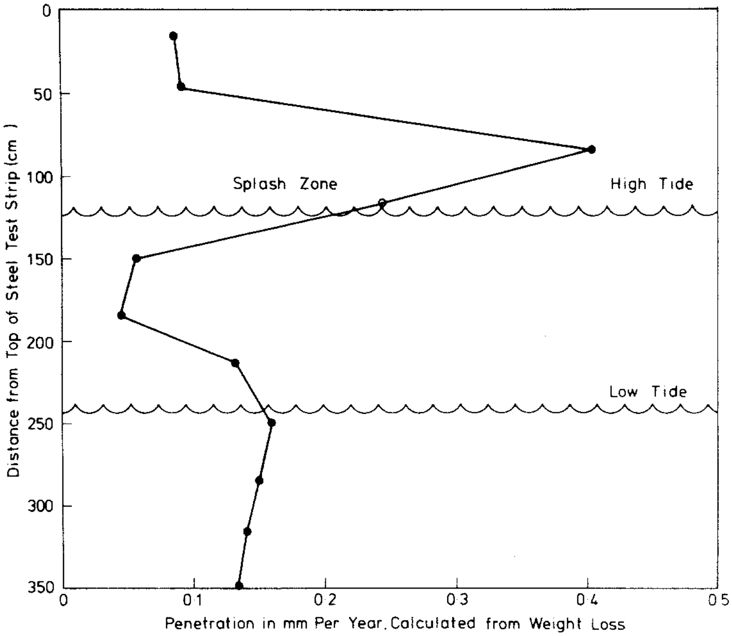

Figure 5 Variation in corrosion rate of unprotected steel exposed in different zones (after Apps *et al.* [22]).

supply of oxygen in this region. Thus, the long-term use of structural steels in seawater requires some form of protection. The submerged regions are normally protected using sacrificial anodes, an impressed current system or both, to a potential of approximately -0.85 V (with respect to the reference electrode) [30]. In a sacrificial anode protection system, external anodes (usually of zinc, but magnesium and aluminium are also used) are connected to the steel structure to force naturally occurring anodic areas on the steel surface to become cathodic, thus suppressing natural corrosion (Fig. 6). The high electrical conductivity of seawater (approximately $25\,\Omega\,\mathrm{cm}^{-1}$) [8] enables a limited number of anodes to be used for protection. Nevertheless, for a typical offshore steel platform, the total weight of sacrificial anodes could be in the region of 200 tonnes; each anode weighing typically 200 kg [30].

The second type of system in use in the North Sea is impressed current cathodic protection in which an external source of direct current provides the protection via an inert anode. Typical anode materials are a platinum-coated substrate or various lead alloys which are energized at potentials of up to 25 volts, which is a hundred times the driving voltage of a sacrificial anode system [30]. The higher operating voltage enables fewer anodes to be used, although this can give problems in poor distribution of the protecting current and the possibility of localized overprotection (i.e. cathodic protection potentials more negative than -0.85 V). One possibility to improve the current distribution is to place the anodes remotely to the structure, electrically connected via a cable.

The preferred system for the North Sea platforms has been sacrificial anodes, not only because of their long history in protecting steel in a marine environment but also because of difficulties with an impressed current system in protecting a steel jacket structure during tow-out and installation. Sacrificial anodes are normally attached before tow-out and are therefore immediately effective. The major advantages of impressed current systems are the greater flexibility of control and less weight penalty in supporting a large number of anodes and for these reasons they are being installed on some structures.

Reduction of oxygen at the cathode results in an increase in the local pH. This increased alkalinity in the solution adjacent to the surface of the cathode (the steel structure when cathodic protection is employed) can produce adverse effects on coatings such as softening and blistering of some paints and careful choice of suitable protective coatings is important. Coal tar pitch epoxy paints are favoured for offshore use because of their insensitivity to increased alkalinity [31]. The cathodic reduction of oxygen and the increase in alkalinity can also upset the equilibrium between dissolved calcium hydrogen carbonate and carbon dioxide in seawater, to form insoluble calcium

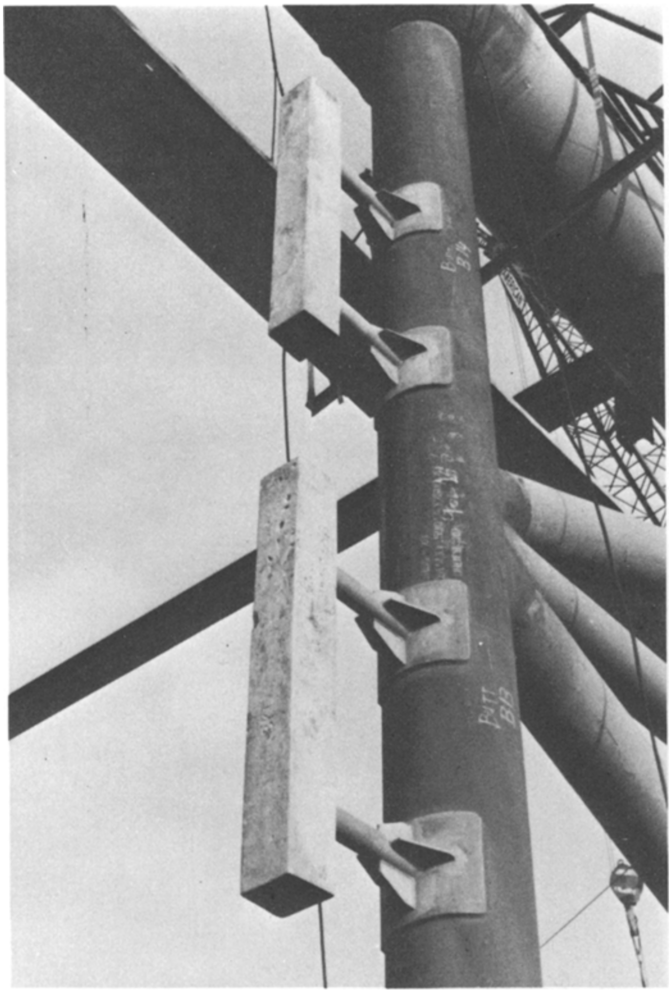

Figure 6 Sacrificial anodes attached to a brace member of a steel offshore platform (courtesy of Impalloy).

carbonate [8].

These calcareous deposits are usually beneficial by reducing the current needed to maintain cathodic protection and thereby lower the power consumption of anode materials or decrease the current requirements from an impressed current system. These deposits can also affect fatigue crack growth (see Section 5).

Correctly applied cathodic protection should result in no areas of the structure being more negative than about -0.9 volts (w.r.t. silver–silver chloride electrode). However, in areas close to anodes and particularly when an impressed current system is in operation, more negative potential can occur and higher duty coatings are required under these circumstances. Excessive negative potentials (less negative than -1.0 volts versus silver–silver chloride) make hydrogen evolution a possibility. This can affect coatings and cause embrittlement of the steel. Overprotection can also have important deleterious consequences for corrosion fatigue, as discussed in Section 5.

Some offshore platforms are uncoated in the submerged regions and rely entirely on cathodic protection. Other platforms rely on a combination of protective treatment and cathodic protection to prevent corrosion at areas where the coatings are inadequate. In the corrosion-vulnerable splash zone, cathodic protection is unsuitable because of incomplete immersion in seawater. This zone, therefore, with ready access to oxygen, frequent wetting and vulnerability to mechanical damage, is the critical region from the corrosion viewpoint. At the design stage, a generous corrosion allowance (of up to 20 mm) is normally provided for splash zone members. In addition a coating (frequently an epoxy) is normally provided to limit access of seawater to the bare steel. The coating systems on the early gas production platforms did give trouble with premature breakdown, sometimes after only one or two years [32]. A number of factors were found to be responsible. These included inadequate surface preparation, incorrect application of coatings (too porous and uneven in thickness) and the general problem of quality control. Stricter specifications and improved site control have removed most of these difficulties for the more recent larger platforms for the northern North Sea. The recoating of areas where failure has occurred has also led to some problems mainly because of the limited weather window for this operation, which can be only two to three months in the summer. The expected life of current protective coatings is considerably less than the life of an offshore platform. Coatings are inspected frequently as part of the statutory certification process. Maintenance and repair will become increasingly important as the platforms approach their design life of 25 to 30 years.

The most important areas of a platform from the corrosion viewpoint are the risers carrying hot oil from the seabed. The submerged regions of the riser are normally carefully cathodically protected, but the splash zone section is particularly vulnerable to corrosion.

In November 1975 an accident occurred on the Ekofisk Alpha platform when three men were killed and the platform badly damaged by an explosion and fire initiated by the fracture of a riser pipe (250 mm in diameter) which had corroded near the waterline [11, 33]. In this case, the riser had been clad with bitumen coating and concrete which had become damaged. The subsequent high rate of corrosion (10 mm in a few months) surprised many offshore engineers and has not been fully explained, although the difference in temperature between the hot oil (up to 80° C) in the riser and the cold seawater is believed to have played an important part. The consequences of accidental damage to materials through ship impact and other causes needs careful consideration at the design stage. The corrosion of a hot tube immersed in cold seawater is not fully understood at present and research funded by the Department of Energy is currently underway to provide more data in this area. In the immersed region of the riser, the flow of heated seawater to the surface around the riser modifies the local corrosion conditions (oxygen content, alkalinity, etc.) in some circumstances leading to enhanced corrosion rates. In the splash zone, current practice is to clad this section with Monel sheeting, which is attached by welding to the steel riser [34]. Monel alloy (approximately two-thirds nickel, one-third copper alloy) cladding reduces the corrosion rate to very low levels. Results obtained over a 25 year period on the corrosion behaviour of Monel-sheathed steel piling showed corrosion rates as low as 0.002 mm per year, for normal seawater conditions (Fig. 7) [35]. The corrosion rate for the hot riser in cold water conditions is expected to be somewhat higher than this. Monel cladding has also been used on tubular members in the splash zone sections of the Ninian northern platform, to minimize corrosion and this could well be the trend for the future corrosion protection of platforms in critical corrosion regions.

5. Corrosion fatigue of steel offshore structures

5.1. Introduction

The fatigue resistance of the main welded tubular connections where the bracing members meet the platform legs is one of the major limiting factors in designing a steel structure for use in the North

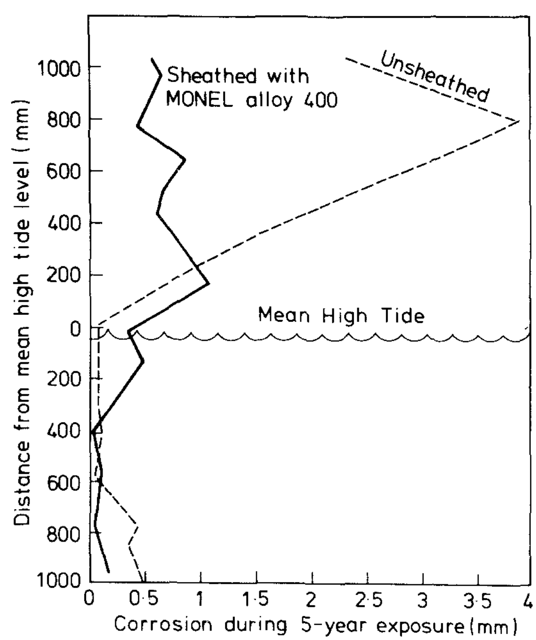

Figure 7 Corrosion of unprotected and MONEL alloy-clad steel pilings over a 5-year period of exposure (courtesy of Henry Wiggin & Co. Ltd).

Sea [18]. The number of wave encounters on a platform in the lower stress range is increased substantially for North Sea operation compared to the Gulf of Mexico and the high cycle–low stress region contributes significantly to the cumulative fatigue damage of these tubular connections [36, 37]. Offshore structures are subject to a range of wave loadings during their lifetime (see Fig. 4); over a 50-year period, the total number of waves affecting a platform can reach 2×10^8. In designing a structure to resist fatigue, consideration must be given to cumulative fatigue damage allowing for all the different stress ranges and their total number of encounters during the platform history. Cumulative damage is normally allowed for in design by assuming a linear summation of the different stress cycles based on the Palmgren–Miner rule [38], i.e. cumulative damage ratio $= \Sigma\, n/N$ where n is the number of cycles applied at a given stress range and N is the number of cycles at that stress range corresponding to failure. Fatigue failure under a varying stress history is assumed to take place when the damage ratio reaches unity. In normal design practice, a substantial safety factor is usually included (e.g. limiting damage ratio of ~ 0.2).

Design assessments using the above type of analysis show that fatigue in welded steel offshore structures is mainly a high cycle fatigue problem and most damage is caused by the loading from the larger number of smaller waves [36]. In comparison, the occurrence of a few severe storms with return periods of greater than a year are less important in consideration of fatigue damage.

At the time that many of the steel structures currently in position in the North Sea were designed, the data available on fatigue performance was limited to air tests on relatively small specimens fabricated from flat plates. As a result of the extrapolation in scale and environment needed for application of the data to offshore structures, designers were forced to be very conservative. The situation has improved since that period and several major research programmes are now producing data obtained from tests in seawater and at a scale commensurate with full-scale tubular members. In particular, the UK Offshore Steels Research Project [39] which commenced in 1975 includes tests on a variety of joint configurations, the largest of which are 2 m in diameter and 100 mm in thickness, close to full scale. The programme also includes theoretical and experimental programmes to establish the local stress patterns and stress concentration factors in welded joints. Basic studies of the fatigue process have also been undertaken to enable the test parameters for the larger welded specimens to be defined.

Corrosion fatigue in structural steels has received considerable attention in the past and recent reviews [40–42] indicate the current state of the art. In the limited space available in this Section, emphasis will be placed on crack growth tests using fracture mechanics as a means of understanding the mechanism of corrosion fatigue of steels in seawater. The performance of welded tubular joints depends on their design, fabrication and materials properties. These factors have been considered in some depth elsewhere [36, 37, 43, 44].

5.2. Background to fatigue in welded connections

The present knowledge of welded steel structures relevant to fatigue is briefly described here to provide a background to materials aspects of the problem.

In welded structures, small sharp defects are unavoidably present in the welds after fabrication and act as crack initiation points for fatigue [45]. The larger defects are normally removed by repair following non-destructive inspection after fabrication. Nevertheless, the remaining small defects can act as fatigue initiation sites, particularly at the toe of fillet or T-butt welds. During the lifetime of the offshore structure fatigue is therefore predominantly concerned with fatigue crack propagation [46]. The major factors affecting the growth of fatigue cracks are the applied stress range and the size of the defect or crack present. In offshore structures, the main source of stress fluctuations is from wave action and the magnitude of the stress range varies according to the forces from wave action. A second factor of importance is the level of mean stress about which the stress fluctuations take place. In welded structures, high tensile residual stresses are inevitably present after welding as a result of cooling and contraction of the weld metal [18]. These residual stresses, together with the dead load of the structure have a major effect on the mean stress level and hence affect the fatigue crack propagation rates and fatigue life. Another important factor affecting fatigue performance is that the shape of a welded tubular connection induces high stress concentrations [47]. Local stress analysis has shown that increases in the applied stress of up to a factor of six can exist at the toe of a weld [47]. Small flaws already present in this region can propagate through the parent metal or heat-affected zone (HAZ) to create a fatigue problem.

The seawater environment, together with cathodic protection, also has major effects on the fatigue performance. Simple fatigue tests indicate the absence of a fatigue limit (at least up to 10^8 cycles) for steels in seawater [41] and all wave encounters are therefore damaging on this basis. Other work mainly in the fracture mechanics area, to be discussed later in this Section, indicates that there may be a threshold stress level below which crack growth does not occur. The synergistic effects of both corrosion and fatigue acting on structural steels in seawater result in much reduced fatigue properties compared to performance in air. There is some disagreement at present about the effects of cathodic protection in restoring fatigue performance; some experiments indicate little improvement for cathodically protected specimens. This is an area requiring further research.

5.3. Fracture mechanics approach to corrosion fatigue

As discussed previously, fatigue in welded structural steels in seawater is concerned mainly with crack propagation rather than crack initiation. Fracture mechanics tests can therefore be used to study the fatigue process and its dependence on environmental factors, load range, cathodic protection potentials, etc. Typical tests involve use of relatively small notched specimens, allowing calculation of the stress intensity at the fatigue crack tip [48, 49]. These specimens are cyclically loaded and the extension of the crack monitored as a function of the number of loading cycles. Crack extension per cycle ($\mathrm{d}a/\mathrm{d}N$) is normally plotted as a function of the stress intensity parameter, Δk, following the Paris relationship [50], where

$$\frac{\mathrm{d}a}{\mathrm{d}N} = c(\Delta k)^m$$

and $\Delta k = \Delta\sigma(\pi a)^{1/2}$ where $\Delta\sigma$ is the cyclic stress range and a is the crack length. c and m are experimentally determined materials constants. In practice, three regions are usually found in the fatigue crack growth curve [41] which correspond to sections of the standard S–N curve (Fig. 8). In the low stress intensity region (Region I), there is a threshold stress intensity below which crack growth does not occur; this corresponds to the normal fatigue limit in the S–N curve. It is Region II where the Paris relationship holds and the slope m of this region in the fatigue crack growth curve is the negative reciprocal of the S–N curve in the intermediate region. At high values of the stress intensity (Region III), cracks rapidly propagate to failure. There is some correspondence between this region and the low cycle part of the S–N curve.

An important consequence of the power growth law for fatigue cracks (m in the Paris relationship usually has values between 3 and 5) [41] is that for most of their life their size is small. This is confirmed by experimental measurements of crack growth during fatigue testing [41]. Consequently, the important regions of the crack growth curve are Regions I and II. Unfortunately in these regions, and particularly in the former, experimental measurements are very difficult because of very small crack extensions per cycle. Another consequence of the long period of small crack size during fatigue loading is that monitoring of structures during their service life is difficult.

Fracture mechanics testing has played an integral part in the UK Offshore Steels Research Project [39] and has been used to determine the

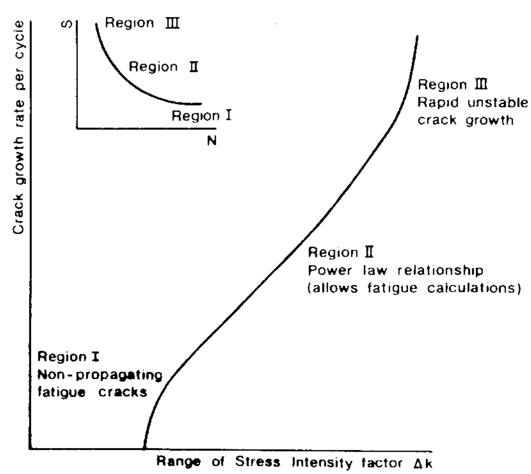

Figure 8 Main features of a fatigue crack growth curve as a function of stress intensity factor. The three regions indicated are also shown on the inset S–N curve.

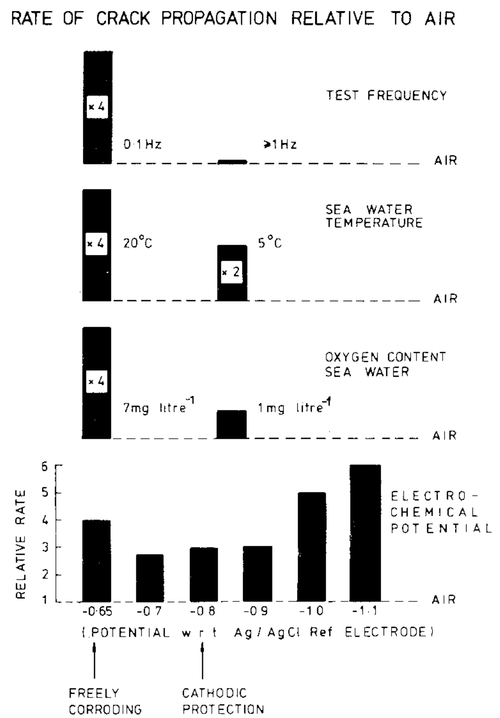

Figure 9 Rate of fatigue crack growth (relative to air) in BS 4360 steel as a function of several environmental conditions (after Scott [51]).

sensitivity of fatigue crack growth to a range of environmental parameters, which are summarized in Fig. 9, presented as crack growth rates relative to air performance [51].

It will be seen that the rate of crack growth in seawater is sensitive to test frequency and although for typical wave frequencies (0.1 Hz) the crack growth rate is increased by a factor of four on average compared to air tests, at higher rates of loading (~ 1 Hz), there is little effect of the seawater environment. Thus it is difficult to accelerate the fatigue testing of components in seawater. The lack of any effect at higher test frequencies indicates that a diffusion mechanism is important in corrosion fatigue, possibly hydrogen diffusion at the crack tip. Possible mechanisms for corrosion fatigue of steels in seawater are discussed later.

The effects of seawater temperature and chemistry on crack growth rate shown in Fig. 9 also point to the need for careful control of the seawater environment during fatigue testing of larger components. For example, particular care needs to be given to ensuring fully oxygenated seawater ($7\,\mathrm{mg\,litre^{-1}}$) during testing for proper comparison with real North Sea conditions (see Section 2).

The effect of cathodic protection on fatigue performance is of considerable importance for offshore structures. At the free corrosion potential (-0.65 V), crack growth rates in seawater are on average a factor four higher than in air. At the normal cathodic protection potential of -0.85 V crack growth rate is lower than when freely corroding but still enhanced (by about a factor of three) compared to air tests. At more negative potentials where there is overprotection, crack growth rates begin to increase and at -1.1 V are six times the air growth rate. This is attributed to the effects of hydrogen evolution and clearly should be avoided in practice. Impressed current systems with a limited number of anodes need careful design and monitoring to ensure that overprotection does not occur (see Section 4).

Other investigators using crack growth tests [49, 52, 53] have generally confirmed these UKOSRP results although some tests reviewed by Jaske *et al.* [53] are more optimistic concerning the effects of cathodic protection on reducing fatigue crack growth rates. A recent review by Scott [54] compares the crack growth data for carbon-manganese steels obtained from several major programmes involving a range of experimental conditions.

The influence of mean tensile stress on fatigue crack growth rates is of interest because of residual stresses in welded structures. The mean stress is normally represented by the stress ratio which is the ratio of minimum to maximum stress in the fatigue cycle. In air tests, the crack growth rate in the region described by the Paris equation, is only slightly dependent on the stress ratio and this is therefore normally discounted [41]. The threshold for crack growth is, however, dependent on stress ratio [41]. In a seawater environment, crack propagation rates are increased (relative to air) by increasing positive stress ratios (tensile cycles), as shown in Fig. 10 which represents cathodically protected specimens (potential of -0.85 volts). Crack propagation rates in seawater are increased on average by a factor of three compared to air tests. The upper bound (solid line in Fig. 10) represents the maximum effect of stress ratio which appears to saturate at values of R between 0.5 and 0.7. One of the problems of relating fracture mechanics data to standard fatigue tests on welded joints is knowledge of the

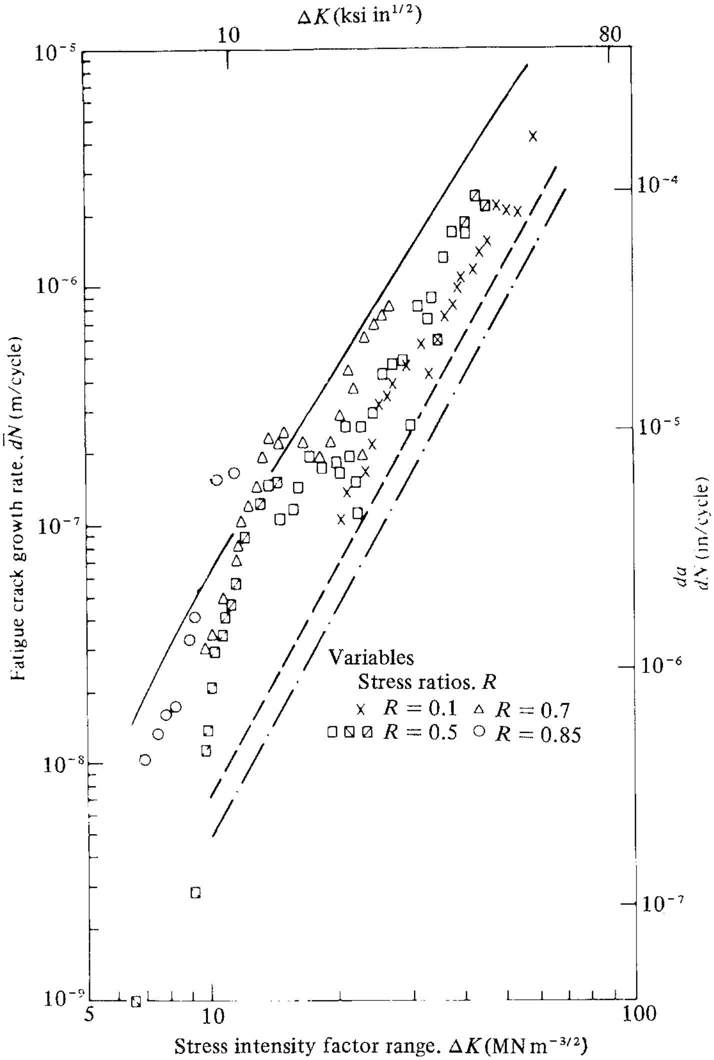

Figure 10 Crack propagation data for BS 4360 steel tested at several different stress ratios and at an electrochemical potential of −0.85 volts (under cathodic protection). The top curve has been drawn as an upper bound for the seawater tests. The two lower curves indicate the range of crack growth rates in air (after Scott [48]).

stress ratio in the latter tests. Costly stress-relieving heat treatments of critical nodes in several offshore structures have been undertaken to reduce residual stresses, particularly when combined with poor weld metal toughness [55].

Several investigators [48, 56] have observed that fatigue cracks become plugged by precipitated calcareous scales from seawater which is encouraged by cathodic protection (see Section 2). These scales consist mainly of calcium carbonate and magnesium hydroxide. The main effect is to slow down crack propagation rates with a consequent gain in fatigue life. It is not clear, however, that laboratory tests accurately reproduce the offshore environment which involves marine fouling and the possible presence of additional trace elements. Both of these factors are known to influence the structure and rate of growth of calcareous deposits [8].

Determination of whether there is a threshold stress intensity level for crack growth is important because of the use in design of linear summation methods for fatigue damage assessment from wave loading. The experimental measurements are difficult because of the need to measure very low crack growth rates ($< 10^{-9}$ m per cycle); no crack growth has been detected at stress intensity levels below about 3 to 5 MN m$^{-3/2}$ in air at a high stress ratio using standard notched specimens [48], but more sophisticated techniques [57] have indicated a lower bound threshold of 2 MN m$^{-3/2}$.

In seawater, tests to detect a threshold are even more difficult because of crack plugging by calcareous scales. In some cases, this results in apparent thresholds created by the test procedure [48]. It is thought, however, that crack growth thresholds for steels in water are similar to the values observed in air [48]. Integration of crack growth data allowing for a threshold in the region of $2\,\mathrm{MN\,m^{-3/2}}$, to produce a standard S–N curve shows reasonably good agreement with data obtained from testing welded joints [58].

The mechanism of accelerated corrosion fatigue in structural steels is not well understood but is generally accepted to be associated with hydrogen interaction with plastically deformed metal at the crack tip or anodic dissolution [48, 59]. An important factor in favour of the former mechanism is that fatigue crack growth studies in environments containing H_2S [59] show similar results and fracture surfaces to specimens tested in a seawater environment. In the tests conducted in a H_2S environment, test frequency has much less effect than for seawater tests and this is explained by the much faster rate of hydrogen supply to the crack tip in these circumstances. The rate of hydrogen availability at the crack tip is dependent on the specimen potential and, in conditions of cathodic overprotection (Fig. 9), the crack growth rate is increased relative to a standard protection voltage of -0.85 volts. The hydrogen level at the crack tip may also be dependent on additional factors such as the position of a fatigue crack in relation to other cracks and sacrificial anodes etc. These are difficult to reproduce in laboratory tests.

The microstructure of the steel is generally not thought to have a major effect on fatigue crack propagation. Some recent tests, however, on a similar steel have shown that low frequency corrosion fatigue crack growth rates in water at 260° C are sensitive to microstructure [60]. Further research is required to establish whether the effect is also relevant to a seawater environment.

6. Durability of reinforced and pre-stressed concrete

Good quality concrete is very durable in seawater and this accounts for its widespread use under marine conditions. There is, therefore, a long history of the successful use of reinforced concrete in a marine environment ranging from 50-year-old wharves and jetties [61], 30-year-old concrete barges still in use [62], the Nab tower built in 1918 and wartime forts still in existence [63].

The first part of this section deals with the effect of seawater on concrete itself; the following parts consider the corrosion of the steel reinforcement embedded in concrete, which is considerably more important as a deterioration process for concrete structures, and the corrosion of steel attachments to the structure.

6.1. The effect of seawater on concrete

Concrete is both porous and complex chemically and over a period can be penetrated by seawater. Of the major ions present in seawater listed in Section 2, it is the sulphate and chloride which are most likely to react with certain phases in the concrete. Sulphate attack is the most usually considered because of the expansive reaction between the tricalcium aluminate ($Ca_3Al_2O_6$ – abbreviated to C_3A) phase of Portland cement and sulphate ions to produce calcium sulphoaluminate [64, 65]. This reaction can produce progressive distintegration of the concrete. In many circumstances where sulphate ions are present (e.g. certain types of soil) use of a sulphate-resisting cement is recommended which has a maximum C_3A content of 3.5% [64]. In ordinary Portland cements (OPC), C_3A contents vary from about 8 to 14% [66]. However, previous experience [64] with OPC concrete in a marine environment, particularly below water level, indicates that the need for sulphate-resisting cements is less important for marine structures, although the Department of Energy Guidance Notes do recommend a maximum C_3A content of 12%. An important factor is the high chloride content also present which increases the solubility of the products of the sulphate reaction, reducing their expansive effect in the concrete. The addition of pulverized fuel ash (PFA) to cement also has a beneficial effect in reducing the overall C_3A content in relation to the amount of cement [64], because of the pozzolanic action of the PFA, and several concrete offshore platforms are constructed with PFA additions to the concrete.

The chloride ions in seawater can lead to the formation of chloroaluminates [64] (reacting with the tricalcium aluminate phase) which also occupy a greater volume than the parent phase, leading to disruption of the concrete. Nevertheless, in good quality concrete in contact with seawater, this has rarely been found to be a problem. The trapping of chloride ions by the C_3A phase can have a marked effect in delaying the initiation of corrosion and loss of passivation of the steel reinforcement (see Section 6.2). Although existing Codes of Practice do not include a recommendation for a minimum level of C_3A, some work [67] indicates that 5% C_3A is necessary to ensure no loss of passivation due to chloride ingress during the normal lifetime of good quality concrete.

A novel feature of offshore construction is that concrete is exposed to high pressure seawater (e.g. 21 atm. (2.1 $N mm^{-2}$) at a depth of 200 m). Some tests have been undertaken of concrete exposed to much deeper water for several years without evidence of any significant deterioration; of these the most well known are those undertaken by the Civil Engineering Laboratory at Port Hueneme in California as part of the US Navy's Deep Ocean Technology programme [68, 69]. Large diameter spheres (170 cm diameter with wall thickness of 10 cm) have been exposed for periods up to five years at depths ranging from 650 to 1600 m. Some leakage of water into the spheres took place over this period. After recovery, tests have indicated no evidence of any deterioration of the concrete properties with, in fact, the strength having improved slightly. Concrete exposed at depth showed a decreasing permeability rate which has been explained as a result of continuing hydration and plugging of pores by organic and inorganic debris.

No results of work have been reported to date of *reinforced* concrete exposed at depth, although some tests at 150 m in Loch Linnhe are in progress as part of the "Concrete in the Oceans" programme [70] which started in 1976 and is funded by the Department of Energy and about twenty UK companies with offshore interests.

A novel feature of concrete oil production platforms is their oil storage capacity in the caissons at the base of the platforms. In the early days of their design some doubts were expressed about the effects of hot oil on the concrete, both from the cracking viewpoint and from chemical attack of the oil on the concrete. Several investigations [71, 72] have since been undertaken of the effect of hot oil on the concrete and the general conclusion is that hydrocarbons do not react chemically with high strength concrete to alter its physical properties. A typical result shows only minor changes (~ 10%) in compressive strength observed after immersion in oil for periods up to 6 months. Research undertaken as part of the "Concrete in the Oceans" programme [73] has also indicated that under normal operating conditions, cracking due to local temperature effects is unlikely to be a problem.

6.2. Corrosion of reinforcing steel embedded in concrete

The highly alkaline environment provided by the concrete cover generally protects the steel reinforcement against corrosion. In a few isolated cases, however, corrosion has occurred resulting in cracks in the cover parallel to the reinforcing bars due to the tensile stresses produced by corrosion products forming on the bar. Eventually the cover spalls off, exposing bare steel to the marine environment with a substantial increase in corrosion rate. In offshore structures, these problems can lead to costly repairs or premature failure. The exact conditions under which corrosion occurs are not well understood at present. Many investigations have been undertaken in surveying existing marine structures [74, 75, 61] and undertaking laboratory investigations [76–79] to elucidate the mechanisms of corrosion failure in reinforced concrete. Recent problems in the Middle East have also focused attention on this.

Well-hydrated Portland cements have a substantial content of calcium hydroxide (up to 30% of the weight of the original cement) which can maintain a local pH within the range 12.5 to 13 [80] in the vicinity of the steel reinforcing bar. Under these conditions, a passive film forms on the steel surface protecting the reinforcement against further corrosion.

Breakdown of this passive layer can, however, be caused by the ingress of aggressive ions such as chloride ions from seawater or by carbon dioxide during atmospheric exposure. In the latter case, the process of carbonation is usually very slow and over the lifetime of an offshore structure, it is unlikely that the recommended cover for reinforcement (75 mm) will be fully carbonated, except possibly in the region of cracks. Ingress of chloride ions is the dominant process for marine structures and even for the high qualities of concrete used offshore, it is accepted [81] that chloride ions will penetrate to the reinforcing steel in a relatively short period, a few years or less depending on the local environment.

Under certain conditions not yet properly defined, electrochemical corrosion of the embedded steel can develop. The driving force for the galvanic cell can arise from differences in concentration of salts and oxygen due to, for example, local variations in the permeability of the concrete cover [76–78]. Iron is dissolved away at the anode by metal ions (Fe^{2+}) leaving the surface of the steel reinforcement. Excess electrons (2e) are produced which pass to the cathodic region to combine with water and oxygen to form hydroxyl ions (OH^-), Fig. 11. The anodic and cathodic reactions are shown below:

$$\text{Anode: } Fe \rightarrow Fe^{2+} + 2e$$

$$\text{Cathode: } O_2 + H_2O + 2e \rightarrow 2OH^-.$$

The negative hydroxyl ions migrate to the anode to form ferrous hydroxide ($Fe(OH)_2$). In the presence of oxygen, this corrosion product can be further oxidized to, for example, ferric oxide

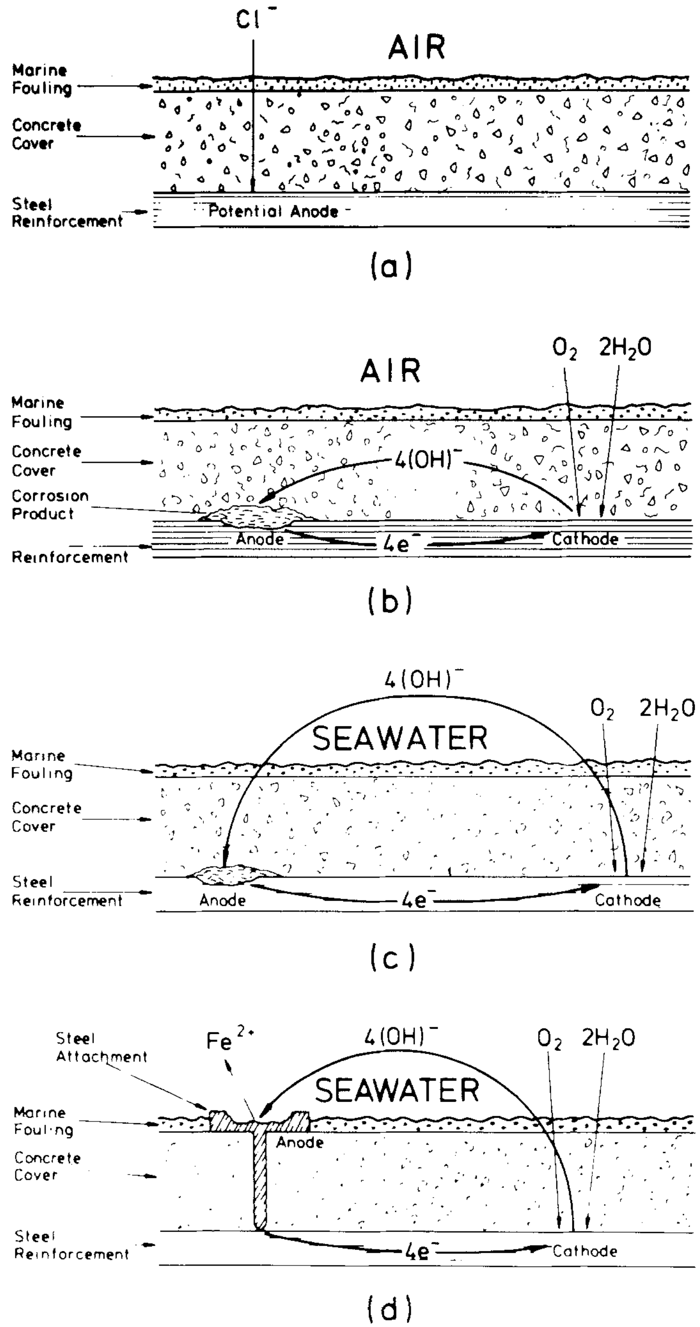

Figure 11 Schematic representation of electrolytic corrosion of steel reinforcement embedded in concrete. (a) Initiation of corrosion; (b) corrosion in the splash or atmospheric zones; (c) corrosion reactions in immersed concrete; (d) corrosion of steel attachment to reinforced concrete.

($Fe_2O_3.H_2O$) which is one of the normal corrosion products of iron. In general, it is believed that the corrosion rate of this galvanic cell is determined by the supply of oxygen at the cathode and if the cathodic area is covered by dense concrete cover, then diffusion is slow and the cell is restricted by cathodic polarization [77, 82]. The splash or tidal zones have a plentiful supply of oxygen compared to immersed concrete and this area is found to be generally the most vulnerable to corrosion. In the splash or atmospheric zones, corrosion cells are likely to occur locally at, for example, areas of poor concrete cover with anodes and cathodes separated by short distances (Fig. 11b) involving relatively easy electronic and ionic conduction paths. The concrete cover is the main path for hydroxyl ions to migrate from cathodic to anodic sites and the resistivity of this clearly limits the separation of effective anodes and cathodes. In contrast to these microcells, there is the possibility in *immersed concrete* of long-range cells with anodes and cathodes separated by large distances (many metres) because the low resistance seawater can act as an ionic path for the cell (Fig. 11c).

Concrete structures in a marine environment are normally covered by a layer of marine fouling, the constitution and thickness of which depends on water depth, seawater chemistry and temperature etc. This acts as a barrier to the diffusion of oxygen and chloride ions through the concrete cover (Fig. 11) and it is possible that the fouling introduces local changes in water chemistry which could affect corrosion of the reinforcement. This area has so far received little attention but a recent review [83] indicates some of the problem areas particularly for steel offshore structures.

Several chemical and physical factors are known to affect the efficiency of the galvanic corrosion cell; each of these will be discussed below in more detail, with particular relevance to their effect on local corrosion sites in the splash zones or long-range corrosion cells in immersed concrete.

6.2.1. Permeability of the concrete cover

This is recognised [77, 78] as one of the major factors controlling the corrosion of steel reinforcement by limiting diffusion of both chloride ions to break down the passivity at anodic areas, Fig. 11a and oxygen to fuel the corrosion reaction at the cathode (Fig. 11b).

The movement of water, oxygen and chloride ions through concrete is not well understood at present and no single parameter exists which relates the protection afforded by the cover to the more usual parameters used to define the quality of concrete (e.g. compressive strength, water–cement ratio, degree of compaction). The extent of the problem is shown by results from Browne and Domone [78], who in recognizing the importance of the permeability of the cover show results for the coefficient of permeability for several low water–cement ratio concretes (0.4 to 0.5) which range over five orders of magnitude (10^{-10} to 10^{-15} m sec^{-1}), Fig. 12. These authors measured permeability by monitoring the rate of flow of water through discs of concrete across which a pressure head was maintained. This process is likely to be dominant for initially unsaturated concrete but once saturated, the main diffusion process is likely to be the transport of oxygen or chloride ions through the water held in concrete pores. There is also evidence [84] that the outer surface of the cover is considerably less permeable than the bulk of the cover, par-

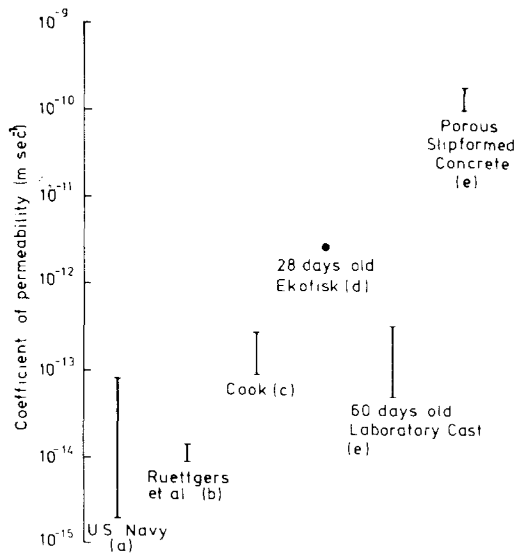

Figure 12 Range of measured permeabilities for concrete of similar water–cement ratios (0.4 to 0.5). Data obtained from the following: (a) H. H. HAYNES and L. F. KAHN, US Naval Civil Engineering Laboratory Technical Report R-774 (1972); (b) A. RUETTGERS, E. VIDAL and S. WING, *Proc. Amer. Conc. Inst.* **31** (1935) 382; (c) H. K. COOK, *Proc. Amer. Soc. Test. Mater.* **51** (1951) 1156; (d) H. MARION and G. MAHFOUZ, *Proc. Inst. Civil Engrs.* **56** (1974) 497; (e) Taylor Woodrow data. (After Browne and Domone [78].)

ticularly after exposure to seawater. This is believed to be due to the precipitation of salts in the pores of the thin surface layer of the concrete.

This is clearly an area requiring further research, particularly to relate permeability of the concrete cover to parameters which can be monitored during construction or subsequently in service. It is recognized that this will not be easy as there are many complicating factors such as the dependence of permeability on the age of the concrete, type of aggregate, degree of compaction etc. as well as the difficulties of measuring permeability in the field.

6.2.2. Cracking in the concrete cover

Design of reinforced concrete structures allows for a certain degree of cracking in the concrete members subject to tensile stresses. The various design codes treat cracking in different ways and, for example, the CP110 [24] code indicates that the maximum crack width should not normally exceed 0.3 mm anywhere on the surface.

Thus, under service conditions, cracks are normally present in concrete members and since these penetrate to at least the first layer of steel reinforcement, these are paths for ingress of seawater and oxygen (Fig. 13). As part of the "Concrete in the Oceans" programme, Beeby [85] has recently reviewed published exposure data on the relationship between cracking and corrosion. This has included tests on close to 500 specimens for periods ranging from 2 to 15 years. On the basis of this data, he proposes that the life of a marine structure in or above the splash zone can be expressed as the sum of two periods, the first of which, t_0, is the time from construction to initiation of corrosion, the second, t_1, is the time from initiation of corrosion to the occurrence of unacceptable corrosion damage (usually considered as major spalling of the cover). Thus, in designing a structure, the following relationship should be satisfied;

$$t_0 + t_1 \geqslant \text{design life of the structure.}$$

The presence of cracks enables the more rapid penetration of chlorides towards the steel (Fig. 13) thus reducing the initiation period t_0, and exposure data indicates that the rate of penetration and therefore t_0 is a function of crack width. However, in a splash zone environment when depassivation is believed to result from chloride penetration, t_0 is likely to be small when compared to the design life of the structure.

Experimental data also indicates that t_1 is not significantly influenced by crack width and, since t_1 dominates the design life of the structure, Beeby concludes that for the environmental conditions he has considered and accepting that corrosion initiates more rapidly in cracked concrete than in uncracked, control of crack width in design is unimportant in controlling the rate of corrosion. This clearly can have important effects on the design of structures. From the materials standpoint, it emphasizes the need to understand the processes controlling the rate of corrosion rather than the initiation of corrosion, although since most concrete structures do not suffer deterioration, the factors which determine vulnerable corrosion sites also need identifying. As discussed previously, oxygen diffusion to the cathode is generally regarded as a major factor in controlling the rate of corrosion under marine conditions.

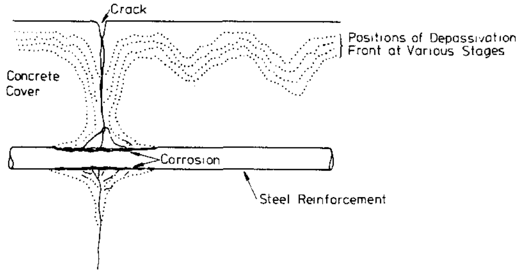

Figure 13 Schematic picture showing the advance of the depassivating front in the region of a crack in concrete cover to reinforcing steel (after Beeby [85]).

6.2.3. Resistivity of the concrete cover

The electrical resistivity of the concrete cover is one of the main factors in the total resistance of the electrochemical circuit, particularly for corrosion in and above the splash zone (see Fig. 11b). Both water and chloride reduce the electrical resistivity and thereby increase the total corrosion current which can flow. In the splash zone or above, the only path for the hydroxyl ions from the cathode to the anode is via the concrete cover. If this is sufficiently dry and non-conducting, corrosion will be negligible. Surveys of existing structures by Bury and Domone [86] have indicated that areas of corrosion and spalling are generally associated with low concrete resistivity (about 3000 to 5000 $\Omega\,cm^{-1}$, cf. 13 000 to 15 000 $\Omega\,cm^{-1}$ in non-spalled areas). Other field observations [87] indicate a higher resistivity (50 to 70 000 Ω cm^{-1}) when steel corrosion would be negligible.

Monitoring of electrical resistivity is therefore one possibility for in-service inspection of offshore concrete structures and has been employed by some organizations to indicate areas susceptible to corrosion. It is, however, limited to use underwater because of the low resistance of seawater.

6.2.4. Conditions at the concrete–steel interface

It is generally believed that the alkaline environment produced by the pore solution in the concrete is the essential factor in passivating the surface of the steel reinforcement. The nature of the passive layer is not well understood nor is the range of alkalinity acceptable to afford corrosion protection [88]. The degree of alkalinity is a function of the age of the concrete both from the effect of the ingress of deleterious ions and from internal chemical changes in the concrete itself. Ordinary Portland cement has a reserve of alkalinity [89] which is available to buffer the effects of, for example, incoming chloride ions. In concretes made with additions of pulverized fuel ash (as used in several North Sea structures), the reserve of alkalinity is much lower [65] and research is needed to confirm that this has no major effect on corrosion performance. Some results [90] indicate that PFA concretes are more durable in marine conditions.

An alternative explanation to breakdown of general passivity by ingress of chloride ions has been put forward by Page [91] who maintains that the presence of a thin lime layer at the steel surface is the controlling factor in limiting corrosion. In poorly-compacted areas where the concrete did not initially contact the surface of the steel reinforcement, this lime layer is absent and these voids are prime sites for corrosion to initiate. Corrosion occurring at voids has been identified [77] but a clearer relationship is required to give a better understanding of the need to minimize voids at the steel interface during construction.

Surface coating of the reinforcement has been used in some concrete structures to give improved durability. Galvanized reinforcement has had limited application offshore to date although elsewhere its advantages of corrosion resistance have been exploited [92] particularly for severe exposure conditions. A few isolated incidents have been reported, however, of rapid corrosion of galvanized steel embedded in concrete when chlorides are present [93]. The presence of zinc in contact with calcium hydroxide (as a hydrolysis product of the cement) can lead to the formation of calcium zincate and the liberation of hydrogen. Hydrogen can have deleterious effects on the steel and to minimize this effect, common practice is to ensure that all galvanised rebar is chromated prior to use; the chromates absorb the hydrogen. The general conditions under which hydrogen evolution can occur are not well understood at present and further research in this area is needed.

6.3. Corrosion of steel attachments to a concrete structure

Steel attachments (e.g. supports for pipework) which are in electrical contact with the reinforcement are vulnerable to corrosion. In this case the driving force for a galvanic cell in seawater can be high, up to half a volt. This is the difference between the potential of bare steel in seawater (−0.65 volts with respect to silver–silver chloride reference electrode) and the potential of concrete coated steel (about −0.25 volts in the passive condition). The galvanic circuit for this situation is shown in Fig. 11d. The consequences of corrosion is loss of material at the anode rather than spalling of the concrete cover. This corrosion problem has received insufficient attention in the past and several instances have been recorded [94] where rapid corrosion of steel in contact with reinforcement has occurred. The rate of corrosion is generally believed to be controlled by the ratio in area between anode and cathode; some tests [95] indicated very high corrosion rates for small anodes in combination with large cathodes. A second important factor is the presence of a low resistivity electrolyte between anode and cathode. In many reported examples of corrosion to steel components attached to reinforced concrete [94] the electrolyte has been very moist soil; in the offshore situation the concrete cover when saturated with seawater is a very efficient electrolyte for corrosion.

From a design viewpoint it is not clear at present whether attached steelwork should be electrically isolated from the reinforcement network or electrically bonded and cathodic protection applied to the whole structure. In the former case it is difficult to avoid accidental contact particularly taking into account the scale of these structures. In the latter case there is some uncertainty at present on the design requirements for a cathodic protection system to reduce corrosion of steel attachments. Some work [96] has indicated enhanced corrosion of sheet steel electrodes attached to test pieces in a potential range normally specified for corrosion prevention. Cathodic protection of attached steelwork can also lead to rapid consumption of sacrificial anodes, because of the "drain" to the extremely large cathodic area represented by the reinforcing network encased in concrete. This general area is one requiring further research.

The offshore industry is however generally aware of this problem and care is taken in fixing attachments to concrete structures. It is a problem that needs particular care at the design stage of hybrid concrete–steel structures.

6.4. Corrosion of pre-stressing tendons

The tower sections of many offshore structures are post-tensioned to reduce the effects of cracking, arising from wave loading. The tendons are high carbon steel with characteristic strengths of $\sim 1500\,\mathrm{N\,mm^{-2}}$ (Section 3) contained in metal ducts at least 100 mm below the surface of the concrete (in and above the splash zone). The ducts are filled with cement grout to minimize access of agressive ions to the tendons.

Pre-stressing tendons are possibly subject to stress corrosion cracking [97] because of their high yield strength and in practice they carry high tensile loads (up to 70% of yield). It is generally accepted however that provided the ducts are filled with grout and adequate concrete cover is maintained the risk of stress corrosion cracking is small [78].

6.5. Summary

Over many years, experience has shown that well-designed structures built with good quality concrete (low water–cement ratio and high cement content) well compacted during construction and with adequate cover to the steel reinforcement have a long life in a marine environment. In a few cases, problems have occurred and it is the purpose of continuing research to identify those factors which lead to corrosion of the steel reinforcement.

7. Fatigue behaviour of reinforced concrete

Fatigue in reinforced concrete is a much less serious problem than corrosion fatigue in welded structural steels because of the absence of welded connections in high stress areas and because of the massive nature of these structures. In fact, fatigue in land-based reinforced concrete structures has rarely been a problem in practice, although it is given some consideration in design of concrete bridges. Although the fatigue behaviour of reinforced concrete in air is well documented and has been recently reviewed [98, 99], there are very limited data on the fatigue performance in seawater. Several test programmes are currently in progress to produce the additional data needed to confirm the fatigue performance of concrete offshore structures. The background and rationale to the "Concrete in the Oceans" fatigue test programme has also recently been summarized [100].

Fatigue of reinforced concrete is usually considered in terms of the fatigue behaviour of its two main constituents, steel and concrete. For normal levels of reinforcement, it is generally accepted that the fatigue properties of the steel reinforcement are controlling in design.

7.1. Fatigue in concrete

The fatigue properties of plain concrete in air under constant stress level tests are well documented and a recent review [98] has highlighted the current position. In compression fatigue, failure is associated with crushing and spalling of the concrete as in static load tests, caused by initial microcracking and progressive growth of these cracks to failure. Normal cyclic loading tests are limited for practical reasons to a limit of about 10^8 cylces and up to this level there is no evidence of a fatigue limit (as in steels in air) in testing plain concrete. The stress level required to cause failure under high cycle conditions is normally taken as $0.55\,F_{\mathrm{cu}}$, where F_{cu} is the cube strength obtained under static load conditions. This reduction in strength causes no serious problems in design.

A limited number of tests have been reported [101] on plain concrete immersed in seawater, although most of those have been at high test frequencies (6 Hz). Preliminary results indicate no major reduction in properties through the seawater environment.

Results of a recent pilot test study [102] of reinforced concrete exposed to cyclic loading and hydrostatic water pressure indicate that the fatigue strength of the concrete is reduced under some conditions. Failure of the concrete in compression occurred first in all the tests carried out; in practice, this could create a corrosion hazard by exposing the reinforcement directly to the seawater. The effect of water washing in and out of the cracks during the repeated loading is believed to have made a significant contribution to the reduction in properties. However, these tests were completed on relatively low-grade concrete by offshore standards (most tests were on about 30 $N mm^{-2}$ concrete) and the frequency of testing was much higher than wave loading (6 Hz, cf. 0.1 Hz for wave loading). Both factors probably accentuate the effect. Further tests are now in progress to establish whether the combined effects of hydrostatic pressure and cyclic loading are general.

7.2. Fatigue in reinforcing steel

Although fatigue performance is not part of current standards for reinforcing steels (see Section 3) the fatigue properties in air of most types of reinforcing steel in general use are well documented [98, 99]. A fatigue limit is observed which corresponds to about 50% of the static strength. Fatigue life in air is also practically independent of cycle time, which is very useful for accelerated testing of components. The type of rib pattern on the surface has been found to be important in fatigue [99]. There is a marked bar size effect with larger bar sizes having poorer fatigue properties [103]. The diameter effect appears to be less significant for bars embedded in concrete, perhaps because the sites for fatigue crack initiation are more limited, being associated with cracks in the concrete cover. Welding of reinforcement seriously reduces fatigue life and is therefore limited in practice to low stress areas in the structure. The reduction in fatigue life at a given stress level is by at least an order of magnitude.

The fatigue properties of reinforced concrete in seawater are less well documented [78, 104, 105] and although several research programmes are underway, the data available on fatigue performance in seawater are limited at present. The current situation as summarized in Fig. 14 shows a significant reduction in fatigue life for bare reinforcing bars tested in seawater. This reduction is about one order of magnitude in fatigue life.

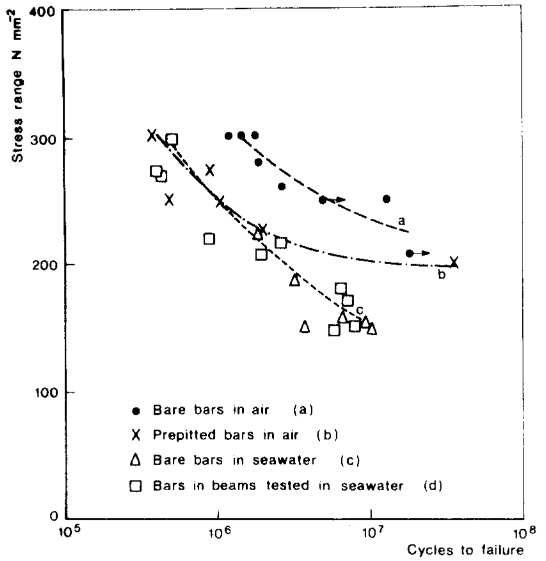

Figure 14 Fatigue *S–N* curves for reinforcing steel, showing results for (a) bare bars in air; (b) pre-pitted bars in seawater; (c) bare bars in seawater; and (d) bars embedded in concrete beams tested in seawater (after Taylor and Sharp [100]).

The effect of embedding the steel in concrete is not clear at present. Tests with a small cover thickness (25 mm) [105] show no improvement over bare steel (Fig. 14) although the data for bare steel was obtained using a high cyclic frequency (145 Hz) where corrosion effects are not fully simulated. A limited number of tests with more realistic cover thicknesses (65 mm) [106] have shown some improvement over bare steel performance.

To date, very few results have been reported from tests undertaken at wave frequency (~ 0.1 Hz) and corrosion fatigue data on steels has shown that cycle frequency is very important in seawater tests. Most tests to date have been completed at 3 Hz or higher where corrosion effects are limited. Current research programmes related to offshore structures are concentrating on tests undertaken at wave frequencies.

Very limited information is currently available on the effects of random loading on fatigue properties; most tests to date have been completed at constant stress level. In practice, of course, most structural members are subjected to randomly varying loads which are allowed for in design using the Palmgren–Miner rule based on linear summation of damage (see Section 5). Some tests on plain concrete in air using two stress levels only [107] have shown that application of the higher load first is more damaging. In the same investigation rest periods were found to be beneficial. This is clearly an area where further research is required.

An understanding of fatigue failures in reinforcing steel either in air or seawater, based on initiation and propagation of cracks, is lacking at present. In fatigue tests, most failures in the reinforcing bar occur close to cracks in the concrete cover [108], which could be a result of slight stress concentrations at these sites. In the case of the seawater environment, it is difficult to accept that crack initiation can be a dominant process, bearing in mind the rough nature of the reinforcing steel surface, the presence of a rib pattern and the possibility of local corrosion pits. This is substantiated by the critical crack size ($\sim 50\,\mu m$) for steels with strengths in the region of 400 $N\,mm^{-2}$ [41]. It has also been argued [100] that since the period for initiation of corrosion in cracked concrete (see Section 6) is short in terms of design life, initiation of fatigue cracks from local corrosion sites or elsewhere should also take place in a short timescale. On this basis, propagation of the fatigue crack is the dominant process. Fracture mechanics has had a limited application so far in composite materials and its use in understanding the behaviour of reinforced concrete is not well developed at present. Fracture mechanics data obtained from bare reinforcing bars (Unisteel 400) tested in air and 3.5% sodium chloride solution have been published [103]. These show that at higher values of the stress intensity, the rate of crack growth in the corrosive medium approximately doubled and at even higher values a seven-fold increase in crack growth rate in the NaCl solution was measured. These results have limited application for two reasons. Firstly, the test frequency was 10 Hz where corrosive effects are not fully simulated and secondly, their relevance to steel embedded in concrete is not clear. Application of fracture mechanics techniques to this area could be beneficial and enable the effect of experimental parameters on fatigue performance to be verified, as has happened in the UK Offshore Steels Research Project (see Section 5).

8. Inspection and maintenance of platforms

Regular inspection of all offshore installations is required for renewal of the Certificate of Fitness by the Certification Authority. In the case of steel jacket structures, this involves inspection of the more heavily loaded tubular nodes of the primary structure (where the bracing members meet the main platform legs) both above and below the waterline to detect defects arising from cyclic stressing or corrosive attack [12, 13]. Removal of the marine fouling is first necessary before a diver can use a non-destructive test method to inspect the weld region. The most widely used method underwater is magnetic particle inspection in which the position of a defect or crack is indicated by a build-up of magnetic particles [109]. The test region is magnetised and then finely divided magnetic particles are applied to its surface. Ultrasonic and eddy current methods are also used underwater. In all cases, the underwater operation of sophisticated non-destructive test methods by a diver is time-consuming and costly. Improved testing methods are currently being developed to improve the efficiency of underwater inspection of steel structures and if possible, to eliminate diver involvement by using unmanned submersibles or remote monitoring techniques.

For concrete structures, the present position is to rely mainly on visual inspection techniques [13]. Corrosion of the reinforcement normally results in spalling of the concrete cover and this can be detected by a visual examination of the structure. One of the major problems in inspecting a concrete structure is the very large surface area (several acres) and the absence of any distinctive features. Techniques for diver location on the surface and for rapid scanning of large areas are obvious requirements.

Underwater repair of steel structures has received considerable attention. Several platforms have required repair [110, 111] mainly as a result of accidental damage (e.g. from a boat collision). Underwater welding is at present the main method. Wet welding is usually considered unsatisfactory for high strength repairs because of both the rapid cooling of weld metal and the production of high hydrogen contents in the weld [112]. Both of these factors lead to brittleness and loss of strength. Underwater welding is normally carried out in a dry habitat and under these conditions, good properties can be attained [113, 114]. Nevertheless, the time involved in preparing the habitat, the support services required and diver involvement make underwater repair very expensive indeed. However, the cost of a repair needs to be compared with the cost of a day's production of oil from a typical platform (~ £1m). Several alternatives to underwater welding are under development as welding may not be acceptable in some cases (e.g. where there is a risk of explosion).

The inspection and maintenance of offshore structures is likely to grow in importance as platforms approach their design life. Accidental damage to structures has also occurred and in view of the complex operations undertaken both on the platform and in its vicinity (particularly involving supply boats) further accidents are inevitable. From both the operators' and the nation's viewpoints, it is important that an adequate range of repair options is available to ensure continuity of oil production.

9. Future development of offshore structures

The present designs of steel offshore structures for the North Sea are a development of the tubular space-frame structures used for many years in places like the Gulf of Mexico. Nevertheless, the more hostile environment and the deeper water has required thicker sections and heavier structures resulting in increased difficulty of fabrication and higher cost. As oil is inevitably found in deeper waters, new and improved structures will be required for its extraction. In the Gulf of Mexico, a steel structure has recently been installed to operate in water depths of 305 m, almost twice the depth of the Continental Shelf. This platform on the Cognac field is of similar design to existing structures, consisting of three sections joined together on site. Operation at those water depths in the North Sea with its more hostile wind and wave conditions would have major effects on the design requirements with consequent increases in cost. Similarly, extension of existing concrete platforms to deeper water would result in much heavier and costlier structures, which could also create problems for the bearing capacity of the seabed. Because of these limitations, new designs for offshore structures have been produced. The design most likely to be used in the near future is a tethered buoyant structure consisting of a floating deck tethered to the seabed by several heavy cables [115]. New materials problems are likely to arise with these structures. In particular, the fatigue performance of the cables and their resistance to stress corrosion cracking are important factors to be taken account of in the design. The riser, too, is likely to need additional attention for its fatigue resistance, because of its increased length and the increased motions of the deck compared to existing structures.

Seabed systems have many attractions for very deep water operation and several designs are currently under development [116–118] for possible use in the next decade. Concrete has many advantages for structural use in deep water as it can efficiently resist hydrostatic compression forces and several current designs utilize concrete as the main structural material.

New designs of structures for extraction of oil from deeper waters in the North Sea and elsewhere will place increasing demands on materials. The behaviour of structural materials in an offshore environment is likely, therefore, to be of considerable interest well into the future.

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