
We Trip The Light
FANTASTIC

# UNDERWATER CONCRETING AND REPAIR

Edited by

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# Preface

The construction of a wide range of structures including bridge piers, harbours, sea and river defences over many decades, and more recently the development of offshore oil fields, has required placement of concrete underwater. This process can be successfully carried out and sound, good quality concrete produced if sufficient attention is paid to the concrete mix itself and the methods of construction employed.

This book is intended for the practising engineer, who whilst being experienced in the techniques and approaches for construction above water needs practical advice and guidance on underwater concreting. The contents of the book are arranged in a progressive order starting with considerations that must be given to the design of the concrete mix to minimise the effects of contact with water, and to take into account the practicalities of placing and compacting the concrete. The methods that can be employed to prepare the construction site, types of formwork available and methods of placement are then described and their relative merits and potential problems discussed. As much underwater concrete is of considerable age and is exposed to severe conditions, techniques for inspecting underwater to identify defects, and the methods of repair that can be employed are important issues that are described. Finally, the durability of concrete in an underwater environment is discussed and the potential areas of concern highlighted.

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B W Staynes

## 1.1 Introduction

Concrete mix design involves the selection and proportioning of available materials to produce concretes which in both the fresh and hardened state meet the requirements of a specified application. Generally these requirements concentrate on the properties of workability/flow, compressive strength and durability. The overall concreting operation needs to be achieved as economically as possible and for simple concrete construction this often requires the mix design to minimize material costs, i.e. the cost of the ingredients. However, for some specialized applications higher concrete material costs are more than compensated for by the savings achieved at the transportation/casting stage, or the speed with which the structure can start to earn revenue.

In the case of underwater concreting operations, mix design plays a significant part in the overall efficiency of construction in terms of technological quality and overall economics. Almost without exception trial mixes will be required.

The properties needed for underwater concrete are directly related to the method of placement, and this technology is covered in Chapter 4. The principal methods include:

- tremie (including the 'hydrovalve')
- pumping with free fall
- skip (bottom opening)
- prepacked (preplaced) aggregate concrete
- prepackaged—above water
  —under water.

In addition, the geometry of the finished top surface (horizontal or laid to falls) needs to be taken into account as most concrete placed underwater has a tendency to flow to a level surface.

Parameters relevant to each type of placing condition are indicated in Table 1.1.

**Table 1.1** Relevant parameters

| Parameter | Placing method: Tremie | Pumping with free fall | Skip | Prepacked (preplaced) aggregate concrete | Prepackaged: Above water | Prepackaged: Under water |
|---|---|---|---|---|---|---|
| Strength | √ | √ | √ | √ | √ | √ |
| Durability | √ | √ | √ | √ | √ | √ |
| Segregation/washout Resistance during: | | | | | | |
| internal flow | √ | √ | √ | | | |
| free fall quiescent | | √ | √ | | | |
| free fall turbulent | | √ | | | | |

The parameters involved in normal concrete mix design and their interaction are given in Figure 1.1 with the additional underwater concrete factor 'washout' and its interactions shown in bold. The placing conditions for a particular application have a significant influence on the degree of washout resistance required. Thus the mix design process needs to take account of this, particularly with regard to aggregate selection, cement content and the use of admixtures.

Unless practical test data relating to the specific combination of aggregates, cements, admixtures and any other constituents are available, the use of trial mix procedures will form an essential part of the mix design process. These are likely to take the form of initial laboratory trials (which may include washout resistance testing) followed by full-scale trial mixes. In the latter case, where new or unusual placing conditions are to be encountered, effective performance in sample pours should also be assessed.

## 1.2 Characteristic/target strength relationships

Variation in the compressive strength of concrete specimens are usually assumed to conform to a 'normal' distribution as illustrated in Figure 1.2. For general concreting operations variability of quality control test results is caused by variations in the materials used, production operations and sampling/testing techniques.

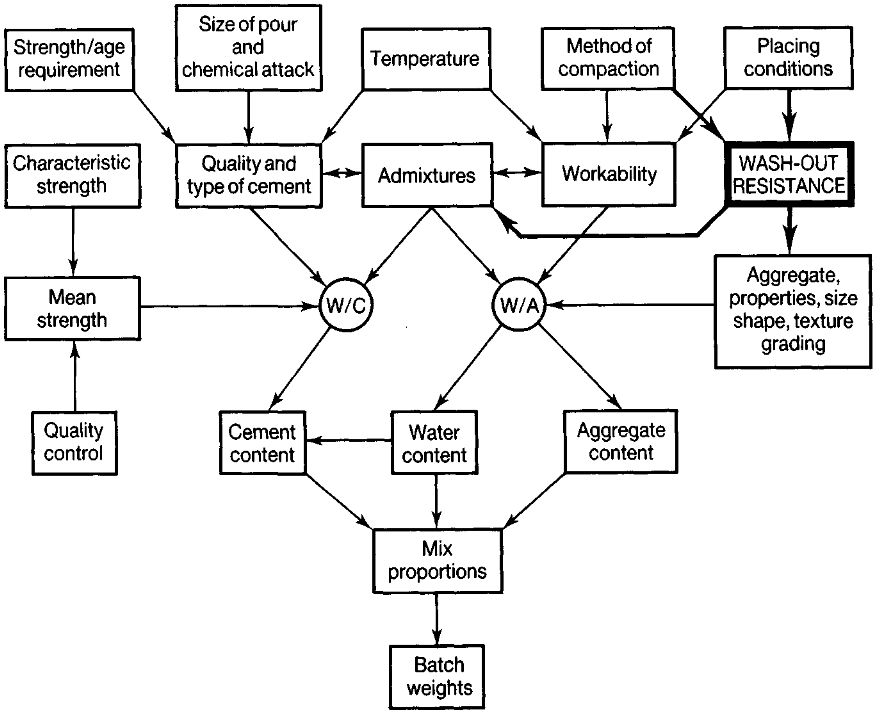



**Fig. 1.1** Concrete mix design. Parameters and interactions

The form of a normal distribution curve can be defined entirely by its mean ($m$) and its standard deviation ($S$), where

$$S = \sqrt{\frac{\Sigma(x-m)^2}{n-1}}$$

and $n$ is the number of test results.

The area under the normal distribution curve shown in Figure 1.2 represents all the available test results. The characteristic strength (spe-

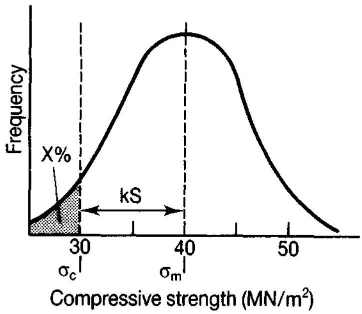



**Fig. 1.2** Normal distribution of concrete strengths

cified strength) is usually identified by the design engineer and is included in the specification (e.g. 30 MN/m$^2$ at age 28 days under standard curing conditions).

As it is statistically impractical to establish a distribution curve for which zero results are defective, i.e. less than the characteristic/specified strength ($\sigma$c), it is common practice to determine the mean/target strength (required average strength) ($\sigma$m) for concrete mix design purposes on the basis of an allowed percentage of defective test results ($X$), i.e.

$$\sigma\mathrm{m} = \sigma\mathrm{c} + kS$$

where

| $X$(%) | $k$ |
|---|---|
| 5 | 1.64 |
| 2.5 | 1.96 |
| 1 | 2.33 |

In practice, $S$ is based on experience or is assumed to be 4–8 MN/m$^2$ (Ref. 1). Typically $X = 5\%$.

Ideally $S$ should be calculated from results taken from the production operation used on the project in question. If these data are not available, values can be assumed such as the 4–8 MN/m$^2$ values recommended by the DOE[1] or by ACI.[2] Thus the mean or target strength for a mix with a characteristic strength of 30 MN/m$^2$, a standard deviation of 6 MN/m$^2$ and allowing for 5% defectives is

$$\begin{aligned}\sigma\mathrm{m} &= \sigma\mathrm{c} + kS \\ &= 30 + 1.64 \times 6 \\ &= 39.8\ \mathrm{MN/m^2},\ \text{i.e. } 40\ \mathrm{MN/m^2}\end{aligned}$$

Depending on how critical failure of particular components may be, concrete specifications often include safeguards additional to the limitation on the percentage defective test results which fall below the specified characteristic strength. Examples of additional safeguards include:

- the average strength of any three consecutive test specimens must exceed the characteristic strength by a given amount, say 7.5 MN/m$^2$
- no individual test result may fall below a specified proportion of the characteristic strength, e.g. 85%.

While the above are details associated with specifications, they can have a significant influence on the approach to the selection of the mean/target strength used for concrete mix designs.

The quality of concrete in the finished structure may additionally be affected by variations due to transportation, placing, compaction and curing operations. As these operations can be witnessed in most 'dry' placing condition applications, good supervision can ensure that the quality of concrete in structural components has a known relationship to the characteristic strength based on quality control specimens.

Detailed observation of transportation, placing, compaction and curing is much more difficult to achieve for concrete placed underwater. Thus, while underwater concrete test specimens cast in the dry can be expected to follow a typical normal distribution, much greater variability can be expected in an underwater structure. Allowance can be made for such variations by increasing the standard deviation and thus the margin between characteristic strength and target strength. The extent of the increase is difficult to estimate and needs to take account of detail placing techniques, the resistance of the specific concrete to washout/segregation and flow/self-compaction qualities in relation to placing conditions. It follows that it is better to increase the partial safety factor for materials at the structural design stage. This enables engineering judgment to be exercised in determining the overall safety factor which will also include allowance for the uncertainties in applied loading. These could be considerable in some underwater concrete applications.

## 1.3 Strength/age requirements

Specific location conditions dictate the characteristic strength requirements for each application condition. Thus specified grades of concrete vary from 25 MN/m$^2$ for cofferdam plugs to 65 MN/m$^2$ in the splash zone of oil production platforms. In the above examples the rate of gain of strength is relatively unimportant as compressive strength is unlikely to be a critical performance parameter for cofferdam plugs and, in the case of oil rigs, a considerable time will elapse between casting and the concrete being subjected to service conditions. Thus the characteristic strengths are likely to be defined at an age of 28 days for simplicity and clarity of specification.

At one extreme, for concrete placed *in situ* in the tidal range, perhaps with limited protection, early age strength will be a critical factor. Under such conditions significant strength may need to be developed within a few hours. Such difficulties may dictate the use of precast sections and/or the use of packaging techniques.

On the other hand, owing, for example, to tidal conditions, concrete cast underwater has to be placed in lifts. To ensure a good bond/homogeneity between successive placements, slow early age strength development can be particularly advantageous. Such requirements need to be built into the specification and taken into consideration in the mix design.

## 1.4 Materials

### 1.4.1 Aggregates

As it is usually impossible to achieve detailed visual inspection during the placing of underwater concrete, and it is usually necessary for the concrete to flow and self-compact, it is important to select aggregates and gradings which are particularly resistant to segregation and bleeding and which have high cohesion.

#### 1.4.1.1 Coarse aggregates

It is well known that rounded aggregates achieve more dense packing and have reduced water demand for a given degree of workability than do crushed rock aggregates. Thus the use of rounded aggregates generally tends to increase cohesion for a given sand friction and cement content and to have a reduced tendency to segregation and bleeding.

However, strength and abrasion resistance are particularly significant parameters in some underwater applications and it may thus be necessary for these reasons to select crushed rock aggregates. When this is the case particular care must be paid to the overall grading of the aggregate.

#### 1.4.1.2 Fine aggregates (sand) (less than 5 mm)

The only special requirement for the sand fraction over and above those needed for normal concreting mixes is that there should be a significant proportion with a particle size less than 300 μm. At least 15–20% of the sand fraction should pass a 300 μm sieve as this is necessary to enhance the cohesive properties of concrete to be placed under water. When suitable sands are unavailable it is necessary to increase significantly the cement content of mixes, or add pulverized fuel ash or ground granulated blast furnace slag.

#### 1.4.1.3 Grading

As underwater concrete needs good flow and self-compacting properties, and sufficient cohesion to resist segregation and bleeding, the aggregate grading requirements are very similar to those needed for concrete pump mixes.[3] Pump mix requirements include the above properties plus the need for the cement paste and/or mortar phase to form a lubricating film on the pipe walls. While this latter requirement is not essential for underwater concrete mixes, it is common practice to have relatively high cement contents to improve cohesion, compensate for segregation effects and allow for the inevitable losses of cement due to 'washout'.

Continuous grading curves have been found to give the best results. Generally 20 mm maximum size aggregate is most satisfactory with a sand content of at least 40% of the total aggregate. The well known Road Note 4[4] grading curves shown in Figure 1.3 provide a useful guide. Grading curve number 3 is a suitable initial target for trial mixes. However, this needs to be adjusted so that the percentage passing the 300 μm sieve is increased from 5% to about 8%. At no stage should the grading be coarser than grading curve number 2.

To achieve cohesive mixes, the relative proportions of coarse aggregate

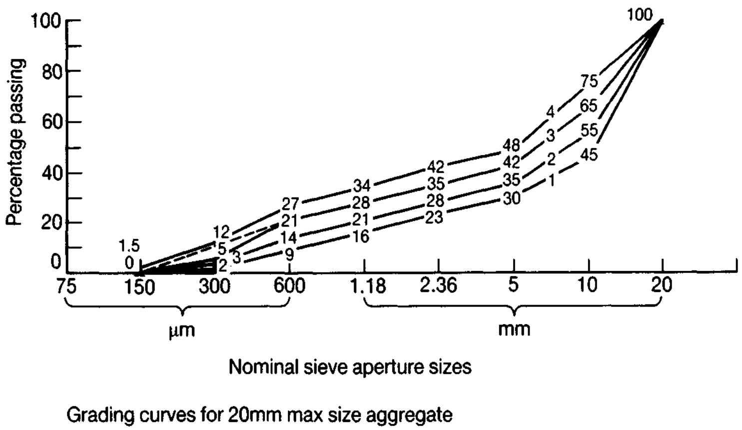



**Fig. 1.3** Grading curves for aggregates

and sand need to be adjusted to minimize the total voids in the mix. This will depend on the shape of the various particles. If necessary a 'void meter' can be used to optimize the proportions. This approach is recommended if crushed rock aggregates are used.

## 1.4.2 Cements

Sulphates in ground water and particularly in sea water present the well known problem of tricalcium aluminate ($C_3A$) reaction, causing swelling and the related disintegration of concrete. As underwater concretes usually have comparatively large cement contents (over 325 kg/m$^3$), attack due to sulphates in ground water can be counteracted in the usual way by adjusting the cement content and/or the use of sulphate-resisting Portland cement.

The presence of chlorides in sea water can reduce the above effect of expansion and deterioration of concrete. The gypsum and calcium sulphoaluminate resulting from sulphate attack are more soluble in chloride solutions and are leached out of concrete permanently immersed in sea water. However, concrete in the splash zone and above is particularly vulnerable as not only does sulphate attack occur, but also pressure is exerted by salt crystals formed in the pores of the concrete at locations where evaporation can take place. Chlorides migrate above normally wetted areas owing to capillary action, and the production of concrete with low permeability reduces this effect.

Fundamental to the durability of concrete subjected to attack due to sulphates in ground water and sea water is minimizing the porosity of the concrete at both the engineering level by achieving full compaction, and at the micro level by minimizing the gel pores. The latter can be considerably reduced by using low water/cement ratios. ACI committee 201.2R recommends that water/cement ratios should not exceed 0.45 in conditions of severe and very severe exposure to sulphates i.e. $SO_3$ content of water exceeding 1250 ppm and 8300 ppm respectively.[5] However, this needs to be accompanied by the use of high cement contents, plasticizers or superplasticizers if a high level of self-compaction is to be achieved. The use of cement replacement materials such as pulverized fuel ash and/or the addition of condensed micro silica (silica fume) can considerably reduce the porosity of concrete and thus its susceptibility to sulphate attack and chloride crystallization.

#### 1.4.2.1 Ordinary Portland cement (OPC)

OPC or ASTM Type I having not more than 10% $C_3A$ is suitable for underwater concrete construction where the sulphate content (expressed as concentration of $SO_3$) of ground water does not exceed 1200 parts per million (ppm), and for marine structures which are permanently submerged.

#### 1.4.2.2 Sulphate-resisting Portland cement (SRPC)

SRPC (ASTM Type V or Type II with a 5% limit on $C_3A$)) with its reduced tricalcium aluminate content should be used where the $SO_3$ content of ground water exceeds 1200 ppm. Its use in marine structures in the splash zone and above is less straightforward. While a low $C_3A$ content provides protection against sulphates, it reduces protection to steel reinforcement in chloride rich environments.[6] The $C_3A$ content should not be less than 4% to reduce the risk of reinforcement corrosion due to chlorides.[7]

#### 1.4.2.3 Low-heat Portland cement (LHPC)

Large pours of concrete cast underwater are particularly susceptible to thermal cracking as relatively high cement content concretes are used. LHPC (ASTM Type II or Type IV) not only reduces the rate of heat evolution but also provides protection against sulphate attack owing to the low levels of tricalcium aluminate in this cement. The use of cement replacement materials is an alternative method of reducing thermal effects and provides additional benefits.

### 1.4.3 Anti-washout admixtures

Anti-washout admixtures can be used to reduce the risk of segregation and washout with the tremie methods of placement, improve self-compaction/flow properties and enable methods of placement which are faster and less sensitive to operational difficulties to be used. In particular, combinations of admixtures have been developed to produce a 'non-dispersible concrete'

(NDC) which can free fall through a depth of about 1 m of water without significant washout of the cement phase.

#### 1.4.3.1 Cohesion improvement

Materials that have been tried with varying degrees of success to produce non-dispersible concrete include:[8,9]

- natural polymers (gum arabic, methylcellulose, hydroxyethylcellulose, carboxymethylcellulose)
- synthetic polymers (polyacrylonitrile, polyacrylamides, polymethacrylic acid, polyacrylates, copolymer of vinyl acetate, maleic acid anhydride)
- inorganic powders (silica gel, bentonite, micro silica)
- surface-active agents (air entraining with and without set retarder, plasticizers).

It is essential that the selected materials are compatible with cement hydrates. Several of the above cause severe retardation of the hydration process and limit the use of superplasticizers. The ionic polymers are insoluble in water containing hydration products owing to the presence of calcium ions and thus fail to increase its viscosity.

Table 1.2 gives details of the properties/influences of some of the more commonly used admixtures to improve cohesion in underwater concrete.

**Table 1.2** Properties and influences of admixtures

| Admixture | Property/influence |
|---|---|
| *Micro silica*<br>0.1–0.2 μm microspheres typically over 90% reactive silica | Compatible with cement<br>Increase compressive and tensile strength<br>Increased rate of gain of strength<br>Reduce porosity<br>Increase durability<br>Increases resistance to abrasion–erosion effects<br>Increase cohesion |
| *Non-ionic cellulose ether*<br>Derivative, up to 500 cellulose ether units; formula, see Figure 1.4; *n* up to 500; equivalent molecular length 0.5 μm | Compatible with cement<br>Retards hydration reaction<br>Large increase in viscosity<br>Large increase in cohesion<br>Very good segregation resistance<br>Self-levelling/-compacting |
| *Non-ionic polyacrylamide*<br>Typical molecular mass $5 \times 10^6$, approximately 70000 units; formula, see Figure 1.5; equivalent molecular length 10 μm | Compatible with cement<br>Retards hydration reaction<br>Large increase in viscosity<br>Large increase in cohesion<br>Excellent segregation resistance<br>Flow resistance (20% surface gradient) |

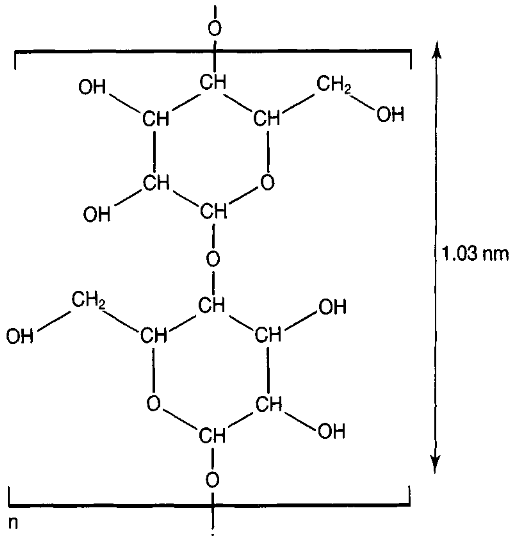



**Fig. 1.4** Cellulose ether unit

### 1.4.3.2 Flow improvement

High slump concretes generally flow underwater and the addition of superplasticizers to enhance this property alone is not usually required. However, proprietary underwater concrete admixtures are a blend of several compounds and usually contain a superplasticizer to improve the flowing properties of what would otherwise be a very sticky concrete. The superplasticizers most commonly used in the construction industry are based on melamine formaldehyde and naphthalene formaldehyde. While the former are compatible with the soluble polymers used to increase cohesion, naphthalene formaldehyde-based superplasticizers have been found to be ineffective when used with cellulose ether.

### 1.4.3.3 Cement replacement with PFA or GGFS

Partial cement replacement with pozzolanic materials such as pulverized fuel ash (PFA) or ground granulated blast furnace slag (GGFS) not only

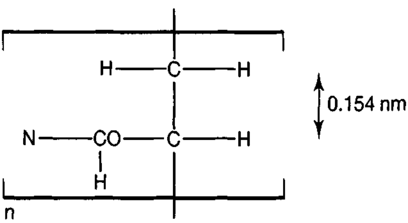



**Fig. 1.5** Polyacrylamide unit

reduces the risk of thermal cracking in large pours, but also improves the performance of micro silica and cellulose ether at producing concretes capable of resisting washout of the cement and fine phases of concrete. Typical commercial underwater concrete admixtures reduce washout from 20–25% down to about 10%. However, when Portland cement is used with 30% PFA or 50% GGFS replacement, washout is reduced still further for the same admixture dose. The most cost-effective method of producing NDC is likely to involve the use of partial cement replacement with PFA or GGFS in conjunction with NDC admixtures.

## 1.5 Properties required of underwater concrete

### 1.5.1 General

The properties required for concrete placed under water by tremie, pump with free fall and skips are:

- specified strength and durability
- self-compaction (i.e. displace accidentally entrained air, and flow to fill formwork)
- self-levelling or flow resistance (depending on placing conditions)
- cohesive (i.e. segregation resistance)
- washout resistance (the degree depending on the method of placement).

The extent of the interelationship between the above properties depends on the mix design approach used to achieve them. As discussed in Section 1.2, the specified strength is normally based on test samples cast in dry conditions. Its relationship to the characteristic strength used at the design stage is chosen to take into consideration reductions to be expected when concrete is placed under water.

### 1.5.2 Concrete without admixtures

Well executed tremie/hydrovalve techniques have been found to produce underwater cast concrete with up to 90% of the strength of the same concrete cast in dry conditions. However, if proper control of the base of the tremie pipe is not achieved and/or the concrete is required to flow over significant distances owing to lack of mobility of the placing locations, strengths as low as 20% of the equivalent concrete cast in air can occur.

This loss of strength can be attributed to segregation/stratification and/or washout of the cement phase of the concrete.[10] It should be noted that if the whole of a vertically drilled core is analysed for cement content there

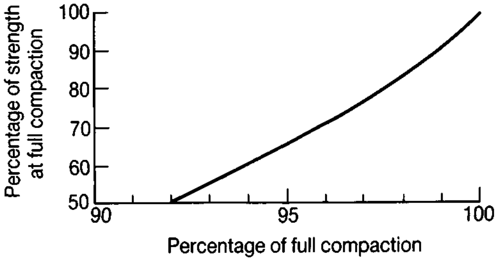



**Fig. 1.6** The influence of compaction on the strength of concrete

may be little apparent loss of cement. More careful examination may reveal that a considerable proportion of the cement is in the upper layers of the concrete, possibly appearing as a thick laitance on the top surface. Parts of the concrete are likely to have lost over 25% of their original cement content.

The significance of a lack of full compaction on concrete strength is well known (Figure 1.6). As it is impractical to compact concrete placed underwater by physical means using vibrators or by tamping, it is essential that the concrete should have sufficient workability to displace any accidentally entrained air during the settlement/flow period after the concrete has been placed.

Established practice is to specify slump values of 120–200 mm. These values offer a useful guide for trial mixes but, as concretes with a given slump can have varying flow properties, the ability to self-compact needs to be assessed by practical trials.

In order to reduce porosity and achieve strength requirements at high water contents and compensate for segregation/losses, it is necessary to have relatively high cement contents. Traditional mix designs have cement contents of 325–450 kg/m$^3$. Experience has shown that concrete with relatively low cement content has better abrasion resistance. Where this performance criterion is important and/or where large pours can give rise to thermal cracking problems, it is preferable to use the lower end of the above range. However, the cohesion needed to avoid segregation and washout requires a minimum fines content resulting in the need for cement contents as high as 400 kg/m$^3$. These conflicting performance requirements have led to the use of admixtures and cement replacement materials.

### 1.5.3 Non-dispersible concrete

The inherently slow nature of tremie placement coupled with its operational difficulties, quality uncertainties and wastage have led to the development of non-dispersible concretes. Non-dispersible concretes can be produced with varying degrees of cohesion and washout resistance. On the one hand it is possible to design a mix which reduces the quality uncertainties of tremie placed concrete resulting from uncontrolled internal flow velocities and changes in the geometry of the concrete/water interface. The relatively modest increases in cohesive properties required can be achieved by the addition of 10% micro silica (by weight of cement) to a traditional mix containing about 325 kg of cement per cubic metre of concrete.[11] Depending on strength and flow requirements, a superplasticizer can also be included.

Fully non-dispersible concretes, on the other hand, can be discharged from a pump delivery pipe through 1 m or so of water without significant loss of cement. The highly cohesive properties required are achieved by the addition of 2–3% of cellulose ether or polyacrylamide. They are often blended with a melamine formaldehyde superplasticizer, and in some cases micro silica, to produce the commercially available underwater concrete admixtures. As extensive testing is necessary to ensure the compatibility of the combined ingredients, it is advisable to use commercial products rather than combine the basic materials on-site. Nevertheless, it is essential to prepare trial mixes from the combination of aggregates, cement and admixtures used on a specific project to ensure that the required performance is achieved. Some proprietary non-dispersible admixtures and non-dispersible prebagged concretes are listed in Tables 5.1 and 5.2, respectively.

Figure 1.7 enables a comparison to be made between the strengths of a control mix and a non-dispersible concrete cast in air and in water. Figure 1.8 illustrates the loss of cement and fines during free fall through water at various doses of admixture.

The increase in speed of placement, reliability of concrete quality and savings in preparation and concrete wastage justify the use of non-dispersible concretes despite their substantially higher unit material cost.

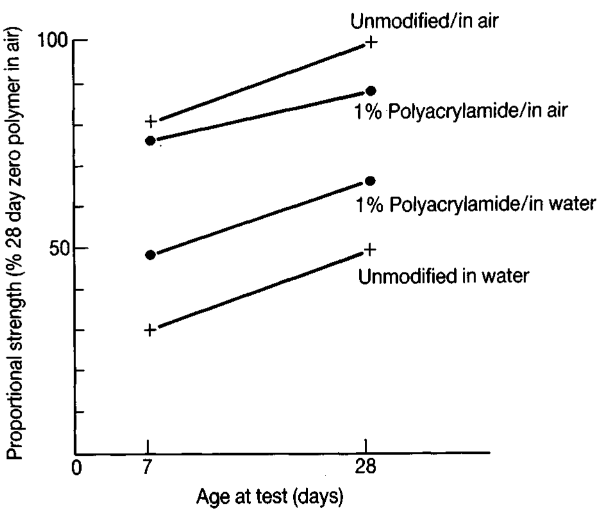



**Fig. 1.7** Comparison between control mix and 1% polyacrylamide-modified concrete

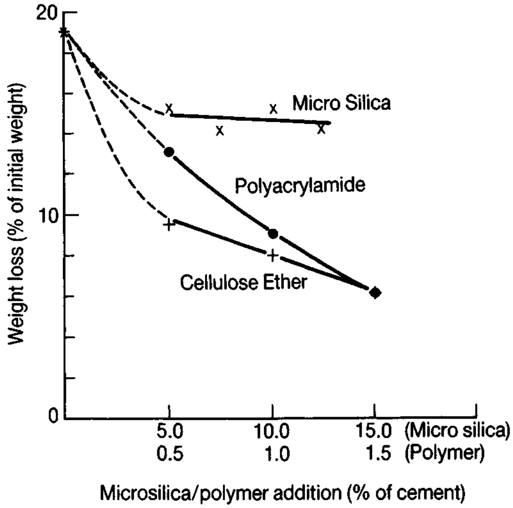



**Fig. 1.8** Weight loss under free fall

# 1.6 Test methods

## 1.6.1 General

It is important to be able to evaluate the effects of non-dispersible concrete admixtures not only in terms of obvious short-term parameters but also their influence in the longer term and over the full life of the structure. Tests are required to evaluate segregation resistance, workability/flow, chemical compatibility, influence of admixtures on strength and effectiveness at full-scale.

## 1.6.2 Washout tests

Resistance to washout of the cement phase is fundamental to the production of a concrete which can free fall through 1 m or so of water without serious degradation.

### 1.6.2.1 Transmittance test

In this case a measured slug of concrete (typically 0.5 kg) is dropped into a vessel containing about 5 l of water. The turbidity of the water is measured using standard light transmittance apparatus. By calibration using standard known dispersions of cement in water, the amount of washout occurring as a result of the concrete falling through the water can be determined (Figure 1.9).[12]

A variant of this test is to agitate the water with a laboratory stirrer for a

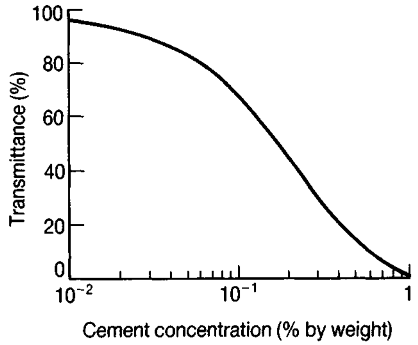



**Fig. 1.9** Relationship between cement concentration and transmittance. Ordinary Portland cement was dispersed in water

prescribed period. This is a more stringent test but produces similar comparative results.

#### 1.6.2.2 Stream test

This is a straightforward test in which a sample of concrete is placed in a 2 m long channel set at an angle of 20°. A measured volume of water is poured down the channel and depending on the segregation resistance of the concrete, cement is washed out.[13] The degree of washout can be judged on a comparative basis by visual observation and on this basis is subjective. However, by standardizing the volume and speed of water flow, and collecting it at the downstream end of the channel, the transmittance of the effluent can be measured as above, thus enabling comparative performance to be judged on a numerical basis.

#### 1.6.2.3 Plunge test

In this case a sample of concrete is placed in an expanded metal or wire-mesh basket and allowed to fall though 1.5 m of water in a vertically mounted tube. The sample is hauled to the surface slowly (0.5 m/s), weighed and then the process is repeated. A total of five drops has been accepted as standard.[11] A typical relationship between the number of drops and percentage weight loss is shown in Figure 1.10. While the rate of fall of the basket and concrete is relatively faster than the free-fall speed of concrete alone, the protective effect of the mesh of the basket mitigates against this. The results of the test are repeatable, enabling good comparisons between different concretes to be made. It is generally thought to relate well to practical conditions of free fall from a pump delivery hose through 1–2 m of water. A similar test method (CRD-C61-89A) has been used by the US Army Corps of Engineers.[14]

A variation of this test has been used to assess the relative performance

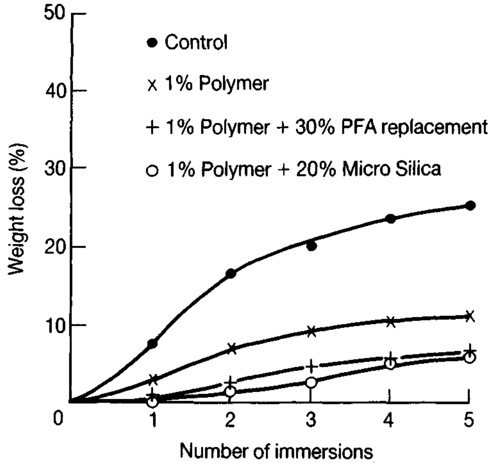



**Fig. 1.10** Plunge test result

of admixtures at a range of velocities of the sample of concrete. The results are shown in Figure 1.11.

### 1.6.2.4 Segregation test

A segregation susceptibility test, originally introduced by Hughes,[15] and subsequently revised by Khayat,[16] may be used to evaluate the separation of coarse aggregate from fresh concrete when placed under water. The test describes the scattering of concrete after having been dropped over a cone

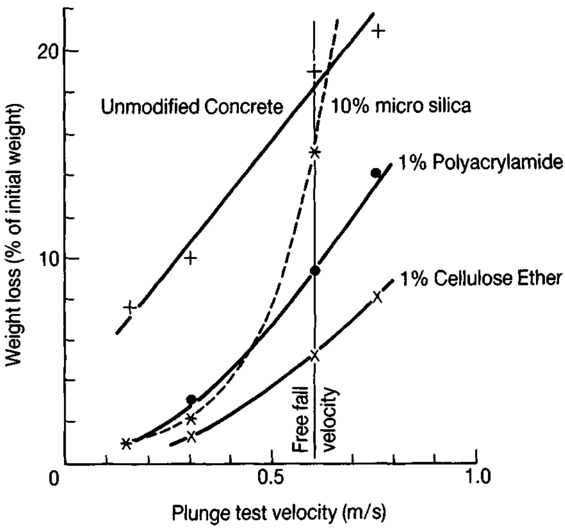



**Fig. 1.11** Influence of plunge test velocity on weight loss

from two hoppers, once in air and another time through water. The upper hopper is filled loosely with concrete, then a trap door is opened allowing the concrete to drop into the lower hopper. The concrete is then allowed to fall over a smooth steel cone, in air or through water, and scatter on to two concentric wooden discs. The weights of fresh concrete and sieved and oven-dried coarse aggregates which were collected from the two discs are used to determine the separation index (*SI*).

### 1.6.3 Workability/flow

Workability and flow properties are very important for concretes used under water, as tamping and vibration to achieve compaction are impractical, and the full extent of the formwork needs to be filled from a relatively few specific pour locations. The standard slump and flow tests (BS 1881: Parts 102 and 105) are appropriate but it is interesting to note that where cellulose ether has been used to produce non-dispersible concrete the slump value gradually increases with time (up to 2 min after removal of the conical mould), and the diameter of the concrete continues to increase following the flow table test. It is common practice to allow sufficient time for the concrete shape to stabilize prior to taking a reading. Figure 1.12 illustrates the way in which slumps changes with time for a high slump concrete.

The US Army Corps of Engineers' standard test method, CRD-C32-84, can also be used for determining the flow of concrete intended to be placed underwater using a tremie.[14]

The value 'slump flow' can also be used[8] where the mean diameter of the concrete in the slump test is measured.

### 1.6.4 Chemical compatibility

The chemical compatibility between non-dispersible concrete admixtures and cement needs to be assessed. To determine the influence (usually

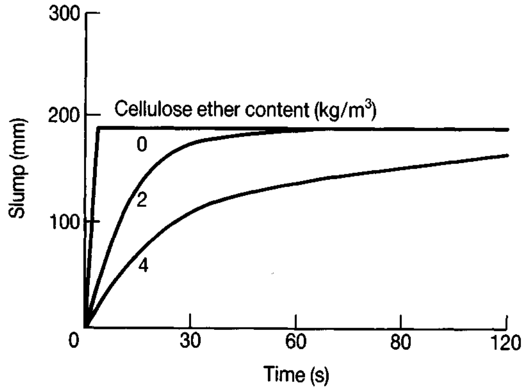



**Fig. 1.12** Influence of cellulose ether on the slump—flow relationships

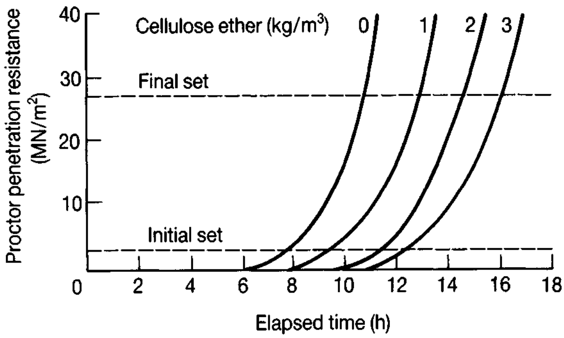



**Fig. 1.13** Influence of cellulose ether on setting time

retarding) of an admixture on early age hydration, the rate of heat evolution using thermocouples in insulated control and live specimens can be used. Of more direct practical value is the speed of setting. Typical values obtained using the Proctor Probe apparatus are given in Figure 1.13.

The rate of gain of strength can be determined by casting multiple specimens and testing at intervals over several weeks. Once again comparison with control specimen results enables the influence of the admixture on hydration to be assessed. Alternatively, the modulus of elasticity can be determined electrodynamically. This has the advantage of using the same specimens at each interval of time.

## 1.6.5 Strength and durability

Strength and durability are essential qualities and methods of measuring the effectiveness of non-dispersible concrete admixtures at maintaining strength following free fall through water are important. Much ingenuity has been used to develop such tests. Production of cubes by dropping concrete into moulds placed in water tanks is the most common approach but does not readily simulate practical conditions. A better approach is to produce 300 mm diameter castings in moulds which include simulated reinforcement. These need to be sufficiently large to enable 100 mm diameter cores to be cut to provide the test specimens.

The long-term durability of concrete containing the normal range of admixtures is well established. Less direct evidence is available for non-dispersible admixtures, particularly in terms of synergistic effects. However, the addition of micro silica to enhance the strength and durability of concrete has become established practice. There is over 15 years of evidence of the durability of non-dispersible concretes containing cellulose ether, and acrylic latex has been used to enhance the properties of hydraulic cement concretes (at much higher proportions than are used in non-dispersible concretes) for well over 10 years. The long-term durability is not therefore likely to be reduced by the use of these admixtures and, in view of the more reliable quality achieved, durability is likely to be enhanced.

### 1.6.6 Full-scale tests

Laboratory tests rarely reflect practical conditions. Unless first-hand experience of the actual placing conditions is available, it is good practice to cast trial sections of projects under the prevailing conditions. Sampling the newly cast and/or hardened concrete will enable the proposed process and materials to be assessed. The costs involved are justified in view of the financial and safety implications of failure.

## References

1 Techenné, D.C., Franklin, R.E. and Erntroy, H.C. (1975) Design of Normal Concrete Mixes BRE, Department of the Environment, HMSO.
2 American Concrete Institute (1991) *Recommended Practice For Evaluation of Strength Test Results of Concrete, ACI 214-77 (reapproved 1989), ACI Manual of Concrete Practice, Part 2.* American Concrete Institute, Detroit, MI.
3 Concrete Society (1990) Underwater Concrete—Technical Report No. 35.
4 Road Research Laboratory (1950) Design of Concrete Mixes, 2nd Edition, HMSO.
5 American Concrete Institute (1992) Manual of Concrete Practice. Guide to Durable Concrete, ACI 201.2R-92.
6 Construction Industry Research and Information Association (1984). *The CIRIA Guide to Concrete Construction in the Gulf Region.* SP31.
7 British Standards 6349: Part 1. *1984 Maritime Structures General Criteria*, Amendment No. 4, July 1989.
8 Sogo, S., Haga, T., and Nakagawa, T. (1987) Underwater Concrete Containing Segregation Controlling Polymers. The Production, Performance and Potential of Polymers in Concrete. (Ed. B.W. Staynes), International Congress Polymers in Concrete. Brighton University.
9 Ligtenberg, F.K., Dragosavić, M., Loof, H.W., Strating, J. and Witteveen, J. (1973) Underwater Concrete, Heron Vol. 19, No. 3.
10 Tomlinson, M.J. (1986) Foundation Design and Construction, Longman (5th Edition).
11 Staynes, B.W. and Corbett, B.O. (1987) The Role of Polymers in Underwater and Slurry Trench Construction. The Production, Performance and Potential of Polymers in Concrete. (Ed. B.W. Staynes). International Congress Polymers in Concrete. Brighton University.
12 Sakata, M. (1983) Use of Acryl-Type Polymer as Admixture for Underwater Concrete, Polymer Concrete. (Ed. Dikeou and Fowler) ACI.
13 Davies, B.A. (1986) Laboratory Methods of Testing Concrete Containing Polymers for Placement Underwater. Marine Concrete '86—The Concrete Society.
14 US Army Engineer Waterways Experiment Station (1949) *Handbook for Concrete and Cement* (with quarterly supplements). US Army Engineer Waterways Experiment Station, Vicksburg, MS.
15 Hughes, B.P. (1961) Development of an apparatus to determine the resistance to separation of fresh concrete. *Civ. Eng. Publ. Rev.*, **56**, No. 658, 633–634.
16 Khayat, K.H. (1991) *Underwater Repair of Concrete Damaged by Abrasion–Erosion.* Technical Report, REMR-CR-37, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

# 2 Excavation and preparation, design and installation of formwork

P J Scatchard

## 2.1 Introduction

On land, formwork for concrete has the prime function of supporting the concrete in its liquid/plastic phase. Secondary functions are to give shape and texture to the exposed faces of the structure.

Under water this changes. Whilst its prime function is still to support the concrete, texture is not important but the formwork must provide protection against washout of cement and scour due to movement of water.

Two fundamental differences exist in working through and under water compared with on the surface:

- if the operator of equipment is located above the surface of the water and working through the water then surface reflections or contamination may prevent him seeing the construction site
- if the operative is working under water he may or may not be able to see the work and his ability to carry out even simple manual tasks will be severely constrained.

Additionally, liaison between surface and bottom will be complicated by difficulties in communication and loss of orientation. Because of this the practicality of each operation required in the preparation of and in fixing and striking formwork must be analysed in detail.

Although there have been major advances in equipment, tools and underwater breathing apparatus, there is little that is essentially new in underwater formwork. The Victorian engineers faced some of their greatest challenges in the construction of harbour works and many of the techniques devised or developed by them to facilitate construction under water are still in use today. These include mass bag work, reusable formwork, precast blockwork and slabwork, precast concrete caissons, tremie and under water skips, as well as mammoth temporary works such as the travelling shield described by Kinipple in a paper on concrete work underwater given to the Institution of Civil Engineers in 1886–87.[1]

Some of these techniques, such as prefilled bagwork and coursed masonry, had already been in use for many centuries but only on a much smaller scale and in shallower water. The introduction of iron and steam-powered mechanical plant and later the development of diving equipment and techniques enabled construction to move into deeper, more exposed water.

A series of papers given at the Institution of Civil Engineers in the 1886–87 session (Carey, Kinipple, *et al.*) give a good idea of the state of the art in those days and since then, with the exception of flexible formwork and antidispersion admixtures, little has really changed.

## 2.2 Excavation and preparation of foundation

### 2.2.1 General

As in any construction project, the first essential in underwater construction is a secure foundation. In principle this will be the same as on land but may have the added risk of scouring of sedimentary and granular materials or soft rock by waves or currents.

Material to be removed in preparing a foundation may range from liquid mud, having the consistency of a thick soup which can only be removed by pumping, to rock, which must first be broken up by drilling and blasting before being removed.

### 2.2.2 Excavation

Excavation in shallow water may be carried out using conventional land equipment such as back acter, face shovel, grab or drag line, mounted on a barge or pontoon (or, in very shallow water, working normally). The effectiveness of this equipment would depend on the depth of water and type of material to be excavated. Free-fall equipment such as grab and drag line will lose some of its efficiency owing to the reduced submerged weight of the bucket and to the resistance of the water. There will be a tendency for finer material to be washed out of any open bucket on its way to the surface.

For excavation of softer/finer sediments or when working in deeper water, the use of dredging equipment will be necessary. Dredger types include air lift, jetting and suction pumps for lifting fine or loose materials, pumps allied to cutting heads for firmer material and bucket ladders.

Control of cutting depth may be complicated by tides and waves, by variations in the nature or firmness of sea bed deposits and by the fact that the operator cannot see the excavated surface. However, plant having precise level control (e.g. cutter suction, bucket ladder) and having a stable reference level (e.g. laser beam) can dredge to close tolerances in suitable materials.

Excavation in rock will normally require the use of drill and blast techniques. Arisings may be cast aside or loaded into barges for disposal elsewhere.

### 2.2.3 Filling and screeding

Accurate placement of fill underwater is even more difficult than excavation. Materials may be dumped from barges or placed by grab but the thickness/surface level of deposits will be uneven unless it is subsequently screeded. Carefully controlled dumping from side dump barges will give much more even deposits than the use of grabs or bottom dump barges but the unevenness of the sea bed will still be reflected by the top of the fill layer. Screeding techniques range from overdumping and re-excavation to profile to setting up screed boards and hand trimming by diver. The use of heavy screeds dragged along preset rails by mechanical plant (winch, tug, etc.) in any appreciable depth of water will require an expensive, sophisticated plant spread and control system.

### 2.2.4 Final preparation

Final trimming of foundations under water may be carried out by divers using hand tools. This is likely to be expensive and it will usually be preferable to design the structure and formwork to be tolerant of an uneven foundation so that trimming of high spots is not necessary.

However, removal of all loose and/or soft material from the foundation may be essential to give a firm foundation and to prevent future scour problems. Diver-operated water jets or airlifts will generally be the most appropriate way of clearing small quantities of loose materials and sediments.

Where depth of water or other conditions preclude the use of dredging or foundation preparation, the structure must be designed to penetrate softer strata or must be supported on piles and have suitable scour protection.

## 2.3 Tolerances and setting out

### 2.3.1 General

The difficulties in accurately locating, orientating and levelling points under water are considerable. Precise survey techniques available on land are, in general, either not suitable for use under water or the necessary equipment has not been developed. In good conditions (good visibility, no currents, etc.) basic equipment such as tape, spirit level, square, plumb-bob and string line can be used under water as could, in theory, the optical level or theodolite if they were available.

Distance can be measured reasonably accurately under water by acoustic means provided that the equipment has been calibrated to ambient water temperature, salinity and density, but these may vary considerably throughout the water column. Depth may be measured by pressure-sensitive equipment but the accuracy of this will also depend on a detailed knowledge of water density and any fluctuation in the water surface (wave, tide, etc.) will affect the pressure at depth. Horizontal angles accurate to 1° or so can be measured by means of magnetic compass (provided that there is no ferrous metal in the vicinity and that sight lines are adequate).

Equipment available for transfer of level and position from the surface includes acoustic transponder array, pressure-sensitive levels and sonar, but none of these can be considered as precision instruments.

Four separate problems require solutions in establishing the level and location of a point on the sea bed in relation to the land:

- level and coordinates of a reference point on the surface
- transfer of horizontal coordinates to the sea bed
- transfer of level to the sea bed
- setting out on the sea bed from the transferred reference points.

### 2.3.2 Coordination of offshore reference point

The level and position of a near-shore reference point, in sight of land and on a stable platform, can be fixed very precisely using terrestrial survey techniques.

Offshore, out of sight of land or out of range of optical instruments, alternatives such as range/range hyperbolic (Decca, Telurometer) and more recently satellite position fixing and levelling systems are all commercially available. Levels may be related to the water surface and thus to a reference point on land, but this too will be uncertain, if not inaccurate, because of variations in the water surface level due to tide, sea bottom profile and barometric and wind effects.

### 2.3.3 Transfer of horizontal coordinates to the sea bed

Techniques used on land for transferring positions vertically, such as plumb or triangulation are not likely to be feasible through water unless it is shallow, still and clear. The only alternative is three-dimensional triangulation using acoustic equipment which may allow coordination of a point on the sea bed to accuracies of say $\pm 1$ m under ideal conditions.

### 2.3.4 Transfer of level to the sea bed

Unless conditions are such that a chain, tape or staff can be utilized, transfer of level/measurement of depth from the surface must be carried out using either a pressure-sensitive level or echo sounder. Both of these are affected by water density and temperature and by how well the surface level of the water can be defined. Accuracies of better than ±100 mm must be considered unlikely even with the most sophisticated equipment.

### 2.3.5 Setting out on the sea bed

This is most likely to be effected by divers using basic setting out equipment such as tape, spirit level and string line. Tasks beyond the range of such techniques will have to be carried out either from the surface or by using acoustic triangulation.

## 2.4 Selection of type of formwork

### 2.4.1 General

Formwork for use under water must:

- support the concrete in its designed profile during the plastic phase
- protect the concrete from scour, washout and abrasion until it has hardened
- be able to withstand static and dynamic loading due to concrete, tides, waves and currents
- tolerate inaccuracies in formation level or alignment of adjacent work
- be easily fixed into position.

It may be designed as part of the permanent works and be left in place or as temporary works either to be left in place or struck and reused.

Formwork intended to form part of the permanent structure will generally be steel or concrete, although more modern materials such as glass-reinforced cement or glass-reinforced plastic and traditional materials such as some hardwoods may have an appreciable useful life under water.

Temporary formwork designed to be struck and reused may be made from any economic and easily worked material, timber, steel and GRC/GRP being the most common. Formwork to be left in place but not having any permanent function may be made from any stiff material, e.g. steel or timber, or from flexible fabrics.

### 2.4.2 Permanent formwork

Common forms of permanent formwork are masonry, concrete blockwork, concrete bagwork, steel sheet piling and precast concrete panels. Unless such formwork is so massive as to be self-supporting it will require some system of support until the *in situ* concrete has been placed and gained strength. Concrete blockwork and masonry are usually keyed to the concrete hearting by laying alternative stretcher and header courses; concrete panels and steel sheet piles should be anchored to the *in situ* concrete by means of hook bolts or ties.

Examples of the use of permanent formwork include the north wall at Brighton Marina, caissons at Brighton Marina, Peacehaven sea wall and Tyne piers.

### 2.4.3 Reusable formwork

Reusable underwater formwork must be robust, tolerant of uneven foundations and, above all, simple to erect and strike. It should be prefabricated on the surface into panels as large as can be handled by divers aided by available plant. The method of tying/propping to withstand horizontal loading will depend on the particular circumstances but will in principle be the same as might have been adopted had the works been on land, bearing in mind the difficult working conditions under water and the possibility of loading from either or both sides.

Particular attention must be paid to joints between panels or to adjacent work and to sealing the inevitable gaps between form and sea bed. This will not normally be difficult to achieve in a rough but effective way provided that differential pressures across the shutter (e.g. due to the rise and fall of the tide or wave action) do not cause joint fillers to be washed out. Panels must, of course, be negatively buoyant and the provision of additional ballast after erection may be an aid to stability.

### 2.4.4 Flexible formwork

Whereas steel, timber, concrete and masonry have been used for many years in underwater formwork, with few real advances except in materials technology, there has been considerable development in the use of flexible formwork, allied to improvements in pumping equipment.

Prefilling hessian or canvas bags with concrete and placing them underwater before the concrete has hardened has long been a common technique, especially for underpinning, filling of scour holes and permanent formwork. For example, the breakwater protecting Newhaven Harbour constructed in the 1880s is founded on 100 ton concrete 'sack blocks' for which purpose-designed batching mixing and filling plant and a special placing vessel were constructed.[1]

In recent years however, techniques first patented in 1920 based on tailored bags and mattresses, but now using high-strength synthetic fabrics able to retain cement-sized particles whilst allowing water to bleed off, have been developed. Mattresses or forms are placed, deflated, in position on the sea bed or within the void to be filled and grout or concrete is pumped directly into place, so inflating the bag to its design profile. The concrete is separated from the surrounding water by the fabric so that no special precautions are necessary to prevent washout. The flexibility of the fabric allows it to mould itself to the sea bed, existing structure or pipe as required. Careful tailoring allied to internal ties between faces will allow simple shapes to be formed without additional support, but the height of such shapes is limited to about 1 m without additional external or internal support.

One variation on the tailored bag is the fabric sleeve or stocking wrapped round a pile and closed by a zip fastener before being filled with grout or concrete. Another is grout mattress only a few centimetres thick, possibly incorporating porous filter zones, used for bank or scour protection.

Non-porous flexible forms have been used to lift and permanently support pipework. Flexible formwork, adequately supported, has been used as single face formwork on encasement work where its principle advantage is probably lightness and ease of handling.

Enhanced early strength of concrete placed in flexible porous forms is claimed as a result of bleeding off excess pore water under pressure from the fluid concrete, but this effect is doubtful except in the case of very thin sections.

Flexible formwork will normally be regarded as being purely temporary although the incorporation of parafil fibre or rope reinforcement is feasible and has been used to maintain the integrity of mattresses designed to crack and take up settlement in the substrate.

## 2.5 Design loadings

### 2.5.1 General

In addition to pressures from fluid concrete, formwork for concrete to be placed under water may have to be designed to withstand additional hydrostatic loads due to water level (e.g. tidal) variations and dynamic loads due to waves and currents. In exposed conditions these may be many times greater than the pressures due to submerged concrete.

Careful consideration of the conditions that the formwork will be required to withstand both before and after placing the concrete is therefore necessary and reference should be made to BS 5795 – Code of Practice for Falsework, Secton four.

### 2.5.2 Out-of-balance hydrostatic loads

Water levels within formwork erected in tidal waters will tend to lag behind the changing level in the surrounding open water to a greater or lesser extent depending on the permeability of the formwork. If the formwork is close fitting and properly sealed to the sea bed and to previous pours, there will be a potential out-of-balance pressure equivalent to the full range of the tide. This may be increased if the cell is dewatered or if waves overtop and so raise the standing water level inside. This hydrostatic pressure is simply calculated from the difference in water levels between inside and outside the formwork and may be exerted on either side of the formwork depending on particular circumstances. Clearly, pressures on seals between the sea bed and bottom of the form in deep pours may be considerable.

In assessing the tidal range, reference should be made to the Admiralty or other tide tables such as those published by the US National Ocean Survey levels and the possibility of significant variations in these due to wave action or surges, for example, must be considered. Such variations are usually a matter for statistical analysis of available records. In the absence of these, an informed judgement must be made for each site. Still water levels of 1 m or more above predicted levels are not uncommon in UK waters.

### 2.5.3 Waves

Temporary works in or adjacent to open water are likely to be subjected to wave forces, the magnitude of which will depend on depth of water, sea bed profile, shape of structures, length, height and period of incident waves and wind conditions.

A realistic assessment of the maximum incident wave and the frequency of occurrence of limiting wave conditions must be made and account should be taken of the probability of occurrence of extreme wave or water level events or combinations of events during the period of construction.

Calculation of appropriate wave parameters either from wave records or wind records is a highly specialized matter and advice should be sought from an experienced maritime engineer.

Works in sheltered rivers or waterways may be subjected to the wash from passing vessels and the magnitude of these waves can be established by direct observation.

Having defined the wave climate to be taken into account, the consequent forces on the formwork must be calculated. These can be very significant and in some circumstances will dominate the design of formwork.

Appendix F in BS 5975 proposes simple equations for the calculation of forces on vertical faces due to non-breaking waves. These will not be applicable to curved or sloping faces where forces, particularly due to breaking waves impinging on re-entrant faces, can amount to hundreds of kN/m$^2$. Typical values for wave forces on structures are 50–100 kN/m$^2$ in coastal work.

Further advice is given in BS 6349 'Code of Practice for Maritime Structures' and in the US Army Corps for Engineers 'Shore Protection Manual' on both the assessment of incident waves and on the calculation of wave forces on structures, but the advice of an experienced maritime engineer is recommended for sites with any real exposure to wave action.

### 2.5.4 Currents

Forces due to even small currents acting on large formwork panels will be significant, particularly when handling formwork and before the concrete has been placed so that current forces may be a limiting factor in underwater formwork design.

Having measured existing currents (and assessed the risk of flood conditions if working in a river), some assessment must be made of the likely changes to current patterns and velocities due to the presence of the formwork and of adjacent works. In simple cases a desk study may be sufficient, but in more complex situations or where current forces are potentially critical it may be necessary to construct a mathematical or physical model.

Dynamic pressures due to currents may be calculated from the equation $F = 500ACV^2$, where $A$ is the effective area normal to the flow, $V$ is the current velocity and $C$ is a coefficient appropriate to the shape of the structure (e.g. 1.86 for flat surfaces or 0.63 for cylindrical surfaces). The force exerted on flat panel formwork is thus of the order of $V^2$ kN/m$^2$ which, when related to panels of any size, may result in horizonal forces of several tonnes.

In tidal situations formwork may usually be handled and fixed during periods of slack water. In rivers, where the current is continuous, it may be necessary to shield the work within a cofferdam.

### 2.5.5 Propeller wash

Ships' propellers create temporary and localized but powerful water currents which may be significant if close to the formwork, as might the movement of water past a hull moving along a narrow shallow waterway.

## 2.6 Selection of type of formwork

In considering whether to use permanent or temporary, stiff or flexible formwork, regard must be paid to:

- ambient conditions during construction
- conditions in service

- design life of structure
- method of placing concrete.

Where calm quiet conditions during construction cannot be guaranteed, flexible formwork or any formwork requiring significant input by divers will not be suitable and only very strong or massive forms (e.g. steel sheet piles, large mass blockwork) installed from the surface will be appropriate.

Where the new structure is to be founded on a rough, rocky sea bed or is to butt up to an existing, uneven structure, the formwork will have to conform to the existing profiles. It is not likely to be feasible to measure these profiles and prefabricate the formwork to fit exactly so that it will have to be designed to be easily adjustable or provision made for sealing the residual gaps.

If conditions in the open water are unsuitable for divers then formwork must be designed for installation into its general position from the surface with finishing touches made by divers working within the protection of the form. (In this instance, however, there may be dangers to the divers due to water currents through the gaps in the formwork.)

Where concrete is to be placed within an existing cavity of irregular shape, the use of a tailored flexible form will obviate the need for special admixtures or precautions to prevent washout of cement. It will also allow the cavity to be completely filled without the need for face formwork or for sealing minor cavities and fissures within the structure. Flexible mattress forms are of great value where a thin layer of concrete is required over a large horizontal or sloping surface as in erosion control works.

In high-energy areas (vertical and re-entrant faces or cusps subject to wave attack), any open joint or weakness in a face will be rapidly eroded so that fabric formwork or thin precast concrete panels backed by inferior concrete should not be used.

Permanent steel formwork, e.g. sheet piles, will have a relatively limited life above about half tide so that although the concrete behind may be sound the superficial appearance will be poor and the corrosion of embedded steelwork such as tie rods will eventually damage the concrete itself.

Several examples of the use of different types of framework may be seen at Brighton Marina in structures built in very exposed conditions. These examples are discussed below.

Permanent formwork in the form of steel sheet piles was used to contain concrete in the mass concrete wall forming the northern boundary of the tidal harbour. The wall was built in the open sea without the protection of the breakwaters and this form of construction was chosen because of the exposure of the site and for speed of construction. Use of heavy section sheet pile permanent formwork minimized the need for temporary supports and allowed the placement of concrete infill using underwater skips rather than more expensive pumping or tremie. There was no need for any bed preparation as what little loose bed material that existed was adequately contained by the piles. The sheet piles were cut off at about mean high water, and above this level plain reusable forms robust enough to withstand the severe wave action were used.

The breakwaters themselves, also constructed in open sea conditions, did require limited diver intervention for bed preparation and for setting formwork. Permanent formwork (precast concrete caissons) and temporary reusable formwork (an adjustable skirt fixed to the bottom of the caisson) together contained the *in situ* concrete foundation plug. The caissons were constructed onshore using conventional slip form and fixed shutter techniques. The adjustable skirt was formed from steel drop panels retained by H-section guides bolted to the side of the caisson. All the divers had to do to drop each skirt was to cut a rope. Each panel and guide unit was removed simply by undoing two wing nuts and these were the only diver operations performed outside the caisson. Small gaps between panels and chalk bedrock were stuffed with sand bags and the final bed preparation was carried out by divers using airlifts working within the protection of the caisson.

Precast concrete plank formwork located in grooves in the caissons at caisson–caisson joints to retain *in situ* concrete in the joints proved easy to install from the surface but because of their small section proved vulnerable to very high wave forced generated by the focusing effect of the cusps. These joints were subsequently reinforced using full height forms to retain micro silica/OP concrete placed under water.

Because of the exposure of the site and the highly reflective vertical face of the breakwater, scour protection was necessary. In places this was constructed in tremie concrete retained by permanent precast concrete beam formwork. Elsewhere 2 tonne prefilled bagwork laid in two layers was used to form a flexible scour apron.

## References

1 Carey, Kinipple *et al.* Minutes of Proceedings of Institute of Civil Engineering. **Vol. LXXXVII.**

# 3 Underwater inspection

F Rendell

## 3.1 Introduction

### 3.1.1 The need for inspection

Concrete has now become a traditional material for construction, and its cost and versatility lend themselves to an economic solution to many construction problems. No material is invulnerable and concrete structures will be subjected to deterioration. The durability of concrete is therefore a vital factor in the in-service life of a structure. Elements of a structure that interface with water are subjected to a wide range of aggressive environmental conditions. This will lead to accelerated deterioration, and consequently maintenance becomes an important factor in the service life of the structure. As structures age the construction materials will degenerate and this, combined with external factors, such as overloading and impact damage, will further detract from the integrity of the structure, leading to a possible downgrading of the facility. Above the water line, deterioration is readily apparent and remedial measures can be taken to protect the structure. In the situation where a structure is partially or totally submerged, deterioration or damage below the water line is not immediately apparent. Often the problem is not detected until the structure has reached an advanced state of distress. Underwater inspection is therefore of great importance; effective inspection can be difficult and costly but it should be seen as an important facet of the operation of the maintenance programme.

The underwater inspection of structure covers a range of applications. Offshore concrete gravity platforms are subjected to high environmental loads, impact damage and the natural attrition of the marine climate. Reinforced and mass concrete have been used extensively for the construction of docks and wharves. These structures are subjected to much abuse from shipping combined with environmental factors. At the other end of the spectrum submerged concrete is to be found in river and canal structures, bridge piers, weirs and lock structures, and these may be threatened by undercutting, settlement and material deterioration.

Offshore concrete production platforms have been used in the North Sea since 1973. These structures are built with a high degree of quality control and are regularly inspected. There are two reasons for the in-service routine inspection of these structures:

- the owner needs to assess the general condition of the structure to ensure that it can continue to operate safely and will satisfy the requirements for the life of the platform[1]
- it is necessary to demonstrate to a Certifying Authority that the condition of the structure is acceptable for the issue of a Certificate of Fitness as required by the Government.[2]

The requirement for underwater inspection of docks and harbours is not so well defined. Many of these structures are old and in advanced states of deterioration. The construction methods and materials used on these structures were in some cases questionable. These structures are often subjected to considerable damage from shipping, and consequently there is a definite need for underwater inspection. Many enlightened operators have policies for routine inspection; however, there are many cases in which failure is the first sign of a problem.

River and canal structures have, in the past, suffered from a similar neglect. Above-water inspection and maintenance are well cared for, but underwater inspection is often neglected. In several cases this has led to the dramatic failure of structures (see Figure 3.1). Deep scour pits can form around bridge abutments, and if undetected and checked structural failure will normally result from overturning of an abutment. Over the past few years an increasing number of operators have redressed the problem by setting up comprehensive inspection programnmes.[3]

**Fig. 3.1** Llandeilo train crash, October 1987: collapsed bridge

To summarize, any part of a structure which is below the water line will be subjected to a variety of aggressive agencies. If undetected the consequence will be very serious and therefore underwater inspection must be seen as a vital part of a maintenance programme. The aim of the inspection should therefore be the identification of problems at as early a stage as possible so that remedial works can be instigated.

### 3.1.2 A strategy for inspection

The inspection of concrete is not an exact science. In the normal above-water situation good concrete inspection requires an integrated approach linking several inspection techniques. Misdirected inspection can lead to an erroneous assessment of the situation and consequently costly errors of judgement. In the underwater inspection of concrete the situation becomes even more arduous, divers are rarely specialists in concrete technology and the working conditions are generally not conducive to good inspection practices.

To conduct an informative cost-effective concrete inspection a carefully formed strategy is required. Good planning is the key factor in the operation. A structure may consist of a considerable area of apparently featureless concrete. A close inspection of the entire structure would be very expensive and therefore it is necessary to adopt a staged programme of work as follows:

- Preliminary inspection
  (identificaton of problem areas)
- Detailed inspection
  (to quantify deterioration)
- Appraisal of the situation
- Repair
- Monitoring.

The preliminary survey is probably the most important stage in the programme. This inspection should involve a reconnaissance of the structure and should identify problem areas for detailed inspection. Emphasis must be placed on briefing the inspection team on problem areas that should receive particular attention. It is advisable to carry out a preliminary inspection of newly constructed installations for construction blemishes and defects, such as construction joints. These may indicate possible sites of deterioration. This inspection will also be of value in establishing benchmark data for future inspections.

Good record keeping is an important factor in all stages of the work. Inspection reports should be cross-referenced to gain an overview of the deterioration of the structure. The detailed inspection should yield information that will enable the engineer to make an evaluation of the situation and consequently to propose maintenance work.

The final stage in the cycle will be the long-term monitoring of the repaired sections of the structure. These areas should now be given higher priority in the routine preliminary inspection programme.

In general, to achieve a cost-effective inspection of a concrete structure the inspection team must aim to collect relevant information and record it in a logical fashion. The following considerations should be considered:

- *Behaviour of concrete.* The planner and inspector must have a good knowledge of the behaviour of concrete and methods of concrete construction and be aware of possible modes of deterioration.
- *Identification of problem areas.* Critical areas must be defined and given priority.
- *Mode of inspection.* This should encompass the selection of the type of underwater operation (i.e. diver or ROV), the establishment of a reference system and selection of inspection techniques.
- *Standard reporting procedures.* To achieve a coherent communication of information, it is of paramount importance to standardize the notation for reporting observations.
- *Recording procedures.* The inspection will yield data that permit the formation of a picture of the structure. All records should be of a standard that will allow cross-referencing of previous work.

## 3.2 The behaviour of concrete in submerged structures

### 3.2.1 General

With the advent of the use of concrete for the construction of offshore production platforms, the UK Department of Energy instigated the 'Concrete in the Oceans' project.[4] This programme of work examined various aspects of the behaviour of concrete in the marine environment. This work has now been continued into a second programme, 'Concrete in the Nineties' (COIN).

Concrete in the submerged state is subjected to a wide range of factors that can cause deterioration. The principal causes of loss of serviceability in concrete are as follows:

- chemical attack on the concrete matrix
- depassivation of the reinforcement
- erosion and scour damage
- impact damage

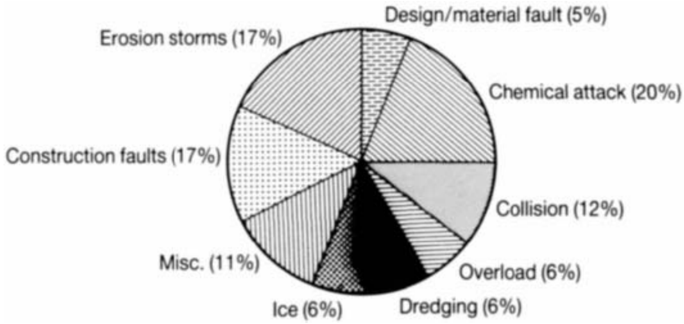



**Fig. 3.2** Causes of damage to marine structures

A detailed discussion of the mechanisms and risk of deterioration of concrete underwater is presented in Chapter 6, with some of the principal problems summarised in the following sections.

Figure 3.2 shows a typical frequency of damage distribution.[5]

## 3.2.2 Chemical attack

Concrete is susceptible to chemical attack when exposed to certain conditions, and the attack usually causes disintegration of the concrete matrix and subsequent loss of strength. A critical factor in this process is the permeability of the material. Concretes with high permeabilities will allow aggressive agents to permeate the concrete matrix and thereby accelerate the rate of attack. Consequently, high-durability concrete is one of the critical factors in the specification of the material.

Sea water has a number of chemical reactions that can cause degeneration of concrete.

*Sulphates:* sulphates in water can be especially damaging to concrete.[6] Sulphates react with the tricalcium aluminate in the cement paste and form ettringite, which will cause swelling and cracking. Sulphate will also react with the calcium hydroxide and form gypsum, which is a soft material that is easily removed by water movement. In an anaerobic environment sulphate-reducing bacteria will produce hydrogen sulphide and sulphuric acid which will attack the concrete.

*Alkali silica reaction (ASR).* Another cause of concrete degeneration is the alkali silica reaction. The most common form of this reaction occurs when the alkaline pore fluids in the concrete react with siliceous aggregates. The result of the reaction is the formation of a gel that expands when in the presence of water. This expansion causes map cracking of the concrete, opening the matrix to other forms of attack.

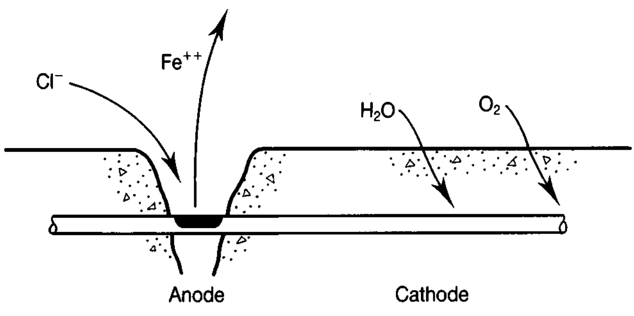
*Acid attack:* Portland cement concrete is generally not very resistant to attack by acids. The deterioration of concrete by acids is primarily the result of a reaction between acid and the products of hydration of cement. Calcium silicate hydrate may be attacked if highly concentrated acid exists in the environment of the concrete structures. In most cases, the chemical reaction results in the formation of water-soluble calcium compounds that are then leached away.[8] In the case of sulphuric acid attack, additional or accelerated deterioration results because the calcium sulphate formed may affect the concrete by the sulphate attack mechanism as discussed above.

*Aggressive water attack.* Some waters have been reported to have extremely low concentrations of dissolved minerals. These soft or aggressive waters will leach calcium from cement paste or aggregates. For aggressive water attack to have a serious effect on concrete structures, the attack must occur in flowing water, which keeps a constant supply of aggressive water in contact with the concrete and which washes away aggregate particles that become loosened as a result of leaching of the paste.[9]

*Alkali–carbonate rock reaction.* Certain aggregates of carbonate rock have been found to be reactive in concrete. The results of these reactions have been characterized as ranging from beneficial to destructive. The destructive category is apparently limited to reactions with impure dolomitic aggregates and are a result of either dedolomitization or rim-silicification reactions.[7]

### 3.2.3 Corrosion of reinforcement

Steel in concrete is normally in a passive state owing to the high pH of the cements film that coats the reinforcement.[7] Once the passive film around the steel is broken, corrosion can commence. The corrosion of reinforcement takes place as a cell, as shown in Figure 3.3.




$$3Fe + 4H_2O \rightarrow Fe_3O_4 + 8H^+ + 8e^-$$

$$\tfrac{1}{2}O_2 + H_2O + 2e^- \rightarrow 2OH^-$$

**Fig. 3.3** The corrosion cell

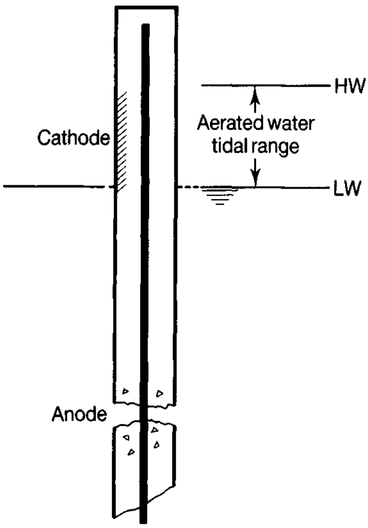



**Fig. 3.4** The macro corrosion cell

This cell consists of an anode and a cathode. The anodic cell is created once the passive layer is broken. The associated cathodic area occurs at any site where conditions are conducive. Three common causes of depassivation of the reinforcement are cracking, carbonation and the ingress of chlorides.

In the submerged state there is little oxygen available to drive the cathodic element of the cell, and consequently a structural element that is totally submerged will show little evidence of rebar corrosion, even when extensive depassivation of the bar has taken place. If the depassivated zone is linked to a zone of the structure that has conditions conducive to a cathodic reaction, then a macro corrosion cell will develop as illustrated in Figure 3.4.

In oxygenated zones of a structure the corrosion products are highly expansive and this is generally associated with surface cracking and spalling of the concrete. In the submerged elements of the structure corrosion products are a different form, namely magnetite. This oxide of iron is a non-expansive compound and therefore difficult to detect from the surface. This reaction can, under macro cell conditions, degenerate reinforcement to a black paste at a significant rate.[10,11]

It was shown by Wilkins and co-workers[12,13] that submerged cracks of 0.5 mm and wider will permit the instigation of a corrosion cell. Smaller cracks in the submerged state tend to self-heal owing to blocking of the pores and crack by corrosion products and salts.

### 3.2.4 Effect of marine growth on concrete

Living organisms that grow on a structure will affect the condition of the concrete. Biczok[6] considered these effects and drew the following conclusions.

Plants of a lower order, such as large and hydroids, can be found on continuously moist or submerged zones of a structure. These lower orders of plant life have the most significant effect in the submerged state. Under these conditions algae and hydroid colonies form a dense, fibrous, tough covering; this layer tends to seal the concrete and decrease the gas permeability, thus reducing carbonation and the availability of oxygen to promote corrosion to the reinforcement. There is much evidence that indicates that dense growths of algae in reservoirs and shafts, for example, will completely seal the concrete, making the structure watertight. The high pH of concrete generally prevents higher plant forms from becoming established; however, with the onset of carbonation, and consequent reduction of pH, mosses and lichens may become established above the water line. Concrete under such conditions may show signs of surface bleaching and moderate surface scaling to a depth of a few millimetres; this is probably due to the release of carbonic acid developed by the decomposition of the plants. Many plants of higher order, such as sea weeds, have root structures. These organisms may break down the concrete by a physical bursting action; in addition, the release of carbonic and organic acids will have a further detrimental effect. An additional factor that must be considered is the release of carbon dioxide in daylight hours, which may increase the rate of carbonation. Several marine structures have shown signs of degeneration after relatively short periods of service. This has been attributed to the action of sulphur developed in the decomposition of seaweed. In general, seaweeds offer no protection to concrete and can be seen as a potentially aggressive agent; consequently, their presence, type and density are of importance in the inspection programme.

### 3.2.5 Scour, impact and abrasion damage

On offshore and harbour structures the principal cause of impact damage is shipping. Offshore structures, such as production platforms, are frequently damaged by materials dropped from supply vessels. The consequence of impact damage is generally a localized structural failure; these failures can range from local spalling to major structural distress. Thus any impact to a structure must be investigated with considerable care. In the event of a major failure, analysis of the structure will have to be carried out, to ascertain the integrity of the unit considering the fact that elements of the structure have passed their yield point. A consequence of impact is possible depassivation of reinforcement through cracking of a member; this will lead to long-term deterioration caused by the corrosion of the reinforcement.

Scour is a phenomenon that is caused by high water velocities. The bed material around the structure may be removed and this may lead to overturning or differential settlement of the structure. A second consequence of scour is the direct degeneration of the concrete. High local water velocities induced by propellers will cause local scouring of concrete. This factor is often found in situations where chemical degeneration of the concrete matrix is occurring; the combined effects become complementary and lead to accelerated loss of section.

Abrasion of concrete can be caused by water-borne solids or cables, for example. The effect is to remove the surface of the concrete, thereby reducing the cover to the reinforcement. When water velocities exceed 12 m/s cavitation damage will occur; the resulting damage will manifest itself as localized areas of erosion. This problem may be encountered at intakes and spillways where local water velocities are high.[9]

## 3.2.6 Problem areas

To ensure a cost-effective survey of an installation, it is necessary to identify areas that will be prone to deterioration. The causes of damage are varied and often interrelated. Impact damage can weaken a structure; spalling and cracking so induced will leave the concrete and steel vulnerable to chemical attack, which will then be accelerated by scour action induced by waves or propeller turbulence. Figure 3.5 sets out typical areas of vulnerability in marine structures.

The splash and inter-tidal zones of a marine structure are generally the most at risk. In these areas chlorides and carbonation depassivate the reinforcement and this is subsequently followed by corrosion and disintegration of the cover concrete. These effects are accelerated by freeze–thaw conditions and constant attrition by water action. The corrosion

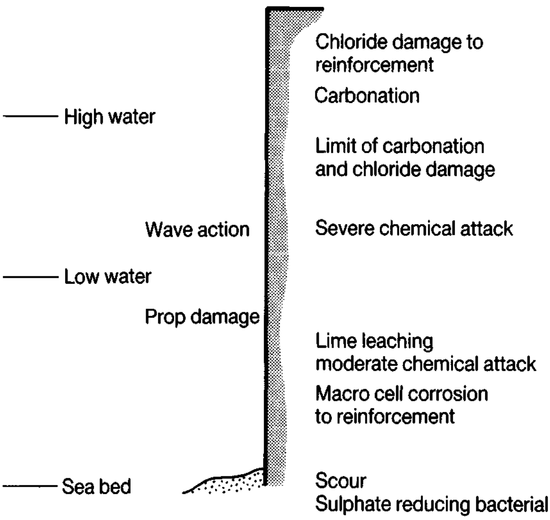



**Fig. 3.5** Problem areas in marine structures

process in these areas is normally very rapid owing to the presence of oxygen and water and a constant replenishment of salts. The corrosion products are expansive and the damage is easily identified by cracking, rust staining and spalling. The problems associated with chloride damage will not be dealt with here but the reader is referred to CIRIA or ACI publications[7,14] for further reading.

The lower part of the tidal zone, and the shallower parts of the submerged zone, are generally prone to impact damage, chemical attack and scour. On a marine structure propeller damage can remove highly significant volumes of chemically softened concrete. With older mass concrete structures, often constructed with inferior concrete, chemical attack is often prevalent owing to the porosity of the concrete. Once the cement paste of the concrete degenerates, the water action from ships' propellers and wave action will act together to form large voids that will eventually undermine the structure.

In the deeper parts of the submerged zone the concrete will not be subjected to such violent attrition. At depth, the permeability of the concrete tends to decrease owing to pore blocking of the surface of the concrete. Chlorides will permeate the concrete and depassivate the reinforcement. The lack of oxygen will detract from the activity of the corrosion cells thus induced. A possible problem that may be encountered in the permanently submerged section of a reinforced concrete structure is the effect of macro cell corrosion. In areas where the bar becomes depassivated it is possible to develop a corrosion cell between the depassivated area and a cathodic site in the inter-tidal zone. It has been shown that the corrosion rates are significant with very little visual evidence to indicate that the mechanism is taking place.[10]

During the construction process it is inevitable that construction defects will occur. Defects are often minor and have no effect on the long-term integrity of the structure, but in some cases construction defects can lead to a weakening of the concrete. Areas of poor compaction, for example, will have a high permeability and will be more susceptible to chemical attack, and plastic settlement cracks can provide a path directly to reinforcement and lead to corrosion.

Owing to the difficulty of maintenance and repair of structures under water, careful quality control and remedying of defects immediately after construction are essential.

Another important aspect of inspection is an assessment of marine fouling. High densities of fouling can increase the hydraulic loading on a structure; this fouling will also obscure other defects and in certain cases further weaken concrete that is being subjected to chemical attack.

Listed below are a typical set of areas of vulnerability:

- highly stressed areas
- areas liable to impact scour and abrasion

- repaired sections of the structure
- areas with construction defects
- construction joints

## 3.3 Inspection

### 3.3.1 Method of inspection

The inspection of the underwater section of any structure will be expensive and difficult. The nature of the underwater environment seriously detracts from an inspector's ability to make a good observation of the structure. Submerged elements of the structure are often shrouded in growth that will obscure the surface of the concrete. The water conditions are normally such that the visibility is very limited, sometimes zero, and furthermore water movement can make working conditions very difficult.

To maximize the effectiveness of underwater inspection, careful consideration must be given to the selection of the mode of inspection operation. There are two methods of operation that are in common use: diver and remote operated vehicle (ROV).

#### 3.3.1.1 Diving

The use of the diver has been the traditional mode of inspection. Diving is a hazardous operation and is therefore carefully controlled by safety regulations. In the UK, diver safety is regulated by the Health and Safety Executive (HSE). Statutory safety regulations control the qualification and training of the diver and the procedures used in the diving operation.[15] The diving equipment, operations and procedures can also be found in the US Navy Diving Manuals, Volumes I and II[16] (1975).

The principal constraint on a diver is depth. As a diver descends, nitrogen gas is absorbed into his blood system, which has two effects. At depths greater than 30 m the diver begins to suffer from nitrogen narcosis; this phenomenon has a similar effect to that of alcohol. Narcosis will therefore render the diver unfit to make any coherent observations and, more important, will endanger his ability to work safely. The second constraint on the diver comes on surfacing; the gases absorbed into the body have to diffuse as the diver decompresses. If the rate of decompression is too fast, the diver will suffer from the 'bends'; that is a potentially lethal condition and consequently decompression is carefully controlled. Table 3.1 gives examples of the time limits that are imposed on air divers.[17] Dive times in excess of these values will necessitate the use of decompression procedures such as in-water 'stops' to allow the diffusion of gas from the system.

**Table 3.1** Maximum no-stop dive durations

| Depth (m) | Dive time (min) |
|---|---|
| 12 | 200 |
| 18 | 60 |
| 30 | 25 |
| 40 | 10 |
| 58 | 5 |

Diving at depths in excess of 50 m requires mixed-gas breathing, where helium is used to replace the nitrogen in the breathing mixture. Owing to the extensive decompression time required for deep dives, divers often work in saturation. In saturation diving the diver is maintained at the working pressure for a prolonged period; this is achieved by housing the diver, on the surface, in a hyperbaric chamber and then transporting him to the working depth in a diving bell, working pressure being maintained all the time.

Diving conditions will vary widely. At one extreme a diver can be expected to work in 2 m of water inspecting a bridge pier, and at the other extreme a dive operation offshore will require a diver to work at depths of 150 m on a concrete production platform. The training and qualifications of the diver will therefore vary according to the type of diving (see Table 3.2).

**Table 3.2** Diver qualifications (UK HSE Regulations)

| Standard | Description |
|---|---|
| Part IV | Self-contained breathing apparatus (Scuba). Light inspection work, up to depths of 30 m |
| Part III | Surface-supplied divers, inshore work, e.g. dock and harbour work |
| Part I | Basic air diving qualification for off-shore work. Depth of up to 50 m |
| Part II | Mixed-gas diving. Saturation diving techniques |

### 3.3.1.2 Remote-operated vehicles (ROV)

As the complexity of a diver operation increases, the cost rises. A development in underwater technology that obviates the use of the diver has been the ROV. The ROV is a surface-controlled unit that can be 'flown' underwater; it is controlled by thrusters and with a closed-circuit television (CCTV) system that acts as the operator's eyes. These units are now used extensively for inspection and intervention work. The obvious advantage of the system is the lack of risk to human life when working in high-risk situations. The basic component of the ROV is a controlled platform that can be steered by an operator using the CCTV system. The unit can then be enhanced with still cameras, manipulators and other items of purpose-made tooling. Small inspection units, such as the Hyball and

Minirover, have been developed to enable ROV technology to be applied to a wide range of applications. These smaller models of ROV are modest in price and therefore present an economic alternative to divers. A great advantage of a ROV is that it can be used in conditions that would place a diver in a dangerous situation.[18] Information on ROVs, including a general description of various types, can be found in ref. 19.

## 3.3.2 Cleaning of concrete structures

Before any underwater inspection can be carried out, it is necessary to clean any growth from the surface. Organic growth on concrete structures can provide protection, as in the case of algae, or can have a detrimental effect. The nature of the concrete surface is somewhat delicate; under aggressive conditions where chemical attack is taking place there may be considerable surface softening of the concrete. Consequently the cleaning process may well add to the deterioration of the structure. Fine crack damage to a structural element can instigate corrosion to the reinforcement; for the effective detection of cracks, it is important that the cleaning process does not scarify the surface to the point where these defects are obscured.

### 3.3.2.1 Survey and classification of fouling

An important aspect of the preliminary inspection will be to make an assessment of the type and extent of fouling on the structure. These observations will assist the inspection team to assess the liability caused by the fouling (e.g. excessive hydraulic drag, chemical attack) and to plan a cleaning programme. Organic growth can be roughly classified into two groups, hard and soft fouling. These groups can then be subdivided into more specific classifications based on the identification of plant types (see Figure 3.6).

The reporting of the density of fouling is best carried out by the visual estimation of percentage cover; with experience an inspector can make a quick visual assessment. Density charts, as shown in Figure 3.7 can be used to assist the inspector in this estimation. A more pedantic method of quantification involves the removal and collection of all growth in specific areas. The collected samples can be identified and enumerated at a later date.

### 3.3.2.2 Cleaning methods

Because of the need to make a surface examination of the concrete, highly abrasive cleaning techniques can be discounted. Methods such as needle guns, bush hammers, mechanical wire brushing and high-pressure jetting may remove not only the fouling but also the surface layer of the concrete.

More suitable techniques for cleaning concrete are hand cleaning with

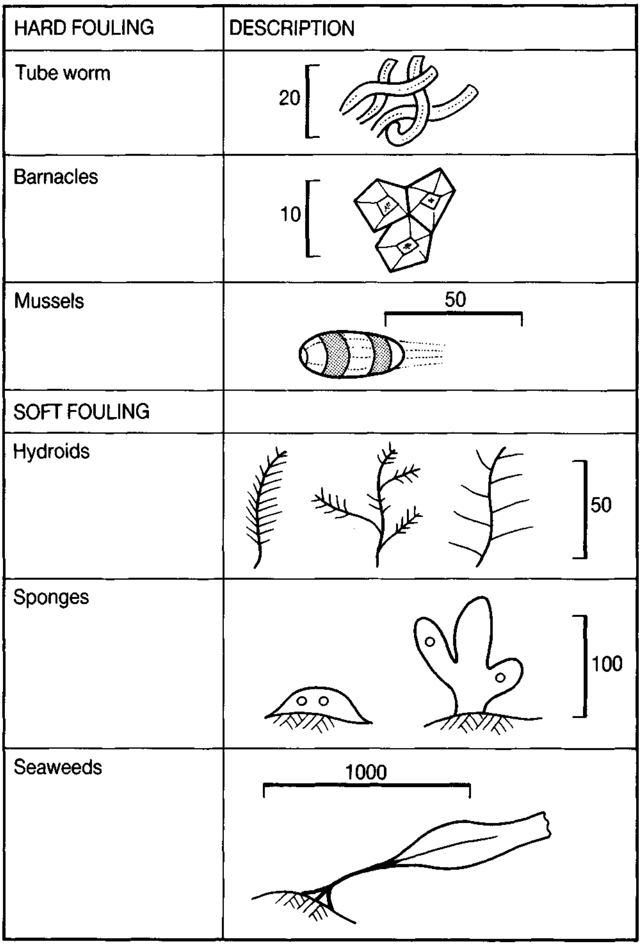



**Fig. 3.6** Marine growth

nylon brushes and scrapers. Hard fouling, such as barnacles and mussels, will require scraping. Soft fouling, such as seaweed, will have to be cut away and the surface cleaned up with brushing. Additional information on underwater cleaning procedures can be found in Refs 20 and 21.

## 3.4 Inspection techniques

### 3.4.1 General

Inspection of concrete can be carried out by a combination of techniques. Generally, no one method will reveal the complete picture of the condition of the structure; therefore, the inspection programme should assemble field information from several sources to enable assessments to be made.

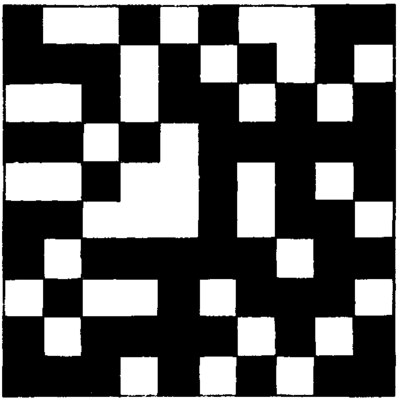


**Fig. 3.7** Density chart

#### 3.4.1.1 Visual inspection

This is the simplest approach to inspection, it is limited in that only surface evidence of deterioration can be detected. It only reveals internal problems, such as reinforcement corrosion, when the problem is in an advanced state.

#### 3.4.1.2 Coring

If there is question as to the interior deterioration of the concrete then it is possible to take cores from the structure. Standard electric core drilling equipment is not readily adaptable for underwater use. However, standard core drilling frames have been modified for underwater use by replacing electric power with hydraulic or pneumatic power drills. Drill base plates are usually bolted to the structures. Rather than have the operator apply thrust to the bit as is the usual case in above-water operation, pressure-regulated rams or mechanical levers are used to apply this force. A diver-operated coring apparatus with the capacity to drill either horizontal or vertical cores to a depth of 1.2 m has been used in the USA. The core diameters were up to 150 mm. In the undertaking of such an operation care must be taken not to sever prestressed tendons or highly stressed reinforcement. Coring and drilling can be used to detect chloride levels and chemical degeneration of the concrete matrix and to assess concrete strength.

#### 3.4.1.3 Other sampling techniques

Pneumatic or hydraulic powered saws and chipping guns can also be used to take concrete samples from underwater structures. Samples of reinforcing steel are usually taken by cutting the bar with a torch, although a pneumatic or hydraulic powered saw with an abrasive or diamond blade can be used.

### 3.4.1.4 Non-destructive testing (NDT)

There are two facets to non-destructive testing of concrete: assessment of concrete quality and assessment of the integrity of the reinforcement. Concrete quality can be assessed using the following techniques.

*Impact hammer (Schmit hammer)*

Impact techniques make an assessment of the surface hardness of the concrete. They are ideal for making comparisons between areas of concrete, i.e. assessing limits of deterioration or identifying sub-standard batches of concrete.[22]

*Ultrasonic (Pundit)*

Ultrasonic devices work on the basis of recording the velocity of a sound pulse through the concrete. The recorded pulse velocity will be a function of concrete quality and it can also give information concerning crack depth.[22]

*Cover-meters*

These instruments are used to determine the depth of the reinforcement from the concrete surface. This technique is useful in identifying areas of low cover that may be prone to early depassivation by chloride ingress or carbonation.[23]

*Electro-potential mapping*

As reinforcement depassivates and corrodes, its electro-potential becomes more negative. Mapping techniques enable a surface scan of these potentials to be made and areas liable to reinforcement corrosion identified.[23]

*Resistivity*

This technique complements the measurement of potential. Devices such as the Wenner probe record the resistivity of the concrete. However, the location of reinforcement, geometry of section and moisture content will all affect the recorded value.[23]

*Gamma ray backscatter*

This technique has been used to determine areas of concrete with poor compaction and high voidage. It is also possible, therefore, to identify areas of degenerated concrete. The instrument measures the backscatter from a radioactive source held against the surface of the concrete, the penetration depth being up to 100 mm.[22]

### 3.4.2 Inspection techniques above water

Concrete in the zone above low water will generally be subjected to aggressive conditions. In this atmospheric zone a wide range of conventional inspection techniques can be applied. The principal problems of carrying out inspection in this area will be time, access and the constantly varying moisture content of the concrete. The inter-tidal zone of a structure is often covered with a high density of marine growth.

Visual inspection is difficult in these areas owing to the marine growth and often considerable cleaning will be required. Non-destructive testing techniques such as resistivity and potential mapping will be affected by the changing moisture content of the concrete, and consequently absolute values indicated by these methods will have no meaning, although comparative readings may be of value. The condition of patch repairs can be assessed by potential mapping over the repair zone, areas of low potential indicating regions where corrosion of the reinforcement may be occurring. Impact hammers can be used for local comparisons of strength; however, these techniques assess surface strength and poor cleaning or surface deterioration will lead to erroneous results.

### 3.4.3 Underwater inspection

Working under water provides the inspector with a host of operational problems. To compound these problems, many of the NDT techniques will not work in water. References 21 and 24 provide excellent information on underwater inspection.

#### 3.4.3.1 Visual inspection

Visible inspection is, currently, the most widely used method of inspecting underwater concrete. There are three techniques that can be used for a visual inspection: human eye, still photography and closed-circuit television (CCTV).

*Human eye*

Using a diver is often the first choice in the inspection of concrete. On many sites underwater visibility is often drastically reduced. If the loss of visibility is caused by turbidity then external light sources, such as torches, will be of no use. Under these conditions the diver may have to resort to survey by feel. When objects are viewed under water their size and colour will become distorted, therefore divers should be provided with a good lighting system and be encouraged to measure rather than estimate. Reporting and recording of eyeball surveys can be carried out by written notes by the diver or by a commentary to the surface on a diver communication system. The identification and classification of defects

**Fig. 3.8** Sea and Sea camera with built-in flash unit

should be accurate. If an untrained inspector is used to assess the condition of concrete, problems in the reporting of the survey will be encountered; therefore training is essential.

### *Still photography*

The great advantage of still photography is the permanent record that can be used to back up visual surveys. Still photography can be carried out by a diver or from a ROV. There are many camera systems that are available for underwater inspection. For smaller projects relatively cheap 35 mm underwater cameras are available; these are ideal for preliminary inspection work (see Figure 3.8) . For more advanced surveys there are camera systems that record a stereo photogrammetric model of the object. This advanced type of system would be used in the detailed inspection of damaged areas to permit assessment prior to repair. There are three important considerations to be considered in underwater photography: light, location and scale.

The use of natural light is normally impractical owing to the low light levels under water, and additionally the object will suffer colour distortion at depth. Flash units are, therefore, generally used for underwater photography. Water generally is turbid, and the particles of material causing this turbidity will cause a reflection from the light source, causing a 'snow storm' effect on the photograph. To avoid this, light sources must be beamed on to the object at a wide angle, about 45° (see Figure 3.9).

Location and scale are defined by identification markers that can be included in the field of view. Concrete is often relatively featureless and therefore the inclusion of 'ident' markers is essential. The identification

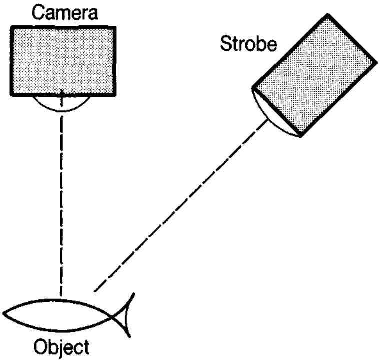



**Fig. 3.9** Camera positioning

markers should include the following information: *where*—the contract name and the location on structure; *when*—the date and time of the survey, and shot number; *who*—the name of the inspector.

### *CCTV*

In recent years there has been a considerable development of videographic systems. CCTV systems have the great advantage that they can provide a video tape record of the inspection. Various camera units are available, including black and white, colour and low light. Generally the black and white systems give better resolution but colour is preferable for concrete inspection. Diver-operated video systems are usually simple fixed-focus, automatic-aperture cameras. Some of the models are small enough to be attached to the diver's helmet, thus giving the inspector greater freedom of movement (see Figure 3.10). The mode of operation with a diver-held camera involves two operators, the diver and the topside video controller. The diver communicates with the topside operator via a communication system, thus enabling the topside operator to record a full visual and spoken commentary from the diver. The topside operator can also interact with the diver and supplement information that is being given.

ROV systems have similar capabilities to that of the diver; in this case the operator will fly the vehicle and provide a technical commentary on the inspection. With the use of ROVs, definition of location and scale may be a problem. For large-scale works, acoustic position-fixing systems can be used; for small-scale work, such as dock work, it will be necessary to set up a reference system using tapes and shot lines.

### *Tactile inspection*

Tactile inspections are inspections by touch. These are usually conducted under conditions of extremely poor visibility, such as might be expected in a heavy silted river or a settling pond. They may also be required where the

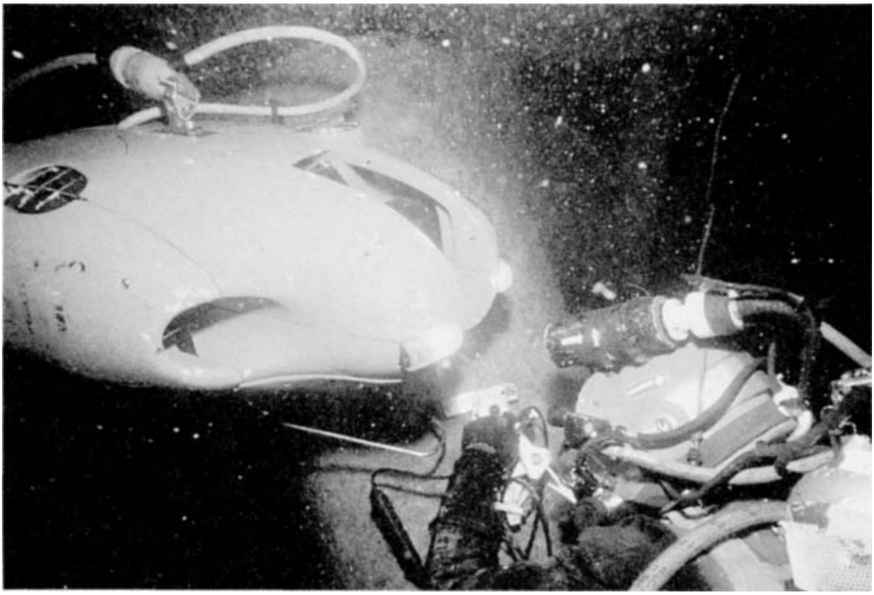


**Fig. 3.10** A diver carrying out inspection work using a helmet-mounted CCTV camera. (Reproduced with permission from Rockwater Ltd.)

element to be inspected is totally or partially buried by silt. The diver merely runs his hands along the structural element to find a defect. The defect is usually quantified relative to apparent size of the inspector's hand and arm lengths. Once a defect is found, the diver may have difficulty in properly describing the position of the defect so that it may be located and repaired at a future date.

### 3.4.3.2 Underwater non-destructive testing (NDT)

There are two main objectives with NDT techniques, assessing concrete quality and the condition of the reinforcement. Water is an excellent conductor of sound and electricity and therefore NDT methods based on ultrasound and electro-potential become severely restricted. There has been some development of NDT methods for underwater concrete inspection, but many of these techniques are still at the development stage. Recent development of non-destructive testing for underwater applications, including acoustic pulse-echo, impact echo, sonar, radar, laser mapping and underwater acoustic profilers, can be found in Ref. 25.

*Ultrasonic techniques*

This technique can be used for assessing the quality of the cover concrete by positioning the two transducer heads on the surface of the concrete. Taylor Woodrow Research Laboratories (TWRL)[26] conducted laboratory tests which showed that the outer 40 mm of the concrete would be scanned,

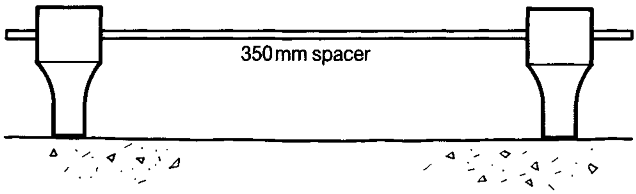



**Fig. 3.11** 82 kHz transducers

provided that the transducer spacing was greater than 350 mm. This ensures that the pulse velocity is not affected by the reinforcement in the structure. To avoid the need for excessive surface preparation, 82 kHz exponential-shaped transducer heads have been used[23] (see Figure 3.11). Above water this technique can detect the presence of cracks; under water these cracks fill with water and transmit ultrasound with very little distortion of the pulse velocity.

Development work has been carried out on an acoustic technique for measuring crack geometry. This method utilizes a focused acoustic surface wave and has the ability to determine crack width and depth.[27,28]

*Potential mapping*

The highly conductive nature of the water will effectively 'short out' the corrosion cell. The implication of this is that all potentials measured will be very close to the lowest electro-potential in the structure. With detailed mapping it may be possible to detect very small changes in potential, possibly under 1 mV, but for large-scale monitoring this would be impractical. An alternative technique is to detect cracks by current density measurement. These techniques utilize a rotating reference cell; as the cell rotates, it detects changes in the potential field,[29,30] as shown in Figure 3.12.

Although the results of the tests were encouraging, problems were found with stray currents and corrosion of the diver's equipment was a significant factor.

One application of potential measurement is to make a preliminary survey of a unit by measuring its overall potential. By lowering a reference cell into the water and measuring the potential, the general state of the reinforcement may be assessed (see Figure 3.13).

The potential of sound steel in concrete will be around −200 mV (Ag/AgCl) and freely corroding steel has a potential of around −600 mV (Ag/AgCl). Consequently, if the potential of a unit is depressed to a lower value, then it will be a *prima facie* case to indicate active corrosion in the reinforcement cage.

*Impact hammers and cover meters*

These units have been adapted for underwater use. The principal problem associated with their use is surface preparation of the concrete.

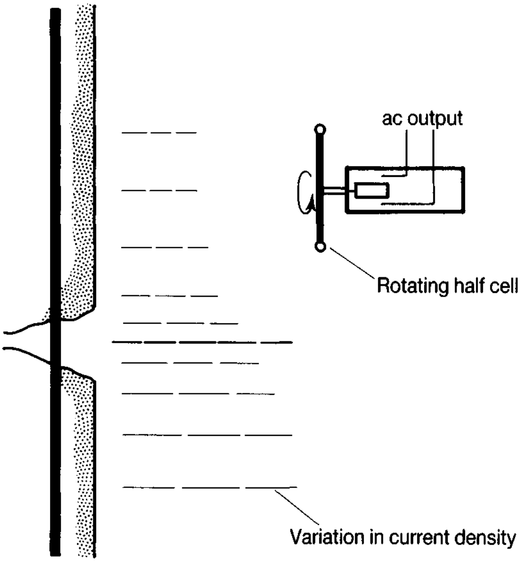



**Fig. 3.12** Current density method of crack detection

# 3.5 The inspection and reporting process

## 3.5.1 The classification of defects

The most effective method of underwater inspection of concrete is visual inspection. The surface of a concrete structure will exhibit a host of blemishes, which can be divided into two categories: construction ble-

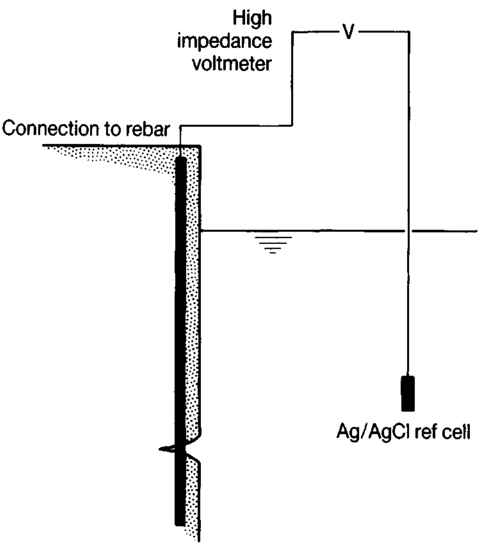



**Fig. 3.13** Crack detection by potential measurement

mishes and in-service blemishes. During the construction process, many irregularities in surface texture will occur; some of these will be of no structural importance whereas others will possibly provide planes of weakness for future attrition. During the inspection process the inspector will constantly view these construction blemishes, and consequently they must be classified for reference. Untrained inspectors have been known to spend considerable time and effort in surveying construction joints and describing them as structural cracks. In-service defects will be caused by a wide variety of phenomena and these defects must be inspected thoroughly as this type of defect may be indicative of a threat to the integrity of the structure. Therefore, it can be seen that a careful description of blemishes and defects is required. Towards this aim for a comprehensive description of concrete, several publications have been produced to aid the inspector. The *Guide for Making a Condition Survey of Concrete in Service*[31] is an example of a description of concrete defects aimed at the above-water inspector. The Cement and Concrete Association has published a wealth of material on the construction process and defects and blemishes in concrete.[24] To aid the underwater inspection in the classification of blemishes, two publications have been produced by the Department of Energy.[33,34] These documents provide an excellent set of annotated photographs and explanatory notes.

A classification system should provide the inspector with the following guidelines:

- description of the feature
- the possible cause
- details to be reported in the inspection
- a classification of the feature.

Table 3.3 sets out examples of a typical form of classification.

### 3.5.2 Inspection planning and procedure

To ensure cost-effective inspection, a structured approach to the work must be formulated. The output from the inspection will be an assessment of the condition of the structure, which will lead to the development of plans for remedial works and/or a downgrading of the facility. To achieve a coherent approach to inspection, an inspection schedule should be prepared. This document should define a rationale for the inspection and be seen as an integral part of the operational procedure for the installation. The inspection schedule would, typically, include the following guidelines:

- the frequency of inspection
- a description of the structure, highlighting potential problem areas

**Table 3.3** Typical form of classification

| Feature | Description | Possible cause | Details to report |
|---|---|---|---|
| Cracking (structural) | Irregular tearing | Overload<br>Corrosion<br>Shrinkage<br>Impact | Direction<br>Width<br>Depth |
| Pattern cracking | Map of fine cracks | ASR | Area<br>Width of cracks<br>Sign of exuded material |
| Exudation | White salts emanating from cracks or joints | Poorly constructed joint<br>Action of water in a crack | Severity<br>Colour<br>Area |
| Erosion | Loss of material<br>Possible softening of concrete | Chemical attack<br>Physical abrasion | Condition of concrete<br>Depth/area of deterioration |
| Staining | Rust-coloured runs in the area of cracks or joints | Corrosion of reinforcement or tie wire | Colour<br>Area<br>Point of origin |
| Spalling | Cusp-like loss of section | Impact damage<br>Expansion of reinforcement | Area<br>Depth<br>Signs of exposed reinforcement |
| Scouring | Loss of bed material from the base of the structure | Hydraulic erosion of bed material | Depth<br>Width<br>Extent of undercut |
| Honeycombing | Voids between coarse aggregate | Poor compaction | Area<br>Severity<br>Signs of staining |
| Construction joints | Linear irregularity | Line between pours of concrete<br>Shutter lines | Dimensions<br>Direction<br>Signs of exudation of staining |

- a specification for inspection techniques
- the method of reporting the inspection.

An inspection operation will either take place as a routine part of a maintenance programme or will be in response to an unforeseen circumstance. The frequency of routine inspections will be a function of the vulnerability and residual service life of the structure. A guideline as to this frequency is given in Table 3.4.

It may be necessary to carry out inspections of a structure in response to

**Table 3.4** Frequency of inspection (after Shah[35])

| Installation | Residual service life (years) | Inspection frequency (years) |
|---|---|---|
| Docks and harbour installations | 25 | 5 |
| | 16–25 | 4 |
| | 9–15 | 3 |
| | 4– 8 | 2 |
| | 4 | 1 |
| Ferry terminals | | 1 |

an unforeseen situation. Ship collision or material dropped from an installation are prime examples of situations requiring immediate investigation. Cracking or settlement of the structure noted above the water line will also indicate the possibility of underwater damage. Breakwater and river structures may require inspection after storms.

If underwater repairs have been carried out, the repair sites should be inspected at least once in the year after the repair.

The aim of the preliminary inspection is to make an overall reconnaissance of the major components of the structure. In the case of dock and harbour installations, this work can be carried out using a small ROV or a Scuba dive team. Offshore work requires a far greater degree of sophistication owing to the scale of the work. A constant enemy in the inspection of large structures is operator boredom, and consequently video backup to the survey can be of great value.

Once the preliminary survey has been completed, the findings can be compared with the inspection database for the structure and areas selected for detailed inspection (see Table 3.5).

The method of detailed inspection will be selected in the light of the type of problem exposed. Generally the underwater inspector must gain as much evidence as possible for the maintenance planners. Photography and written reports will invariably form the cornerstone of the findings, but

**Table 3.5** Actions following preliminary inspection

| Findings from the preliminary inspection | Action |
|---|---|
| Minor damage that does not expose reinforcement | No action |
| Softening of concrete | Detailed inspection: record loss in section and sample material and, if severe, assess stability |
| Rust staining or exposed reinforcement | Detailed inspection: assess loss in steel section, analyse, repair and monitor |
| Cracking | Detailed inspection of crack geometry, analyse, repair and monitor |
| Undercutting of foundations | Detailed inspection: plan remedial works |

specific NDT techniques may be called upon to clarify various situations. Cover meters, for example, can be used to determine areas in which the reinforcement will be at risk.

## 3.5.3 Inspection reporting and recording

### 3.5.3.1 Reporting procedures

The object of inspection is to assess deterioration of the structure, and it is therefore pointless to record every construction blemish and minor construction defect. The recording of observations of major defects should be carried out in a systematic order. A sequence of operations may be defined in an Inspection Schedule; a typical approach would be as follows:

- note location with reference to known features
- note the time, date and depth of inspection site
- set up a reference tape system related to known features
- mark the position of the defect with a paintstick
- note the density and type of fouling
- note signs of damage
- note any surface deposits, e.g. efflorescence
- note signs of surface staining
- note cracking: width, depth, orientation
- check if the concrete is sound: tap the surface to detect hollows behind sheet spalls, check for loose concrete
- record the extent of concrete loss
- note any apparent causes of the damage
- attempt to classify the defect.

### 3.5.3.2 Recording procedures

The aim of a good recording system should be to form a database that will permit cross-referencing of recorded data. The key to setting up such a system is to establish a location reference system for the structure, which will enable the operator to accurately monitor deterioration.

## 3.5.4 Inspection qualifications

The industry's awareness of the need to inspect structures has led to the need for more divers and ROV operators with a sound knowledge of

**Table 3.6** Units in the CSWIP certification scheme

| Grade | Content |
|---|---|
| 3.1U (Diver) | Visual inspection of steel |
| | Cathodic potential measurements |
| | Ultrasonic digital thickness measurement |
| | Underwater photography |
| | Use of CCTV with oral commentary |
| 3.1UC (Diver) | As above, with concrete endorsement |
| 3.2U (Diver) | Ultrasonic A-scan |
| | MPI assessment of steel |
| | Weld toe profiling |
| 3.2UC (Diver) | As above, with concrete endorsement |
| 3.3U | ROV inspection |
| | 3.1U plus concrete |
| 3.4U | Underwater Inspection Controller |

concrete inspection. An increasing number of civil engineers have now gained HSE diving certificates and can use their technical knowledge in inspection work. As stressed in this chapter, there are intrinsic dangers in using untrained operatives for the inspection of concrete. The most notable problem is the description and classification of defects and blemishes; in many cases this is a good example of a little knowledge being very dangerous.

The offshore industry insists on its inspectors being trained in various aspects of underwater inspection. In the UK the principal set of inspection qualifications are controlled by the Certification Scheme for Weldment Inspection Personnel (CSWIP).[36] This scheme was originally aimed at weld inspection; with the use of steel structures offshore, modules were developed to cover underwater inspection of welds. The scheme has now been widened to cover underwater inspection of concrete. The CSWIP certification scheme now consists of the units listed in Table 3.6.

It is a common practice in the USA to require that divers be certified. Organizations that certify divers in the USA include the Professional Association of Diving Instructors, the National Association of Underwater Instructors and the American Diving Contractor Association.[37]

## References

1 Collard, M.J. (1986) Inspection and maintenance of North Sea concrete platforms, in *Maritime and Offshore Structure Maintenance*. Thomas Telford, London, p. 51.

2 Day, I.J. and May, B. (1986) Certification philosophy in respect of the Offshore Regulations 1974, in *Maritime and Offshore Structure Maintenance*. Thomas Telford, London, p. 311.

3 Bakun, G.B. (1986) Cost effective approach to pier inspection and maintenance, in *Maritime and Offshore Structure Maintenance*. Thomas Telford, London, p. 301.

4 Venables, R.K., Sharp, J.V. and Leeming, M.B. (1986) Concrete in the oceans—a coordinated offshore research programme, presented at *Marine*

*Concrete 1986. International Conference of Concrete in the Marine Environment, London, 1986.*

5 Erikson, K., Mattson, L., Klungenberg, H. and Wannerburg, C. (1985) Swedish experience of degradation of structure due to the marine environment, presented at *Permanent International Association of Navigation Congresses, 26th International Navigation Congress, Brussels, 1985.*

6 Biczok, I. (1967) *Concrete Corrosion and Concrete Protection.* Akadémiai Kiadó, Budapest.

7 Pullar-Strecker, P. (1987) *Corrosion Damage Concrete—Assessment and Repair.* Construction Industry Research Laboratories (CIRIA). Butterworth, Guildford.

8 American Concrete Institute (1991) *Guide to Durable Concrete, ACI 201.2R. ACI Manual of Concrete Practice, Part 1.* American Concrete Institute, Detroit, MI.

9 Holland, T.C., Husbands, T.B., Buck, A.D. and Wong, G.S. (1980) *Concrete Deterioration in Spillway Warm-Water Chute, Raystown Dam, Pennsylvania.* Miscellaneous Paper SL-80-19, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

10 Rendell, F. (1991) Macro cell corrosion of reinforcement in concrete, in *UK Corrosion 91*, Vol. 3. Eds. C.L. Page, K.W.J. Treadaway and P.B. Bramforth. Elsevier, Manchester, pp. 167–177.

11 Beaton, J.L. and Spellman, D.L. (1967) Corrosion of steel in continuously submerged reinforced concrete piling, *US Fed. Highway Res. Rec.*, No. 204, 11–21.

12 Wilkins, N.J.M. and Lawrence, P.F. (1980) *Concrete in the Oceans.* Report No. 6, Cement and Concrete Association, Slough.

13 Wilkins, N.J.M. and Sharp, J.A. (1986) The corrosion of steel reinforcement immersed in sea water, presented at *Marine Concrete 1986. International Conference of Concrete in the Marine Environment, London.*

14 American Concrete Institute (1991) *Corrosion of Metals in Concrete.* ACI 222R-89, American Concrete Institute, Detroit, MI.

15 Health and Safety Executive (1981) *A Guide to the Diving at Work Regulations 1981.* H&SE Booklet HS(R)8, H.M. Stationery Office, London.

16 US Navy (1975) *Diving Manual, Volumes I and II.* NAVSEA 0994-LP-001-9010 and NAVSEA 0994-LP-001-9020, Office of the Navy, Washington DC.

17 Larne, R. and Whistler, R. (1984) *Commercial Diving Manual.* David and Charles, Totnes.

18 Chew, M.J. (1986) Effective use of divers and remotely operated vehicles offshore, in *Maritime and Offshore Structure Maintenance.* Thomas Telford, London, p. 147.

19 Marine Technology Society (1984) *Operational Guidelines for Remotely Operated Vehicles.* Marine Technology Society, Washington, DC.

20 Keeney, C.A. (1987) *Procedures and Devices for Underwater Cleaning of Civil Works Structures.* Technical Report REMR-CS-8, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

21 Lamberton, H.C., Sainz, A.J., Crawford, R.A., Ogletree, W.B. and Gunn, J.E. (1981) *Underwater Inspection and Repair of Bridge Substructures.* National Cooperative Highway Research Program (NCHRP) Synthesis of Highway Practice No. 88, Transportation Research Board, Washington, DC.

22 Cement and Concrete Association (1980) *Concrete in the Oceans. Marine Durability Survey of the Tongue Sands Tower.* Technical Report No. 5, Cement and Concrete Association, Slough.

23 Department of Energy (1985) *Concrete in the Oceans. Development of Inspection Techniques for Reinforced Concrete.* Technical Report No. 10, OTH 84 205, H.M. Stationery Office, London.

24 Brackett, R.L., Nordell, W.J. and Rail, R.D. (1982) *Underwater Inspection of Waterfront Facilities: Inspection Requirements Analysis and Non-Destructive Testing Technique Assessment.* TN No. N-1624, Naval Civil Engineering Laboratory, Port Hueneme, CA.
25 Thornton, H.T. and Alexander, A.M. (1987) *Development of Nondestructive Testing Systems for In-Situ Evaluation of Concrete Structures.* Technical Report REMR-CS-10, US Army Engineer Waterways Experiment Station, Vicksburg, MS.
26 Taylor Woodrow Research Laboratories (TWRL) (1976) *Concrete in the Oceans. Survey of Existing Reinforced Concrete Marine Structures—Tongue Sands Tower. Preliminary Study of the 'PUNDIT' as a Surface Survey Instrument.* Report No. P4/12. Cement and Construction Acc., Slough.
27 Poole, M.J., Crook, M.J. and Anderson, M.R. (1982) The ultrasonic torch, *Journal of the Society of Underwater Technology.*, **2** (1), 18–23.
28 de Lange, J.I., Verhulst, K. and Breeuwer, R. (1985) Underwater inspection of concrete structures, in *Behaviour of Offshore Structures*, Battjes, JA (ed), 513–522. Elsevier, Amsterdam.
29 Eggen, T.G., Vennesland, O. and Bardal, E. (1983) Detection of failures on concrete structures in sea water, in *Norwegian Maritime Research*, No. 2, Norges Tekniske Hogskole, Norway.
30 Bournat, J.P., Stankoff, A. and Auboiroux, M. (1980) Inspection of concrete platforms: crack detection by current density measurement, presented at *Offshore Technology Conference, Houston, Texas, May 1980*, OTC Paper 3765.
31 American Concrete Institute (1991) *Guide for Making Condition Surveys of Concrete in Service. ACI 201.2R ACI Manual of Concrete Preactice, Part 1.* American Concrete Institute, New York.
32 Monks, W. (1981) *The Control of Blemishes in Concrete. Appearance Matters 3.* Cement and Concrete Association, Slough.
33 Department of Energy (1984) *The Classification and Identification of Typical Blemishes on the Surface of Concrete Underwater.* OTH-84-206, H.M. Stationery Office, London.
34 Department of Energy (1987) *Typical Blemishes on the Surface of Concrete Underwater, a Supplementary Classification and Identification.* OTH-87-26, H.M. Stationery Office, London.
35 Shah, V.R. (1986) Outline of guidelines for inspection and maintenance of marine facilities, in *Maritime and Offshore Structure Maintenance.* Thomas Telford, London, p. 321.
36 CSWIP (1987) *Requirements for the Certification of Underwater (Diver) Inspectors.* Document No. CSWIP-DIV-7-87, Certification Scheme for Weldment Inspection Personnel, Cambridge.
37 Popovics, S. and McDonald, W.E. (1989) *Inspection of Engineering Condition of Underwater Concrete Structures.* Technical Report, REMR-CS-9, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

*Further reading*

38 Allen, W.L. (1989) Underwater inspections, *Conc. Int. Des. Construct.*, **11**, No. 9, 37–40.
39 Guthrie, R.L. (1991) Underwater bridge inspection. A practical approach, *Publ. Works*, **122**, No. 1, 46–48.

# 4 Methods of placing concrete underwater

F Rendell

## 4.1 Introduction

The range of application of underwater concreting is wide: early examples of the placing of concrete beneath water level were generally in the form of bagwork, a natural extension of the use of masonry. Wharves, docks and piers all found use for the application of underwater concreting and many of the older structures were constructed using mass concrete, thus giving a versatile, economic alternative to masonry construction. Many of these older applications of underwater concreting suffered from deterioration due to the poor quality of the placed concrete and to lack of control in placement. With advances in placing techniques and materials it is now possible to use underwater concreting techniques to produce high-quality reinforced structural members. In the Concrete Society report on underwater concreting,[1] in 1973 it was considered unwise to base a design on strengths greater than 22.5 MPa (N/mm$^2$); with the introduction of non-dispersible concrete it is now possible to work with concretes of strengths up to 55 MPa (Ref. 2). Another innovation in underwater concreting has been the development of flexible formwork, which has allowed the construction of relatively thin concrete scour aprons and has provided a very versatile technique for repair work.[3]

In many respects there is no difference between the placing of concrete above and below water. Concrete sets by the action of hydration of the cement and does not rely on the drying out of the concrete matrix; consequently, the immersion of the wet concrete should theoretically, have no effect on the setting of the mix. In practice, the concrete quality will suffer a deterioration in quality when placed under water. The principal cause of this lowering of quality is the washing out of cement and fines and by segregation of the concrete. Agitation of the wet concrete, by the action of the surrounding water, will cause the washout of constituent elements of the mix; this degraded concrete becomes mixed with silt and will form a layer of aggregate and laitance on the surface of the pour. Segregation of the concrete mix will generally manifest itself when the concrete is dropped through water. The overall quality of the work will ultimately be affected by factors such as the degree of control over the placing operation.

When devising a technique of placing concrete under water, the quality of the concrete must be ensured. The first consideration must be to avoid segregation of the mix; the simplest remedy for this is not to drop the concrete through the water. Techniques such a tremie and pumping are based on the principle of piping the concrete through the water and thus eliminating mixing with the water and segregation. Over the past few years, additives have been developed to prevent the segregation of concrete, which has given greater freedom in the development of placing techniques.

The concrete/water interface is a zone of contaminated, weak material and therefore of no structural value. If this weakened material becomes entrained into the heart of the pour, weakening of the structure will occur. The aim, therefore, when placing concrete under water should be to minimize the surface area of concrete in direct contact with the water, and to avoid agitation of the exposed surface. Consequently, the ideal method of placing concrete would be to inject the fresh concrete into the heart of the concrete already deposited, which would leave the weakened outer layer as a skin over the pour. The concrete must be of sufficient workability to allow the pressure of the added concrete to cause displacement of the placed concrete. It is not practical, or advisable, to use compaction equipment, such as vibrators, under water. The agitation caused by compaction and screeding may cause the inclusion of water and layers of detritus into the body of the mix. It is, therefore, of importance to use a concrete that has a workability to allow self-compaction and, if possible, to be self-levelling.

Great care must be taken when working under conditions where high water velocities are found, as cement and fines will be removed from the surface of the pour. To minimize this scouring of the concrete, it may be necessary to divert flows or work at slack water (see Figure 4.1).

When the operator places concrete under water he rarely has a knowledge of what is going on at the site of the pour. This lack of control over the underwater concreting causes many problems. A notable problem caused by this 'blind working' is ensuring compaction around reinforcement and the forming of construction joints. If reinforcement is used it must be of a simple nature, with wide spacing and large bars being a preferable design detail. The wide spacing will allow tremie pipes or similar to be introduced into the centre of the unit and the large diameter bars will be less inclined to impede the flow of concrete, thus avoiding forming voids around the steel. Horizontal construction joints are potential planes of weakness and will be particularly vulnerable to erosion or abrasion. The weakened layer, on the top of a pour, may be of considerable thickness and

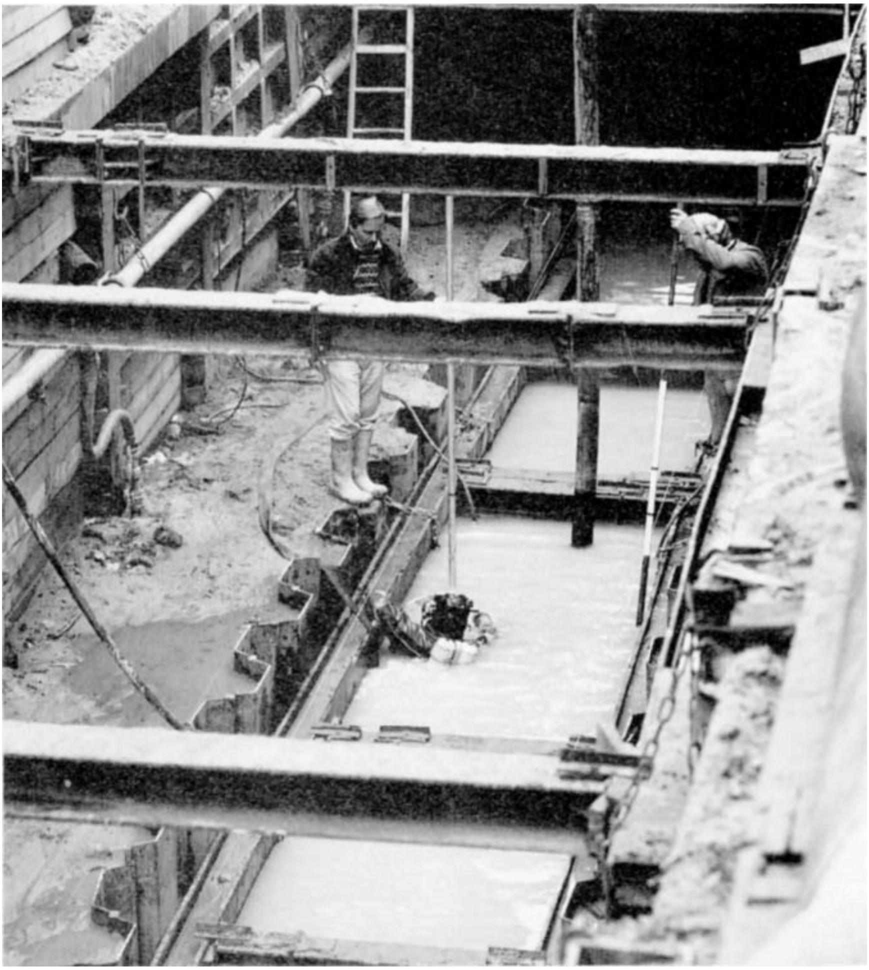


**Fig. 4.1** Placing concrete in a cofferdam; note the arduous working conditions of this type of operation. (Reproduced with permission from Gewatech)

this layer of laitance and segregated aggregate may well become mixed with silt deposited from the surrounding water. This layer of detritus has to be removed before subsequent pours are made. This is normally a diver operation and consequently expensive. Therefore, construction joints are generally avoided and placement methods aim to achieve a one-pass pouring of concrete.

## 4.2 Selection of method

### 4.2.1 General

The method of placing concrete will be governed to a great extent by the location and the volume of material to be placed. As indicated above, the

**Table 4.1** Techniques for underwater construction and possible applications

| Technique | Possible application |
|---|---|
| Contact between concrete and water controlled by injecting concrete into the mass of previously placed fluid concrete, e.g. tremie, pumping, hydrovalve | Suitable for mass concrete and simple massive reinforced concrete sections |
| All contact between fresh concrete and water prevented, e.g. bagwork, pumping into collapsed flexible forms | Used for underpinning, scour aprons, filling voids at the surface of a structure, pipe support and protection |
| Introduction of admixtures and/or additions to the concrete to give enhanced cohesive and self-levelling properties | Suitable for reinforced members, this sections and encasement of structural members |
| Grouting, with or without admixtures | Preplaced aggregate concrete, annulus grouting, fissue void and joint grouting |

critical factors in concrete placing underwater are avoiding segregation and minimizing the contact between the surface of the concrete and the water. The techniques used for underwater construction, and their possible applications, can be summarized in four groups,[4] as shown in Table 4.1

With the advent of non-dispersible concrete, the application of underwater concreting has taken on a new dimension. The limitations on quality assurance inherent in the more traditional techniques has now been drastically reduced. The characteristic strength achieved by the new concrete types allows the production of high-quality structural members. The self-levelling and compaction of these mixes imply that the use of reinforcement becomes a more viable proposition. Recent research and development work has demonstrated the possibilities now open to the engineer for underwater construction and repair.[5,6]

### 4.2.2 Bagwork

Bagwork is probably one of the oldest and simplest techniques of placing concrete under water. The method is a natural extension of the use of masonry but has the flexibility to enable the building blocks to be moulded together, thus achieving good bonding. This construction method is labour intensive but has great adaptability in its application. Early underwater engineering projects used bagwork to construct large elements of temporary and permanent works. Although recent developments in underwater concreting have made this technique appear to be an anachronism, it should not be discounted as such. A common application of bagwork is the construction of retaining walls to act as formwork to mass concrete pours. Another common application is the pell-mell placing of bagwork to form scour protection. For small-scale remedial works bagwork gives the engineer a low-cost, simple solution to many problems. Given the constraint of permanence, the technique will not compare with modern placement methods, but for temporary works or short-term solutions the method should be considered.

The bags used in the process are generally made of hessian and should be of a man-handleable size. The concrete for use in bagwork should be of a rich mix, with a maximum aggregate size of about 12 mm. The bag should be half filled with a very plastic concrete. The bags will be malleable, thus enabling them to interlock. The cement paste will seep through the weave of the hessian and aid bonding between the bags. Overfilling the bags will detract from the malleability required in the placing operation.

The bags will be placed by hand in the type of bonding one would use in brickwork. As the bags are placed they are trodden down and spiked together with short lengths of reinforcement. Figures 4.2 and 4.3 show a typical methods of placing bagwork.

### 4.2.3 Skips and toggle bags

The skip and toggle bag methods of placing concrete both give an intermittent rate of delivery. Consequently, these methods are both prone to surface mixing and silt inclusion at the surface of the placed concrete. These methods of placing are ideally suited to the placing of relatively small volumes of concrete. The accurate placing of each delivery also makes this technique well suited to the pouring of thin slabs where screeding may be required.

An important feature in the design of the skip is ability of the concrete to be deposited with the minimum of disturbance. The skip should have straight sides with no taper at the bottom. The doors should swing clear of the bottom opening to allow a free discharge of the concrete. The fall of

**Fig. 4.2** Bagwork

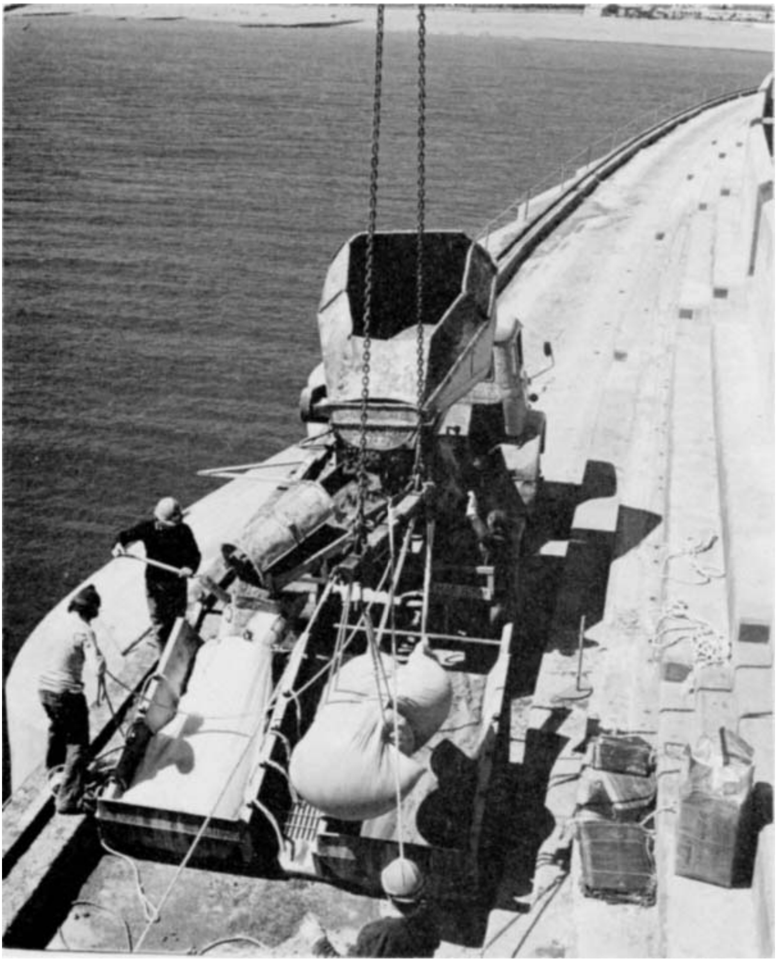


**Fig. 4.3** Concrete placement using prefilled concrete bags. (Reproduced with the permission of the Concrete Society)

the concrete, and thus mixing with the water, should be minimized; therefore, the skip is so constructed that it cannot be opened until its weight is resting on the bottom. A skirt is often built around the bottom of the skip to confine the concrete as it is placed. Figure 4.4 shows a typical arrangement of a bottom-opening skip designed for underwater work. The top surface of the concrete in the skip is liable to mixing during the placing operation, and consequently two loose canvas flaps are used to cover the top of the skip.

When placing the concrete, the skip is completely filled and the top covers are closed. The skip is then slowly lowered into the water, avoiding disturbance of the concrete. When the skip is on the bed the doors are released and the skip is slowly raised. Ideally the concrete is placed into the

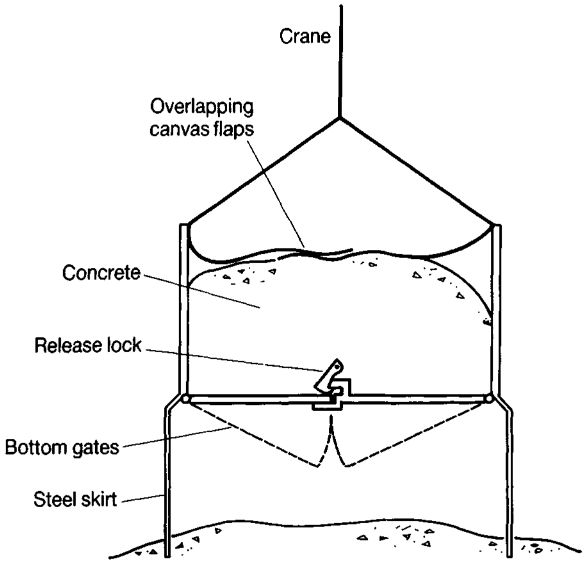



**Fig. 4.4** Arrangement of a bottom-opening skip

body of previously placed concrete, thus achieving a degree of injection, and the pour advanced laterally across the site.

The toggle bag is a reusable bottom-opening canvas bag. The top end of the bag is sealed and the bottom is tied with chain or rope and secured by a toggle. The placing technique is similar to that of the bottom-opening skip. The technique is best suited to small pours and the placing of discrete quantities of concrete into exact locations, e.g. for plugging voids.

### 4.2.4 Tremie and hydrovalve

The tremie is one of the traditional techniques used for the placing of concrete underwater.[7–11] The basic objectives of the tremie are two-fold:

- the concrete is piped to the site of placement and therefore segregation and infiltration by surrounding water are avoided
- it is inherent in the technique that concrete is placed into the heart of the pour, and therefore the contact surface with the water is minimized.

The tremie is suited to the placement of large volumes of concrete from a fixed platform.

The tremie consists of a steel pipe mounted vertically in the water. To the top of the pipe is fixed a hopper, to receive the concrete; it also acts as a reservoir for the supply of fresh concrete. The tremie tube should be watertight and have a smooth bore. The integrity of the seals at joints in this pipe is essential, as it ensures that no water is entrained into the fresh concrete during the pouring operation. It is vital that a continuous flow of concrete is achieved and therefore the discharging pipe should be of sufficient diameter to ensure that blockages do not occur. As a guide to the selection of pipe size it is common practice to use 150 mm diameter pipes for aggregate sizes of up to 20 mm. Pipe diameters of 200 mm are used for aggregate sizes of up to 40 mm. In view of the requirement to achieve a steady flow of concrete, it is important that if the length of the tremie pipe has to be varied, pipe joints can be made quickly and maintain a good seal. The flow rate through the tremie is controlled by the raising and lowering of the tremie, this operation normally being carried out by cranage or block and tackle. Figure 4.5 shows the basic arrangement of the tremie. It is of upmost importance that the end of the tremie is always immersed in the heart of the freshly placed concrete. If the seal is broken then fresh concrete will mix with the water and a discontinuity will occur in the pour.

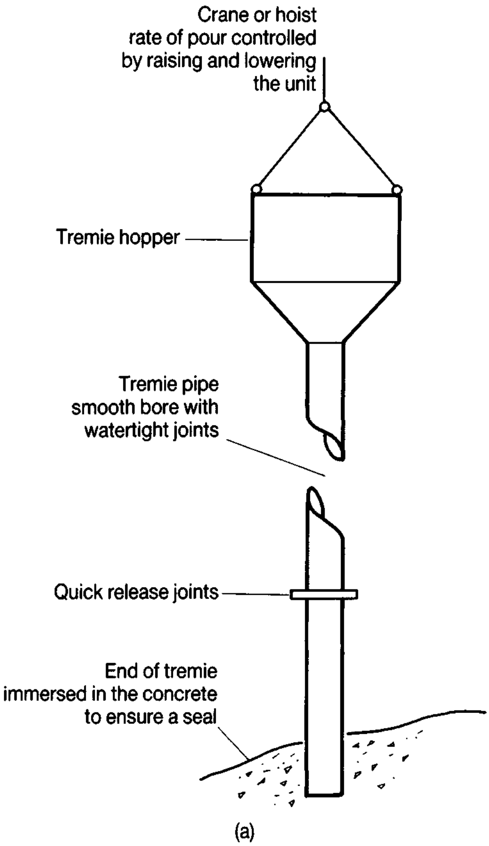



**Fig. 4.5a** Arrangement of the tremie unit

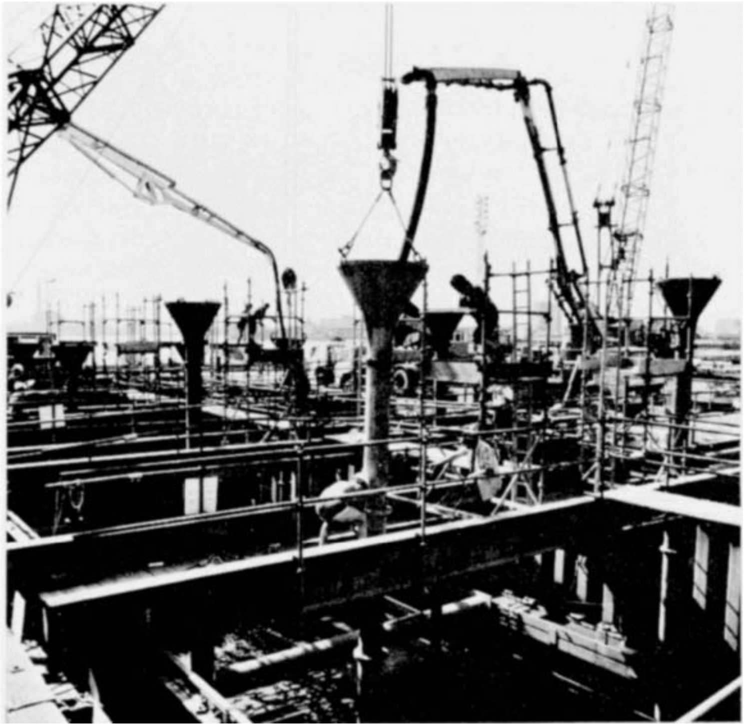


**Fig. 4.5b** The surface set up for a multiple tremie pour. (Reproduced with the permission of the Concrete Society)

The technique used for the initial filling, or charging, of the tremie is an important feature in the operation. As with other stages of underwater concreting, the basic objective is to keep the concrete separated from the water. If the first flush of concrete is allowed to fall freely into the tremie pipe, the concrete will segregate and entrain water. To control the condition on the initial charging there are two options open to the operator. The first is to seal the bottom end of the tremie tube until charging is complete. This seal can be released once charging is complete, and the pour can then commence. It has been found that the design and operation of an effective bottom flap or seal present problems. These operational problems are compounded by the initial upthrust on the tremie pipe before filling.

The second, more usual, way of charging a tremie is using a travelling plug. The function of the plug is to form a barrier between the water and the concrete. This plug is placed in the top of the tremie pipe prior to placing the concrete. The sealing plug is displaced down the length of the tube by the weight of concrete and the plug is then either buried by the mass of the concrete or recovered. If the inclusion of the plug in the concrete is not acceptable then the recovery is best achieved by the use of a floatable plug. Attempts to extricate a buried plug may well cause more damage than leaving it in place. Travelling plugs have been made from foamed plastic, inflatable balls, cement bags and sacks. The more recent trend is towards the use of exfoliated vermiculite. As a general rule, the length of the plug should be at least 1.5 times the diameter of the pipe.

In cases where the concrete is being placed on to a surface that is liable to scour, granular fill or similar, it is advisable to position a steel plate at the base of the pipe.[12] This plate will diffuse the shock of the first flush of the concrete. If unchecked, this initial charge of concrete could scour a pit and cause the loss of the concrete seal at the bottom of the pipe.

#### 4.2.4.1 The method of placing concrete

The operation of a tremie takes the following form. Once the tremie is charged, the pipe is lifted a small distance to allow space for the plug to escape. The tube is immediately lowered to place the point of discharge into the core of the placed concrete. The flow rate through the tremie can then be varied by lowering and raising the pipe. The rate of supply of concrete to the hopper should be as regular as possible. Sufficient head must be provided by the weight of concrete in the hopper to overcome pipe friction and back-pressure from the placed concrete. If the flow becomes too rapid, often as a result of raising the tremie, there is a danger of air becoming entrained into the placed concrete.

If the tremie has to be shortened during the operation, care must be taken not to break the seal. The concrete level in the tremie is allowed to fall to just below the level of the joint, and the section is then removed. Rejoined sections of pipe should be watertight, as a poor pipe seal will allow water to ingress into the concrete.

As can be seen from the discussion of placing, the position of the end of the tremie in relation to the surface of the poured concrete is of great importance. Techniques for monitoring this level will be discussed later in the chapter.

The lateral movement of a tremie within a pour is not an advisable operation as it may lead to the loss of seal and possible washout from the core of the placed concrete. The conventional way of placing concrete over a large area is to use a network of tremies. Each pipe will serve an area of approximately 20–30 m$^2$. This effective area of placement is dependent on the properties of the concrete being used. In the situation where a number of tremies are being used, the face of the placed concrete should be advanced across the site by the 'bringing on line' of tremies as the pour advances. To avoid the inclusion of water in the pour, it is advisable to charge the tremies before the face of placed concrete engulfs the end of the pipe. Maintenance of the desirable pouring conditions on each tremie will require the construction team to plan a sequence of placing, the required objective being the continuous supply of concrete to each tremie.

#### 4.2.4.2 Broken seal

The most important consideration in a tremie operation is the maintenance of the concrete seal at the bottom of the pipe. Once this seal is broken, two things happen: first, fresh concrete is placed on top of the previous pour. This is highly undesirable as the water/concrete interface is of dubious quality and further placement of concrete on top of this layer would create an inclusion of weak material into the pour. The second problem is the loss of charge in the tremie, when the entire operation has to be restarted.

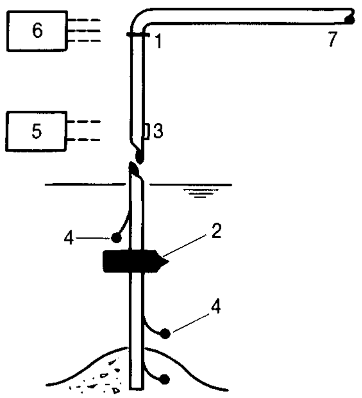
If the concrete seal at the base of the tremie is broken, the pour should be immediately halted. Reforming the seal presents the operator with a recharging operation that will be time consuming and may endanger the integrity of the pour. To allow the pour to continue, the end of the tremie must be re-positioned in the mass of the poured concrete, thus preventing the inclusion of the outer skin of concrete into the heart of the pour. The recharging operation must not allow water to be injected into the poured concrete, therefore recharging must be carried out using an end-plate on the tremie. In the event of a loss of seal the tremie is removed from the water and a removable end-plate fitted. The tube is then re-positioned, kentelidge being used to submerge the pipe and to drive it into the mass of the poured concrete. Once the tremie is in position and charged, the end-plate is removed and recovered; the pour then recommences.

The great danger with a break in the seal is damage caused to the mass of poured concrete. At the time of pouring it is impossible to know the extent of damage that has been caused by the interruption. The use of divers to inspect and remove wet concrete would, most likely, result in more damage being caused to the pour. If a major problem is suspected then the best policy is to abandon the work and allow the concrete to set. The extent of damage can then be ascertained and, after suitable remedial work and surface preparation, the pour may be continued.

A recent development in the tremie is the utilization of a hydraulically operated valve situated at the lower end of the pipe, as described by Yamaguchi *et al.*[13] Figure 4.6 shows the layout of the adapted tremie system. The crushing valve is hydraulically operated and linked to a series of level sensors. A tremie system such as this can minimize the possibility of the loss of seal and will allow for discontinuities in the supply of concrete to the tremie hopper.

#### 4.2.4.3 Description of the hydrovalve

The hydrovalve was developed in Rotterdam in 1972[14,15] with the aim of eliminating the problems associated with flow control in the classical tremie system. The hydrovalve (see Figure 4.7) consists of a vertically mounted pipe with a flexible, collapsible, inner tube. At the top of the down tube is a hopper which is maintained at a constant level; in some cases the hopper can be mounted on rails to facilitate lateral movement of the unit. In the




| | Part name | Specification |
|---|---|---|
| 1 | Tremie pipe | 150 mm dia |
| 2 | Crushing valve | 150 mm dia |
| 3 | Pressure sensor | Strain gauge |
| 4 | Level sensor | Tilt switch |
| 5 | Control unit | Hydraulic |
| 6 | Lamp indicating panel | |
| 7 | Flexible delivery hose | |

**Fig. 4.6** Tremie with crushing valve

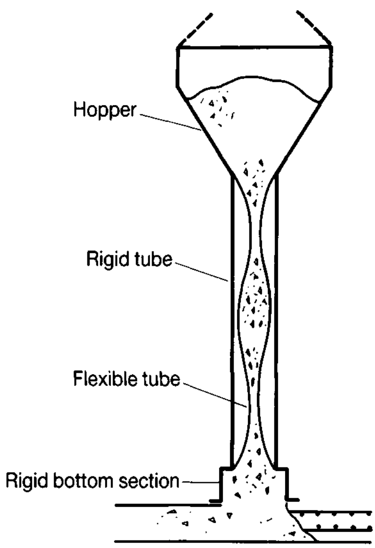



**Fig. 4.7** The Hydrovalve

initial condition the flexible tube is collapsed owing to the pressure of the water. As concrete is charged into the top of the tube, the weight of the concrete overcomes the pipe friction and moves down the pipe. The concrete dilates the flexible tube as it moves downward, the tube collapsing after the plug of concrete has passed. This peristaltic motion ensures that the concrete is isolated from the water and does not allow free fall, and thus segregation, of the concrete.

The lower sector of the hydrovalve consists of a rigid section. After the initial charging of the system the unit is lifted to the level of the top surface of the finished work. The tube is then traversed at this level over the area of the pour.

A method very similar to the hydrovalve method was developed by Kajima Construction Company in Japan in 1976, known as Kajima's Double Tube (KDT) tremie method.[16] The KDT also uses a collapsible tube, but it is enclosed in a steel pipe that has many slits. This steel pipe makes it possible to withdraw, move horizontally and re-set the KDT. When the last slug of concrete goes down by its own weight, the water pressure from the water entering through the slits in the outer tube causes the tube to flatten. Therefore, no water enters the tube, nor does washing of the concrete occur. Field tests have shown this method to be reliable and inexpensive.

### 4.2.5 Pumping

With the conventional tremie method, cranage often imposed logistical problems on the operation. The tremie pipe had to be raised and lowered to control the pour and concrete placed in the hopper by skip. With the advent of concrete pumping, the underwater placement of concrete was partially freed from the constraint of cranage. The principal advantage of pumping is that concrete can be delivered to the pour site quickly and virtually continuously. Using static pipe runs, concrete can be pumped over distances of up to 1000 m and the hydraulic boom on most mobile units permits great versatility in placing. The placement of concrete can be carried out using either a tremie or direct pumping to the underwater site. If a tremie system is being used then the concrete pump offers an ideal method of delivery to the hopper; the steady flow and the ability to reposition the delivery pipe, with a hydraulic boom, provide the operator with a highly efficient method of operating a series of tremies. Another great advantage with pumping is that concrete can be delivered under water without the need for gravity delivery. This attribute of a pump system eliminates problems such as segregation, but the same philosophy of placement must be adhered to as with conventional tremie methods.

Although problems of segregation may have been eliminated, it is still of upmost importance to minimize the water/concrete interface, i.e. freshly placed concrete should be placed in the core of the placed mass. Pipe blockage is a distinct possibility with pumping and it is therefore advisable to have a standby pump on site. When pumping to depths greater than about 35 m, it is necessary to incorporate a non-return valve in the pumping line.

The delivery rate of a pump is directly related to the head loss in the pipe line. Thus, the longer the pipe run the higher is the required pumping head; this is limited by the pump unit and the seals on the delivery pipe.

When a fluid is conveyed along a pipe there will be a pressure loss due to friction, and the loss in pressure will be a function of the translational velocity of the fluid:

$$\Delta P \propto Q/A$$

where $\Delta P$ = pressure loss, $Q$ = flow rate and $A$ = pipe area.

Typically a concrete pump can deliver about 90 $m^3/h$ but its operation is limited by the pressure. A top limit on delivery pressure is of the order of 300 psi. The head loss due to friction for an ordinary pumped concrete is of the order of 0.06 $kg/cm^2/m$. Figure 4.8 shows the pressure distributions that can be expected in a pump system.

In the situation where the concrete delivery point is under water, the back-pressure from the surcharging water and concrete will both have a large influence on the pumping pressures and, consequently, the delivery rate. Typically, if the pipe is embedded into the concrete by 100–200 mm, the working pressure of the pump can be kept at an acceptable value.

When pumping to an open form, a trailing pipe can be used to deliver the concrete. Figure 4.9 shows a configuration that can be used for such a mode of placing concrete.

A problem that may be encountered in pumping concrete vertically downward is that the pour may free fall. This is often the result of the flow through the vertical section exceeding the pumping rate; this free falling of the concrete will allow the concrete to segregate. This situation can be

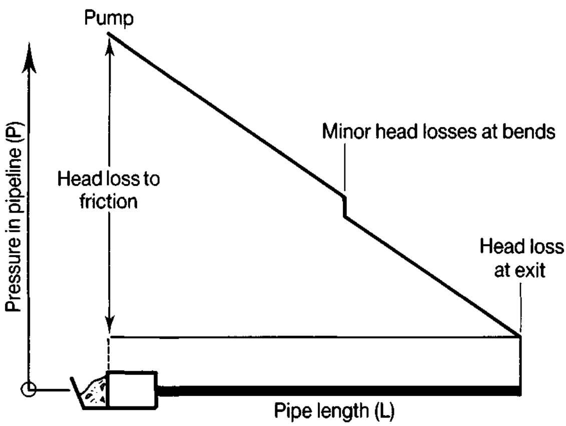



**Fig. 4.8** Pressure distribution in a pipeline

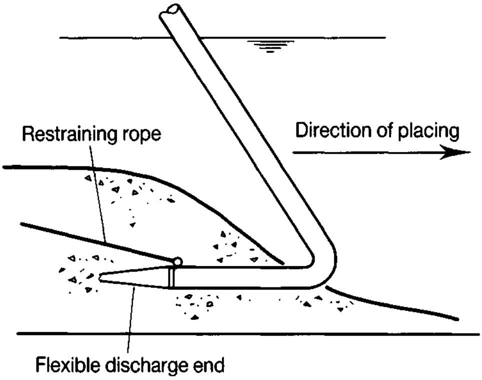



**Fig. 4.9** Pumping with a trailing pipe

eliminated by using a plug as in the case of tremie work. A sponge plug can be inserted into the top of the delivery line before pumping; this will support the concrete as it is forced down the pipe and will be expelled at the outlet. In cases of very deep pours, it may be necessary to introduce pairs of 90° bends into the line at approximately 15–20 m intervals.

### 4.2.6 Placing non-dispersible concrete

Non-dispersible concrete is generally used to minimize segregation; it also has properties of flowability, non-bleeding and self-levelling. Consequently, the material is therefore very resilient to poor placing techniques. The bonding of the wet concrete by polymers prevents segregation even in situations where the concrete is dropped through water. The flowability makes the concrete ideally suited for use with reinforcement or in situations where the concrete must compact around encased sections.

The greatest impact of non-dispersible concretes has been the facility to produce high-quality underwater concretes. The cohesive nature of the concrete allows free fall through water of up to a few metres.

Using conventional techniques and concretes, the concrete/water interface is prone to erosion and mixing and therefore the deposition of fresh concrete on top of a previously poured batch will cause an inclusion of weak material. In the case of non-dispersible concretes, the viscous nature of the mix will prevent these surface problems, therefore making placement on to previous pours possible. This quality of the concrete makes the use of skip-poured concrete a more viable alternative.

The pumping of a non-dispersible concrete is probably the easiest method of placement; the qualities of the material will allow a relatively fast delivery rate as the maintenance of the 'concrete seal' is no longer vital to the operation. The viscous nature of the concrete has the effect of producing higher friction losses in the pipework associated with a pump system. In general the pressure losses when using non-dispersible concrete can be up to 50% higher than those with ordinary concrete.

An interesting set of experiments were carried out by Kawai *et al.*[17] to assess the qualities of non-dispersible concretes. The work was carried out prior to the construction of a pump room for a dry dock. Conventional techniques would have involved cofferdam construction, but underwater concreting was used to shorten the working time. As a high-quality product was required, the placement techniques and concrete quality had to be assessed before adoption of the construction method.

### 4.2.6.1 Self-levelling

To investigate the flowability or self-levelling properties, the concrete was placed into a 20 m long water-filled trough. Concrete was placed in the former, from one end, in stages and allowed to settle to an angle of repose. Figure 4.10 shows the changes in surface gradient in the placement. On completion of the test the compressive strength of the concrete was assessed by coring. Figure 4.11 shows the variation of compressive strength over the 20 m over the pour length.

This test demonstrated that non-dispersible concrete has a good self-levelling quality and there is very little loss of strength associated with the flow of the concrete.

### 4.2.6.2 Embedment of reinforcement

A second set of tests were carried out to examine the placement of concrete into a reinforced member. A 5 m long trough with a transparent

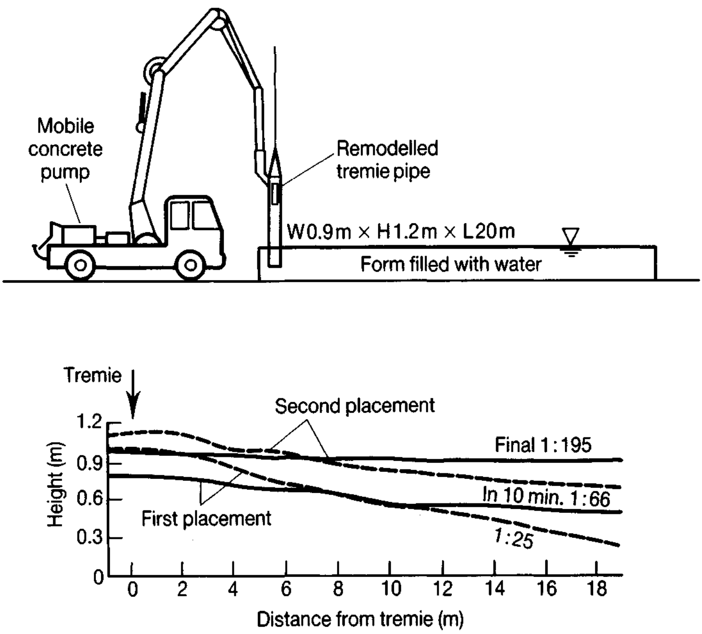



**Fig. 4.10** Changes of gradient of concrete surface (after Ref. 17)

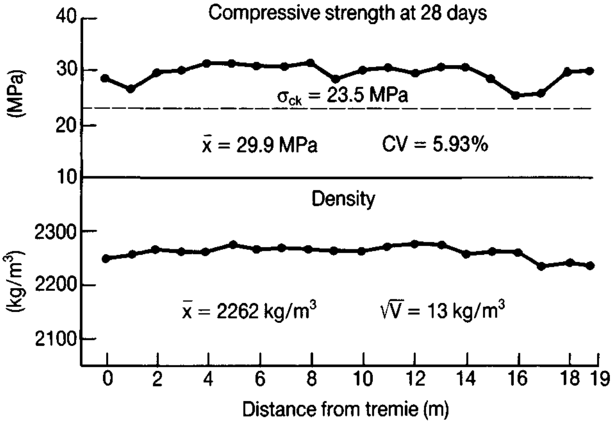



**Fig. 4.11** Changes of compressive strength and density of concrete

side was constructed. A cage of reinforcement was fabricated, consisting of a three-dimensional grid with a spacing of 150 mm. Concrete was pumped, via a tremie, into one end of the form. The placing was carried out in two stages and the rates of repose are shown in Figure 4.12. During the pour the flow of concrete around the steel was observed. It was noted that the concrete flowed into the reinforcement cage by slow degrees, and it was postulated that this slow and steady flow was due to the high viscosity of the mix. Coring of the specimen confirmed the good bonding of the steel and the absence of voids within the cage. As with the first set of tests, it was found that there was no significant change in strength along the length of the bay. Subsequent tests also showed that the strength and homogeneity of the mix were independent of the placing rate.

### 4.2.7 Preplaced aggregates

The preplaced aggregate method, also known as the grouted aggregate concrete method, is suitable for use in the inter-tidal range and for underwater work.[18] The technique is particularly applicable in conditions where there is limited access to the work, in situations where high water velocities exist or where the site is subjected to wave action, which would normally prohibit the use of conventional placement methods. The use of preplaced aggregates in repairs is discussed in Section 5.4.4.

The preplaced aggregate method, as the name implies, consists of filling a form with aggregate and then injecting a grout to fill the voids. The aggregate that is used in this process is typically 40 mm or larger; if aggregates smaller than 20 mm are used then the injected grout tends to bridge the interstices, thereby impeding grout flow. In certain cases a fine aggregate may be required. In these situations a sand-free grout is used, but this is an undesirable application as severe bleed lens form under the aggregate.[19]

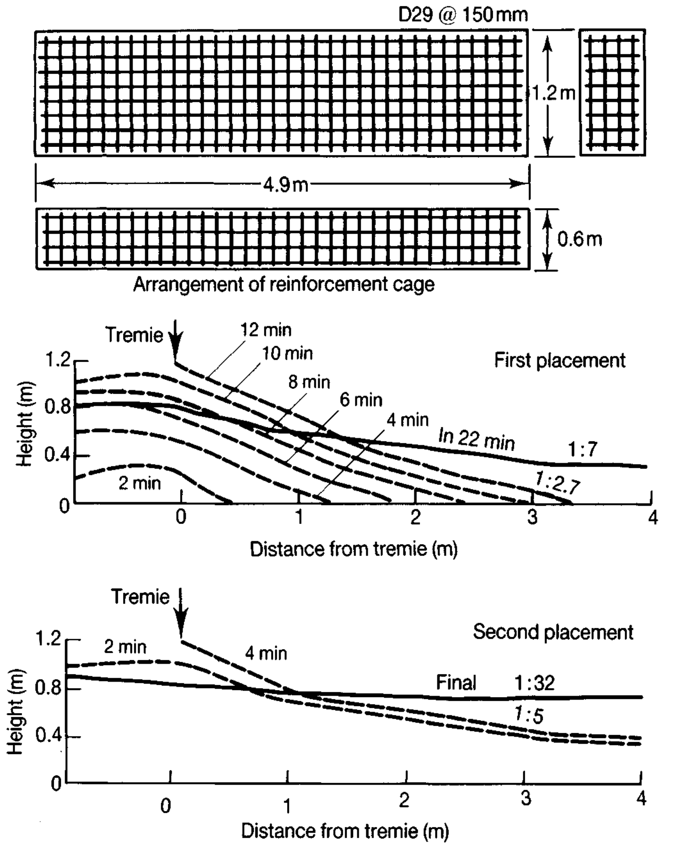



**Fig. 4.12** Changes in gradient of concrete with reinforcement (after Ref. 17)

The grout that is used in this process normally consists of ordinary Portland cement and well graded sand. The flow of the grout around the aggregate is essential, therefore plasticizing admixtures are normally recommended. Pulverized fuel ash is also used to improve the flow of the grout.

The final product of the process is a concrete of high aggregate/cement ratio with point contact between the aggregate. This point contact ensures a low restrained shrinkage that is about 50–70% of that of conventional concrete; the reduction in voids ensures a dense concrete, and consequently a good strength can be attained.[20] However, if a high-strength concrete is to be produced, care must be taken in the design of the grout as microcracking may occur owing to the settlement of the grout.

The first stage of the placing process consists of filling the form with aggregate; it should be compacted by vibration, or by rodding, prior to the injection of the grout. The formwork should be designed to ensure that there is no leakage of grout during the injection process; flexible formwork is often used in this application. The grout is injected into the bottom of the form, the water and air being displaced upward by the rising grout front. Injection is continued until a free washout of grout is emitted from the top of the pour. If a small volume is being poured, e.g. a repair to a pile, a tapered vent will be left at the top of the formwork. In the case of large open areas, it is normal to overfill the form and disregard the surface layer as washout will be inevitable in this upper zone. The injection is achieved by pumping the grout through vertically mounted pipes. These pipes are normally 20 mm in diameter and placed at 1.5 m centres; the exact dimensions will be determined by the nature of the aggregate. As the grout is pumped into the form the injection pipes are slowly raised. Vibration of the shutter will aid the release of trapped air and water. Injection of grout into small units can be achieved by pumping into the bottom of the form. The preplaced aggregate method can be used to produce reinforced concrete but a danger, in the marine environment, is the inclusion of chlorides in the completed work.

Epoxy resin can also be used with preplaced aggregate. Underwater grades of epoxy resin can be used in place of grout. The cost of epoxy resin is several times the price of cement grout but it does have some unique properties. The resin is of low viscosity, even at low temperatures, therefore placement can be easier. The low viscosity also means that smaller aggregates can be used. The setting time of a resin can be controlled by the formulation of the mix, thereby adding versatility to the technique.

Rip rap aprons can be bonded using a variant of the prepacked aggregate technique. Large units of stone will permit the inter-flow of fine aggregate concrete, therefore it is possible to pour non-dispersible concrete into a stone matrix without the need for injection.

### 4.2.8 Placing concrete into flexible formwork

Flexible formwork is, basically, a tailored fabric bag that can be easily assembled to fit a desirable shape (see Figure 5.3). The placing of the fabric form can be simply carried out by divers. The concrete is normally pumped into the form but can be placed by tremie techniques. When pumping concrete into a flexible former it should be pumped in at the lowest point and continued until concrete flows freely from a top vent. If a non-dispersible concrete is used, a strengthened fabric will have to be used. This is because the higher 'flowable' nature of this type of concrete will exert greater loadings on to the forms.

The mattress type of form is generally filled with a sand cement grout. When placing a double-skinned form, allowance must be made for shrinkage due to the inflation of the mattress on filling. The fabric of the form is permeable and allows for the release of excess water, thereby aiding the early stiffening of the mix. This property also improves the long-term strength and durability of the mattress. The forms are filled by pumping, injection must work bottom to top and a constant supply rate is essential.

## 4.3 Control and monitoring

The control on the quality of the concrete is of upmost importance in an underwater concreting operation. Blockages in a pipe run or a tremie will possibly have disastrous consequences, therefore all concrete arriving on site should be tested prior to use. The rate of delivery is also a critical factor in the placing operation and standby plant is essential to allow for breakdowns. In the case of a large pour using tremies, a careful plan should be drawn up to ensure the correct and steady delivery of concrete to the hoppers.

The underwater monitoring of the placement process is often impossible. Divers can be used to monitor the progress of a pour but all too often the underwater visibility is such that the diver would be ineffectual as a controller. Generally the work is often carried out blind and inspected on completion. The diver's role tends to be one of investigation, 'what went wrong?' being a common brief. After placement, diver surveys should be carried out to assess the quantity of laitence to be removed.

In tremie work the control of the depth of immersion of the tremie pipe in the concrete is a vital factor. Figure 4.13 shows two methods that can be used for the monitoring of levels. The technique of sounding is the more traditional method but the more advanced system using flote switches can be incorporated into a fully automated delivery system.

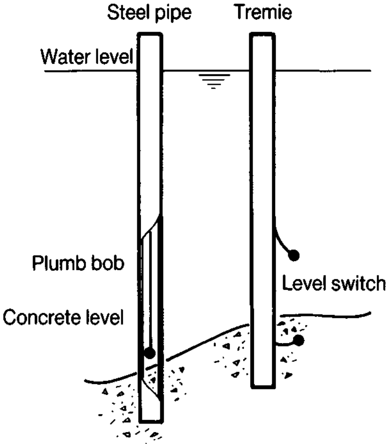



**Fig. 4.13** Configuration of level monitoring methods

# References


1 Concrete Society (1973) *Underwater Concreting*. Technical Report, Concrete Society, London.
2 Sogo, S. (1986) Properties of underwater concrete containing segregation controlling admixture, in *Marine Concrete '86*. Concrete Society, London.
3 Rathmell, B.T. (1984) Fabric formwork for coastal protection, harbours and marine structures, *Concrete*, **18** (1), 11–13.
4 Concrete Society (1986) *Underwater Concreting*. Technical Report No. 35, Concrete Society, London.
5 Khayat, K.H. (1991) *Underwater Repair of Concrete Damaged by Abrasion–Erosion*. Technical Report, REMR-CS-37, US Army Engineer Waterways Experiment Station, Vicksburg, MS.
6 Gerwick, B.C. (1988) *Review of the State of the Art for Underwater Repair Using Abrasion-Resistant Concrete*. Technical Report, REMR-CS-19, US Army Engineer Waterways Experiment Station, Vicksburg, MS.
7 American Concrete Institute Committee 304 (1973) *Recommended Practice for Measuring, Mixing, Transporting and Placing Concrete*. ACI 304-73, Chapter 2, American Concrete Institute, Detroit, Michigan.
8 Holland, T.C., Gerwick, B.J. and Turner, J.R. (1982) Tremie concrete placement, Wolf Creek Dam cutoff wall. *Concr. Int. Des. Construct.*, **4**, No. 10, 26–34.
9 Anon (1968) Concrete down the spout. *Concrete Construct.*, **13**, No. 7, 247–250.
10 American Concrete Institute (1911) *Guide for Measuring, Mixing, Transporting, and Placing Concrete, ACI 304R-89, ACI Manual of Concrete Practice, Part 2*. American Concrete Institute, Detroit, MI.
11 Gerwick, B.C., Holland, T.C. and Komendant, G.T. (1981) *Tremie Concrete for Bridge Piers and Other Massive Underwater Placement*. Federal Highway Administration Report No. FHWA/RD-81/153, US Department of Transportation, Washington, DC.
12 Wilson, C.L. (1986) Diving into underwater concrete, in *Marine Concrete '86*, p. 269–277. Concrete Society, London.
13 Yamaguchi, M., Tsuchida, T. and Toyoizumi, H. (1986) Development of high-viscosity underwater concrete for maritime structures, in *Marine Concrete '86*, p. 235. Concrete Society, London.
14 Scheowert, L.C. and Hillen, H.F. (1972) Underwater transporting of concrete with the hydro-valve. *J. Am. Concr. Inst.*, **69**, 584–88.
15 Netherlands Committee for Concrete Research (CUR) (1973) Underwater concrete (Ligtenberg, FK). *Heron*, **19**, No. 3, 5–52.
16 Nakahara, Y., Ohtomo, T. and Yokota, S. (1976) *Development of New Method for Underwater Concreting—KDT Tremie Method*. Report No. 22, Kajima Institute of Technology, Kajima.
17 Kawai, T., Takegawa, K. and Okumura, T. (1986) Dock construction work using newly developed high quality underwater concrete, in *Marine Concrete '86*, 257–267. Concrete Society, London.
18 Littlejohn, G.S. (1974) Grouted pre-placed aggregate concrete, presented at *Conference on Concrete in the Ground, 21–22 May 1974*.
19 Robery, P.C. (1982) Structural repairs. *Concrete*. **17**, No. 2.
20 Waddell, J.J. (1968) Preplaced aggregate concrete, in *Concrete Construction Handbook*, Chapter 38, pp. 38–41, McGraw-Hill, New York.

*Further reading*

21 Cooke, A.R. and Simpson, J. (1991) Underwater concrete construction of Phuket Port quay wall foundation. *Dock Harbour Auth.* **72**, No. 831.
22 Anon (1989) Underwater concreting—a study. *Indian Concr. J.*, **63**, No. 2.
23 Gerwick, B.C. (1988) *Review of the State of the Art for Underwater Repair Using Abrasion-Resistant Concrete.* Technical Report, RE-MR-CS-19, US Army Engineer Waterways Experimental Station, Vicksburg, MS.
24 Saucier, K.L. (1990) Placing specialised concretes. *Concr. Int. Des. Construct.*, **12**, No. 6.
25 Einert, M.P. (1990) Development of an underwater canal lining method, in *Irrigation and Drainage, Proceedings of the 1990 National Conference, Durango, July 1990.*
26 Wilson, F.C. (1988) Ball valve concrete pumps. *Conc. Construct.*, **33**, No. 2.

# 5 Underwater repair of concrete

A McLeish

## 5.1 Introduction

The civil engineering industry has extensive experience of repair concrete structures above water. Many of the techniques used can, often with minor modifications, be used under water. On the other hand, the materials used may not perform well when used under water. Cementitious materials may be affected by washout of cement, whilst resin-based materials may intermix with water and fail to bond to the structure being repaired.

Prior to designing or specifying underwater repairs, the advice of specialists should be sought. In many instances laboratory trials of repair methods and materials may be appropriate to ensure problems are identified prior to work beginning on site. This will help to avoid very costly failures.

This chapter looks at various aspects of underwater repair to concrete including:

- methods of gaining access to the repair site
- methods of preparation and breaking out of concrete and cutting of reinforcement
- properties of cementitious and resin-based repair materials
- different repair techniques for concrete
- repair of reinforcement and prestressing tendons.

## 5.2 Access to the repair site

### 5.2.1 General

Clearly repairs can be carried out more effectively if they can be undertaken in air rather than under water. This permits more thorough preparation, easier provision of formwork and a greater choice of materials and placement methods. It also enables repair specialists, rather than divers, to gain access and undertake the repairs.

In tidal areas consideration should be given to rapid repair methods and quick-setting materials that can be applied at low tide. Some very quick setting gunites (shotcrete) can even be applied between waves with a minimal loss of material.

Where the area to be repaired is always under water then two options exist: either the water can be excluded from the area or the repair can be carried out under water. A range of approaches to enable access to be gained to the repair site are discussed in the following sections and are illustrated in Figure 5.1.

### 5.2.2 Water-retaining barrier

For some applications it will be feasible to exclude water from the damaged area by use of a barrier in the form of sheet piling or earth bund. For practical reasons this will be limited to shallow water depths and localized areas of damage.

Once the water-retaining barrier is in place, extensive structural repairs can be carried out with relatively unrestricted access.

### 5.2.3 Atmospheric caisson

An atmospheric caisson consists of a prefabricated steel chamber which can be sealed against the structure to be repaired. The chamber has an access tube extending from the top which reaches above the water level and provides access for equipment and personnel. It can be equipped with all necessary services such as lighting, hydraulic power, communications and cutting and welding connections.

The caisson can be attached to the structure using anchor bolts or by strapping around structure where this can be accomplished. The seal to the structure can be provided by a rubber sealing ring (sometimes inflatable) fitted around the perimeter of the chamber.

Once the caisson has been attached and sealed to the structure, the water can be pumped out to allow repair work to be carried out in the dry.

The caisson should be as small as possible to minimize the effects of waves and currents, whilst still allowing room for the repair operations inside.

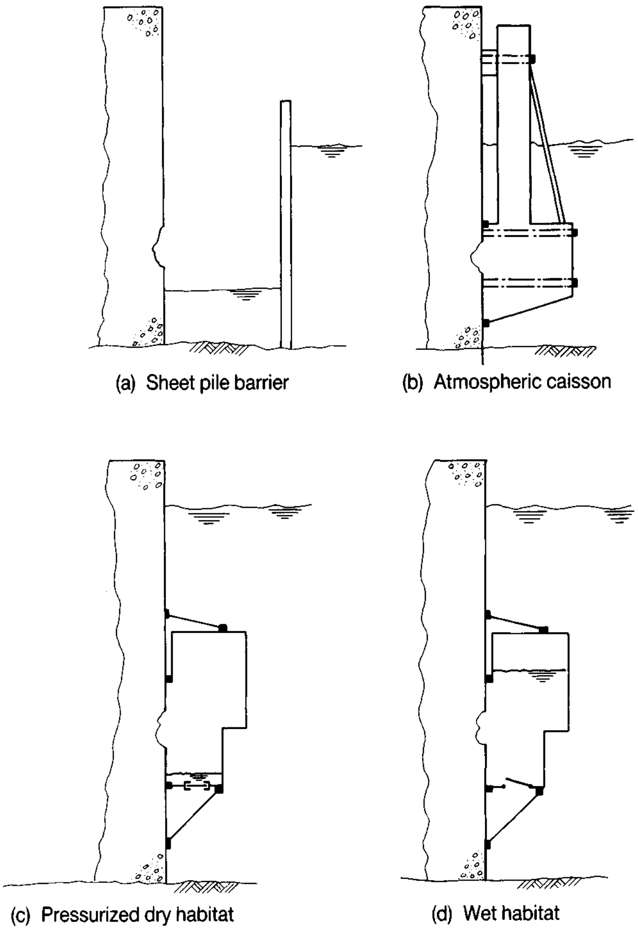



**Fig. 5.1** Access methods

The atmospheric caisson involves a high initial cost (compared with free-swimming divers, for example) and is not suitable for extensive repairs and for use on some forms of structure. Where a number of localized specialist repairs are required or in the splash zone where free swimming divers are at risk, then it represents a practical access method.

## 5.2.4 Pressurized dry habitat

This is similar to the atmospheric caisson except that diver access is provided from the bottom. The habitat is attached and sealed to the structure and then pressurized to displace water.

As there is no access tube to the surface, the pressurized dry habitat can be used at greater depths than the atmospheric caisson (generally down to 200 m water depth).

The pressurized dry habitat is the most complex access method considered and may often be precluded owing to its high cost. In situations where cost is less important than ensuring that a complicated repair operation can be carried out effectively, or where the damaged area is outside the depth range of a diver, then the pressurized dry habitat may offer a practical solution. These situations are generally limited to repairs on offshore oil installations.

### 5.2.5 Free-swimming diver

The diver-only approach is the method generally adopted for repairs in relatively shallow waters. The free-swimming diver can enable a more rapid repair to be carried out than would be the case if the damaged area was to be dewatered or access chambers constructed and installed.

The free-swimming diver can also undertake repairs to areas where access chambers could be precluded owing to the complexity of the structures. Piers and jetties with numerous members and pipe outfalls are examples where divers may offer the only method of access.

Divers can also move readily from one location to another. This flexibility enables simple repair tasks to be carried out at different positions on the structure without the need to move an access chamber.

The work that divers can be expected to undertake is limited by their protective clothing and the effect of waves and current. In the splash zone the free-swimming diver is at his most vulnerable and should be considered only for relatively minor operations of short duration. Below the splash zone, wave action will be considerably less significant and the diver could be expected to work for longer durations and undertake more complex tasks.

Trials carried out at Aberdeen[1] showed that free-swimming divers protected from waves or current could readily carry out a range of complex tasks, including the following:

- breaking out of concrete using a high-pressure water jet
- cutting of reinforcement using an oxy–arc burner
- installation and tying-in of a replacement reinforcement cage and additional prestressing tendons
- attachment and later stripping of prefabricated formwork
- control of concrete placement
- stressing of prestressing tendons using conventional stressing tools.

When designing the repair method it is essential to use large components that the diver, wearing thick gloves, is able to manipulate. The effectiveness of the diver, particularly in the splash zone, can be increased by the provision of a stable work platform which can provide support for the diver and a securing point for equipment, tools and lighting.

### 5.2.6 Wet habitat

Provision of a wet habitat for a diver is of great benefit for prolonged or complex work particularly where significant currents occur. The wet habitat consists of a chamber which can be attached to the structure. As for the pressurized dry habitat, all equipment and services can be provided in the chamber, although in the case of the wet habitat no attempt is made to pump down the water.

The prime function of the wet habitat is to provide a stable working platform, protected from adverse currents. In order to prevent wave forces on the chamber it must be installed below the level of the wave troughs.

## 5.3 Preparation of the concrete and reinforcement

### 5.3.1 General

An important first stage in repairing a damaged structure, whether above or below water, is to ascertain the extent of the damage. Concrete which is under water, or which is regularly submerged, is often covered by a layer of marine growth including seaweed and marine encrustation. In addition to removing this growth, it is also essential to break away all crushed or badly cracked concrete. Where reinforcement is badly distorted or severely corroded then this must also be cut away.

The removal of concrete and cutting of steel under water present a considerable number of problems. Where the operations can be carried out in air, for example at low tide or by using a cofferdam, then this will greatly facilitate the task.

There are many well established breaking and cutting techniques for use above water. In many cases, however, these require complete modification when used under water to ensure that they remain safe and practical. Electrical cutting equipment must be completely insulated. The expense involved in achieving this, and the resulting bulkiness of the equipment, make electrical tools unsuited to underwater work. Much underwater equipment, particularly for cleaning or breaking out concrete, relies on pneumatic or hydraulic power. This results in an extremely safe and rugged system.

A detailed survey of tools available for use underwater has been undertaken by the Underwater Engineering Group (UEG) of CIRIA.[2]

The choice of the cutting technique will be determined by the nature of the work; the thermic lance will cut through concrete and steel simultaneously, while high-pressure water jetting can be used to remove the concrete alone, leaving the reinforcement intact.

The following sections summarize some of the techniques for preparing damaged reinforced concrete under water prior to carrying out a repair.

## 5.3.2 Surface preparation

The amount and type of marine growth will depend on the depth below sea level and the age of the structure. Removing this growth is essential, both to be able to determine the extent of the damage and also to ensure a good bond between the repair material and the existing structure.

Hand-held or mechanical wire brushes, needle guns or scabbling tools are adequate for cleaning localized areas. A rotary wire brush cleaning tool, powered by hydraulic or pneumatic drive, is capable of removing soft growth such as seaweed and harder deposits such as barnacles and molluscs from steel or concrete. Underwater needle guns and percussion hammers can be used for removing the top surface of the concrete itself.

For larger areas a high-pressure water jet can be employed (see Section 5.3.3). Where hard deposits are to be removed, an abrasive slurry of powder may be introduced into the jet to give a more powerful cutting facility. Detergents can also be added to the jet to remove oil or other contaminants from the concrete surface.

During the period of breaking out the damaged concrete, cutting of reinforcement and the provision of formwork if required, the surface of the concrete may become contaminated with microscopic marine growth. This may develop over a period of only a few hours and can substantially reduce the bond between the repair material and the base concrete.

Before placing the repair material, the concrete surface should be thoroughly flushed with clean water to remove any bacteria (or microbiological growth). In some cases the use of a fungicide or alternative additive to the water may be required to remove all surface contamination.

A detailed survey of underwater cleaning procedures and devices that are appropriate to use on submerged portions of underwater structures has been undertaken by the US Army Corps of Engineers.[3] Reference 3 summarizes the application, advantages, disadvantages and operation of each type of equipment, along with recommendations for those tools best suited for specific conditions.

## 5.3.3 Concrete removal

For repair of reinforced or prestressed concrete structures it is often essential to cut away concrete without cutting damaging the embedded steel. The following techniques may be used to remove concrete, leaving the steel in place for subsequent cutting out or inclusion in the repaired section.

### 5.3.3.1 High-pressure water jetting

High-pressure water jetting is one of the most commonly used methods for breaking out concrete under water. The system consists of directing a fine, high-pressure stream of water at the surface of the concrete. A typical water pressure is 70 MPa although pressures up to 120 MPa may be used.

When used under water, the size of the jet is enlarged by the surrounding water, resulting in a larger cleaning area but possibly a reduced cutting rate compared with equivalent surface working. No depth limitations have been found as the pressure of the water jet is up to 100 times that of the surrounding water.

The reaction of the jet on the surrounding water generates a considerable force on the equipment. To balance this, an equal and opposite dummy jet is provided on underwater water jetting equipment.

Removal of concrete may be achieved in two ways, either by working from a free surface, causing the concrete to spall off, or by traversing the concrete creating an ever deepening groove. The pressure of water erodes the cement matrix although the aggregate itself is not cut but merely washed out.

In cases of high-strength concrete or where the reinforcement or other metal is to be cut, a steady stream of abrasive slurry or silica sand is introduced into the jet. The abrasive is drawn from an underwater hopper into the stream of water, greatly enhancing its cutting power. Where water alone is used the reinforcement is in no way damaged but is cleaned in preparation for coating or the application of repair material.

#### 5.3.3.2 Mechanical cutting

For small-scale or precision work, mechanical cutting using hydraulically powered diamond-tipped saws and drills is often used. If required, the method can be used to cut both concrete and steel, although the depth of cutting is limited by the diameter of the cutting disc. The use of disc cutters is generally limited to providing a sharp edge to a broken out area. This ensures a good interface between the repair material and the structure, and avoids a feather edge.

For shallow work (less than about 6 m), conventional pneumatic breakers can be used to remove concrete. Hydraulic powered breakers are similar to conventional pneumatic breakers but can be used at far greater depths. When using mechanical breakers, care must be taken to avoid causing micro-cracking in the sound concrete adjacent to the repair region.

#### 5.3.3.3 Splitting techniques

A number of holes are drilled into the concrete section along the line where the concrete is to be cracked. The direction and shape of the plane of cracking can be controlled by correct positioning and orientation of the predrilled holes. When an internal pressure is applied to the inside of the hole, the surrounding concrete fails in tension.

Two forms of hydraulic bursters are available, the plunger burster and the wedge burster. The plunger burster acts in a similar manner to a row of hydraulic jacks. The device consists of a hollow control body, often square in cross-section, with a series of plungers set into opposite faces. The bursters are inserted into a predrilled hole with a steel liner and pressurized to 125 MPa. This causes the plungers to extend and split the concrete. As the plungers extend in one direction only the direction of cracking can easily be controlled.

The wedge burster consists of a steel wedge which is forced under hydraulic pressure between two tapered steel liners. When inserted into a predrilled hole and pressurized the liners are forced against the inside face of the hole and lead to splitting of the concrete. Again the direction of cracking can be controlled by the orientation of the burster.

Splitting can also be accomplished using an expansive cement (e.g. Bristar). This is mixed with water to form a paste which is transported under water in plastic bags and inserted into predrilled holes. The cement slowly expands over a period of 12–24 h to burst the concrete. Alternatively, a preshaped plug of Bristar can be exposed to water at the work site for a preset period and then inserted into the hole.

The Cardox system is a splitting technique which uses cartridges of compressed carbon dioxide to create internal pressure. Cardox tubes charged with liquid carbon dioxide are placed in predrilled holes and corked in firmly. A non-explosive chemical mixture, in a paper container, acts as the energizer of the carbon dioxide. By passing an electric current through this mixture a reaction is initiated, raising the pressure of the carbon dioxide which escapes into the hole and causes cracking of the concrete between adjacent holes.

Following the use of Cardox bursting tubes, reinforcement bars may still need to be cut. By experienced positioning of the charges, however, reinforcement bars of up to 15 mm diameter can be sheared by the detonation.

### 5.3.4 Reinforced concrete removal

Some of the techniques described above, whilst principally used to break up concrete, can be cut through reinforcement. In many cases, however, they must be used in conjunction with steel cutting techniques if reinforced concrete is to be completely removed.

Two approaches, explosives and the thermic lance, can be used to break out or cut through reinforced concrete in one operation. Explosives have been used for many years in underwater applications by the use of contact demolition charges. As with many underwater cutting techniques, the use of an experienced contractor is essential as successful controlled demolition is very dependent on the size and placing of the explosive charge. The break-out achieved by conventional contact explosives is very irregular and damage to the adjacent structure can result.

Where a more precise cut is required, the use of a 'shaped' charge is a better option. This consists of a selected explosive contained in a sheath of soft metal. The sheath normally contains a conical section which, when the explosive is detonated, is propelled forward as a stream of metal particles. This concentrates the cutting action in a localized area of concrete. The shaped charge produces a smoother cut than a conventional charge and, as the explosive is used more efficiently, reduces the intensity of shock waves.

The thermic lance is a well proven technique for cutting through thick sections of reinforced concrete. The lance consists of a mild steel tube in which a number of steel rods are packed. Oxygen is passed down the tube to fuel the iron–oxygen fusion process, which is initiated at the end of the lance by heating with conventional burning gear. Temperatures of around 2200 °C are generated, enabling concrete and steel to be cut through. When cutting reinforced concrete the concrete is melted by the intense heat and runs off as slag. When reinforcement is reached it is 'burned' generating considerable extra heat.

The thermic lance is limited for use in shallow water, as at depth the hydrostatic pressure increases both the pressure of oxygen required and the rate at which the lance is burnt away. 'Steam explosion' between hydrogen from the water and unburnt oxygen can also be sufficiently severe to be a hazard to the operator.

Both explosive cutting and the thermic lance are best suited for complete removal of sections of structure as both the concrete and reinforcement are cut. Prior to undertaking repairs, further cutting out of the concrete (e.g. by water jetting) to provide a connection to replacement reinforcement is required.

## 5.3.5 Reinforcement cutting

Before re-concreting the damaged area, broken and severely distorted reinforcing bars must be cut away and replacement bars installed. When cutting away reinforcement, consideration must be given to the method of joining in the replacement bars so as to ensure that the cut ends are suitable for couplers or welding if required. The three most commonly used methods for cutting reinforcement under water are outlined below.

### 5.3.5.1 Mechanical cutting

For small-scale repairs, where only a limited number of bars have to be cut away, the most convenient approach is by the use of mechanical cutting. A range of hydraulically driven tools for underwater use including disc cutters and bolt croppers is available. For smaller diameter bars, hand-operated cutting tools can be used.

### 5.3.5.2 Oxy–hydrogen cutting

Widely used above water for cutting steel, the oxy–acetylene torch relies on the interaction of the gas flame and the carbon steel to be cut. The steel is oxidized and 'burnt' away. Under water the acetylene is replaced by hydrogen to overcome the problem associated with the instability of acetylene at depth.

The oxy–hydrogen flame, however, is not as hot as the oxy–acetylene flame, resulting in a much slower cutting or burning process. The increased speed and ease of cutting using the oxy–arc method has resulted in a decline in the use of oxy–hydrogen cutting.

#### 5.3.5.3 Oxy–arc cutting

The heat for oxy–arc cutting is generated by an electric arc rather than a gas flame. Oxygen, at a pressure of between 5 and 8 MPa, is forced down the centre of a hollow electrode causing the reinforcement steel to be oxidized and blowing away the oxidized product. This leaves the end of the cut bar clean, ready to be welded or to receive a reinforcement coupler. Much of the available oxy–arc cutting equipment can also be used for underwater welding by changing the type of electrode used.

## 5.4 Repair materials

### 5.4.1 Selection of material

An extensive range of materials is available for use in underwater repair. They can be broken down into two main types: cementitious and resin-based.

#### 5.4.1.1 Cementitious materials

These can range from conventional mortars and grouts to materials with greatly enhanced properties achieved by the use of admixtures. The use of admixtures can result in cohesiveness, high rates of strength gain, greater workability, resistance to washout of cement and reduction in bleed and shrinkage. Many cementitious repair materials are available as proprietary products, their properties specifically developed to allow placement under water.

The principal advantage of cementitious materials over resin based materials include:

- compatibility with the structure in terms of modulus of elasticity and thermal expansion
- can be used in thicker sections without excessive heat build-up and risk of thermal cracking
- considerably cheaper
- less susceptible to errors in mixing and applications
- safe for use by divers.

### 5.4.1.2 Resin-based materials

These are generally based on epoxy resin and include injection resins, pourable mortars and hand-applied putties. Epoxy resins have a lower modulus of elasticity and higher creep than cementitious materials and are therefore usually less suitable for structural repairs. Where thick sections are to be repaired the temperature rise during curing may lead to high stresses and subsequent cracking.

The principal advantages of resin-based repair materials over cementitious materials include:

- very low viscosity for injection into fine cracks
- high bond strength
- high flexibility if required to accommodate movement
- high strength and rate of strength gain
- resistant to penetration by water, salt, etc.

The cost of access, preparation and provision of formwork for underwater repairs is significantly higher than for normal repairs, and the ability to supervise and inspect the repair as work progresses is limited.

Whatever material is selected, it is essential that either by prior experience, or by a programme of testing, the material is shown to be suitable for the particular application. Where possible the performance of the material should not be unduly sensitive to errors in the mixing and placing techniques.

As a minimum, the following tests on the material selected should be considered prior to undertaking the repair. These should be carried out in an environment and at temperatures representative of those that will be experienced during the repair operation.

- Structural properties—are strength, modulus and creep properties suitable for a structural application?
- Flexibility—is the flexibility adequate for the expected movements? Will the material become more brittle with age?
- Washout of cement—when allowed to free fall through water, does the cement wash out?
- Pot life/rate of hardening—will the material remain usable for a sufficient time to allow proper placement? Are the subsequent rate of hardening and strength gain adequate?
- Placement trials—can it be pumped slowly and continuously, and will it self-compact and self-level?
- *In situ* cores—is the *in situ* strength as required, has washout of cement

resulted in weakness of top layer, has intermixing with water reduced strength?

- Bond strength—has material bonded adequately to parent concrete or other areas of repair?

## 5.4.2 Concrete

For large-scale repairs the use of concrete as the repair material will usually be the most economical alternative. Concrete for underwater use should be free-flowing, cohesive and self-compacting. It should not be prone to bleeding or plastic shrinkage, both of which could result in weaknesses at the repair/structure interface.

The characteristics required for a pumpable concrete will generally be suitable for most underwater applications. The principal constituents of the mix are discussed below.

### 5.4.2.1 Cement

A range of cement types with or without the addition of ground granulated slag or pulverized fuel ash can be used for underwater concrete. To achieve a cohesive mix of cement content of between 350 and 425 kg/m$^3$ is often used, although some properietary mixes have a considerably higher cement content.

### 5.4.2.2 Aggregate

To achieve a high-workability mix, a rounded aggregate is much more suitable than crushed stone. In particular, crushed rock fine aggregate should be avoided because the gradings are usually poor and the particle shape unsuitable. The use of a harshly graded sand can greatly increase the amount of bleed that occurs.

### 5.4.2.3 Admixtures

The quality and ease of placement of the concrete can be greatly enhanced by the use of admixtures. The principal types of admixtures that are used for underwater concrete and their uses are summarized below:

- Plasticizers—
  Enable a lower water content to be used for a given workability. This results in concrete of a higher strength and density and reduced permeability. For the same water content, the workability is increased, facilitating placing and compaction.

- Superplasticizers—
  Result in a very high workability concrete of a flowing consistency.

The concrete can be self-compacting and self-levelling. The need for vibration, which can increase the risk of mixing between the repair concrete and surrounding water, can be avoided.

- Air entrainers—
  These improve the cohesiveness and workability of the concrete. This minimizes bleed where fine aggregate grading is poor and may assist in pumping.
- Retarders—
  These delay setting times, reducing the risk of cold joints and allowing more time for placement. Retarders are often combined with plasticizers or superplasticizers.
- Polymer modifiers—
  These can greatly enhance the properties (particularly bond and tensile strength) of concrete and mortar and are discussed in Section 5.4.3.
- Non-dispersible agents—
  These reduce the risk of washout of cement from the repair concrete where it comes into contact with the surrounding water. They usually consist of a water-reducing agent in combination with a viscosity increaser such as cellulose ether or polyethylene oxide (see Table 5.1). In many cases they are included in premixed underwater concretes or mortars (see Table 5.2).
- Microsilica—
  The addition of microsilica (usually accompanied by a superplasticizer) can result in a high strength, washout-resistant concrete.

**Table 5.1** Non-dispersible admixtures

| Manufacturer | Product | Comments |
|---|---|---|
| Rescon | Rescon T<br>Nonset 400UV | Retarding effect 15–20 kg/m$^3$ |
| Shimizu Construction | Aklith 12A<br>Aklith 12S | |
| Cormix Construction Chemicals | UCS | Combination of two admixtures |
| Armorex | UW3 | |
| Sika Intertol | UCS | |
| Fosroc | Conplast UW<br>Conbex 250 | Cement content at least 400 kg/m$^3$ |
| Hydraulic Underwater Concrete | Hydrobond UWA-1<br>Hydrobond UWA-2 | Microsilica-based<br>Polymer-based |
| Scancem Chemicals | Betokem S-UV<br>Hydrocem | Two components (liquid + powder)<br>Microsilica-based |

**Table 5.2** Non-dispersible concretes/mortars/grouts

| Manufacturer | Product | Comments |
|---|---|---|
| Hydrocrete | Hydrocrete concrete | Permeable or impermeable |
| | Hydrocrete grout | Pourable/pumpable |
| Armorex | UW4 | Grout mortar, plastic or pourable |
| | UW5 | Coarse grout mortar, plastic or pourable |
| | UW6 | As UW4 but fast set |
| | UW7 | As UW5 but fast set |
| Fosroc | Conbextra UW | Non-shrink pourable/ pumpable |
| Rescon | 50 UV-T | 0.2 mm aggregate pourable/ pumpable |
| | 600 UV-T | 6 mm aggregate pourable/ pumpable |
| | Nonset 400 UV | Expanding, pourable |
| Thoro System Products | Waterplug | Rapid setting, hand/trowel |
| Ronocrete | Monoset U/W | Rapid setting, hand/trowel |
| MRA | Protongrout | |
| Hydrobond Underwater Concrete | Hydrobond UWC-3 | Polymer/microsilica concrete |
| | Hydrobond UWC-4 | Microsilica-based rapid setting concrete |

#### 5.4.2.4 Proprietary underwater concretes

A range of proprietary pre-bagged non-dispersible concretes is available (Table 5.2). These have been formulated to be highly cohesive and resistant to washout of cement, particularly when they are allowed to free fall through water. Their consistency is generally such that they are self-levelling and self-compacting and therefore do not require vibration.

Concretes designed to be non-dispersible should be used where the concrete is to be poured into formwork, particularly where reinforcement will increase the risk of cement washout. Their use is of lesser importance when the concrete can be gradually pumped into formwork to result in the gradual displacement of water.

### 5.4.3 Cementitious mortars and grouts

Where the thickness of the damaged area of congestion of reinforcement precludes the use of concrete, cementitious mortars and grouts may be required. The consistency and sand content and grading will depend on the nature of the repair.

For small patch repairs, a trowel-applied sand/cement mortar may be suitable. Where this is to be placed under water or in the splash zone, the use of a specially formulated Portland cement or ultra-rapid-hardening cement can be used to prevent the repair being washed off. Very rapidly hardening mortars can be used to seal the surface of the structure prior to permanent repairs by ground injection. More flowable cementitious mortars can be used to repair surface spalls, being placed either by pumping or pouring into letter-box formwork.

For injection into cracks. or for preplaced aggregate using small-sized aggregate, a sand-free cement grout may be required. The omission of sand allows greater penetration of the grout and prevents blockage owing to 'bridging' of the sand. In general, cement grout should not be used for injection into large voids as thermal and shrinkage cracking are likely to occur.

The properties of cementitious mortars can be enhanced by the use of polymer modifiers. These are generally monomer, resin or latex liquids added as a partial replacement for the mixing water and result in an improvement in the properties of the mortar, including:

- a marked increase in flexural and tensile strength
- increased compressive strength
- reduced permeability, bleed and plastic shrinkage
- increased bond strength without the need for a bond coat.

Polymer modified mortars, however, do have a lower modulus of elasticity and a higher creep, both of which can result in problems in structural applications. Care in the selection of polymer-modified mortars is essential as some materials are unsuitable for use under water or in moist environments. In particular, the strengths of polyvinyl acetate (PVA)- and acrylic copolymer-modified mortars are seriously affected by use under water. For these conditions styrene–butadiene rubber (SBR)- or styrene–acrylic copolymer-based modified mortars are the most suitable.

Polymer-modified cement slurries can be brushed into the prepared concrete surface prior to placement of the repair mortar to act as a bonding aid. Owing to the specialist nature of polymer modifiers, it is essential that the manufacturer's advice on the use of their materials is sought.

A range of proprietary cementitious mortars suitable for use under water is listed in Table 5.2.

### 5.4.4 Preplaced aggregate concrete

Preplaced aggregate concrete is formed by injecting sand/cement grout into formwork containing compacted aggregate. The grout fills the formwork leaving a high aggregate/cement ratio concrete with 'point-to-point' aggregate contact. This has the advantages of low restrained shrinkage (50–70% that of conventional concrete), no segregation and low settlement. However, if the concrete is to be a strong, low-permeability mix, microcracking due to settlement of the cement paste and bleed water below the aggregate particles must be avoided. This can be achieved by suitable design of the grout mix.

The recommendations in the Concrete Construction Handbook[4] should be followed to ensure successful placing of preplaced aggregate concretes. These recommendations are summarized below.

*Coarse aggregate.* Use as large a maximum size as is convenient to handle, subject to the limitations of the aggregate being smaller than either one quarter of the minimum dimension of the form or two thirds of the minimum reinforcement spacing. A minimum size of 14 mm for sections up to 300 mm and 19 mm for thicker sections should also be used.

Where reinforcement is congested, or where thin section repairs are to be undertaken, then a smaller maximum aggregate size may be required. The aggregate should be graded to give a minimum void content, which is usually between 35% and 40% after compaction, and it should be free from silt.

*Fine aggregates.* The grading of the sand should satisfy the zone 3 classification of BS 882, but of particular importance is that the sand is uniformly graded, to facilitate pumping of the sand/cement grout into the aggregate interstices. Where 'small size' (say 10 mm) large aggregate is used then problems may arise in pumping in sand/cement grout because the sand bridges between the aggregate particles and causes blockage. In this case a pure cement grout with suitable admixtures to reduce shrinkage may be necessary.

*Cement.* Any of the standard types of Portland cement can be used. If particular problems are envisaged (e.g. sulphate attack), then the appropriate cement can be selected.

*Admixtures.* Usually, a pozzolanic filler is used to improve the flow of the grout. Also, proprietary admixtures are used which prevent bleed, plasticize, entrain air and create a slight expansion during setting of the grout. Typical admixtures which fulfil this task are based on a combination of a cellulose ether thixotropic thickener and a plasticizer. An accelerating admixture can be included in cold waters or where early form strippings is required.

*Proprietary grouts.* A range of proprietary cementitious grouts suitable for underwater prepacked aggregates are available. These may include additives to reduce shrinkage, bleed and washout of cement. Some of the available grouts that could be used for preplaced aggregate repairs are summarized in Table 5.2.

### 5.4.5 Resin mortars and grouts

Normal epoxy or polyester resins are unsuitable for underwater use as they often fail to bond to the damaged concrete and can be adversely affected by reaction between the hardener and water. By special formulation of the base resin and hardener, however, some epoxy resin systems have been developed for use under water.

Even with underwater-grade epoxy resins, a severe reduction in performance occurs where turbulence during placement results in an intermixing between the resin and water. In general, polyester resins remain unsuitable for underwater use owing to poor bond performance.

Epoxy resin systems are available in a range of consistencies from very low viscosity injection resins, through sand-filled pourable mortars to hand- or trowel-applied putties. In many cases the same resin system is used with differing proportions of inert filler, often sand, added to achieve the required consistency.

When selecting a suitable material particular consideration must be given to:

- Consistency—in relation to the method of placement and sizes of void or formwork into which the material is to be placed.
- Flexibility—if the material is required to carry load it must have a high modulus and low creep. If the material is to be used as a sealant then high flexibility may be desirable.
- Heat generated during curing—depends on the rate of hardening, the amount of inert filler and the thickness of the repair. The thickness of repair must be limited to prevent large temperature increases and subsequent cracking.
- Rate of hardening—the time required during which the material is still usable will depend on the temperature, the method of placement and the complexity of the repair. For some repairs, such as crack sealing and injection nipple attachment, a rapid-hardening material will be required. Many epoxy resin systems are formulated in both normal and rapid-hardening versions.

A selection of epoxy resin base materials suitable for use underwater is given in Table 5.3.

**Table 5.3** Epoxy resins

| Materials | Manufacturer | Product | Comments |
|---|---|---|---|
| Injection resins/grouts | Rescon | BI-PA 1.6 resin<br>UL-L-1.5 hardener | Crack injection/thin layers |
| | Rescon | UV-L | Bonding aid/adhesive |
| | Colebrand | CXL 78R | Crack injection, resin anchors |
| | | CXL 600 | 15 μm cracks |
| | | CXL 78T | Grout, small splits, adhesive |
| | | CXL 194 | Adhesive |
| | Sika Inertol | Sikadur 53 LV | Low-viscosity injection |
| Mortars | Rescon | UV-S | Paste, hand/trowel applied |
| | Structural | NM205 U/W | Putty, hand/trowel applied |
| | Chemicals | NM208 U/W | Mortar, hand/trowel applied |
| | Expandite | Expocrete UA | Sand-filled mortar |
| | Armorflex | Armorex | Putty |
| | Sika Inertol | Sikadur 53 | Can be filled, pourable |
| | FEB | Underwater epoxy | Mastic |

The basic epoxy resin–hardener system has a density similar to that of water and can therefore float around and cause a hazard to divers. Even when used as a mortar or putty care must be taken to avoid contamination of the diver and his equipment. When using epoxy resins it is essential that the manufacturer's instruction for mixing and application are followed to ensure satisfactory performance. To assist with quality control, most epoxy resins repair materials are supplied in prepacked quantities of resin, hardener and filler.

## 5.5 Repair techniques for concrete

### 5.5.1 Surface spalling repair

Where accidental damage has resulted in localized spalling of the concrete cover it is imperative that the cover is replaced to prevent future corrosion of the reinforcement occurring. In the splash zone, in particular, corrosion can quickly turn an area of minor damage into more widespread deterioration.

Prior to replacing the concrete cover, the area must be thoroughly prepared to remove any loose concrete and marine growth as described in Section 5.3. The perimeter of the spalled area should then be saw-cut to a depth of 12–20 mm depending on the extent of damage to eliminate feather edges.

In the splash zone it will generally be feasible to use a trowel-applied cementitious mortar to the damaged area. Where the extent of damage is very limited then a water-tolerant epoxy mortar/putty may be appropriate.

After preparation, the basic steps in the repair of a localized area of spalling in the splash zone are as follows:

- thoroughly flush the area with fresh water and leave damp (except where some epoxy materials are to be used)
- apply a bonding coat, working it well into the surface (not necessary with some epoxy repairs and polymer-modified mortars)
- before the coat has set apply the repair mortar
- apply a curing membrane to cementitious repairs
- provide protection against wave action until the repair has hardened sufficiently.

For larger patch repairs in the splash zone and for most repairs underwater it will be necessary to provide formwork to contain the repair material. The provision of any formwork causes a delay in the repair operation, allowing marine growth to develop. The delay may also prevent the use of a bonding coat as it is essential that the repair material is placed

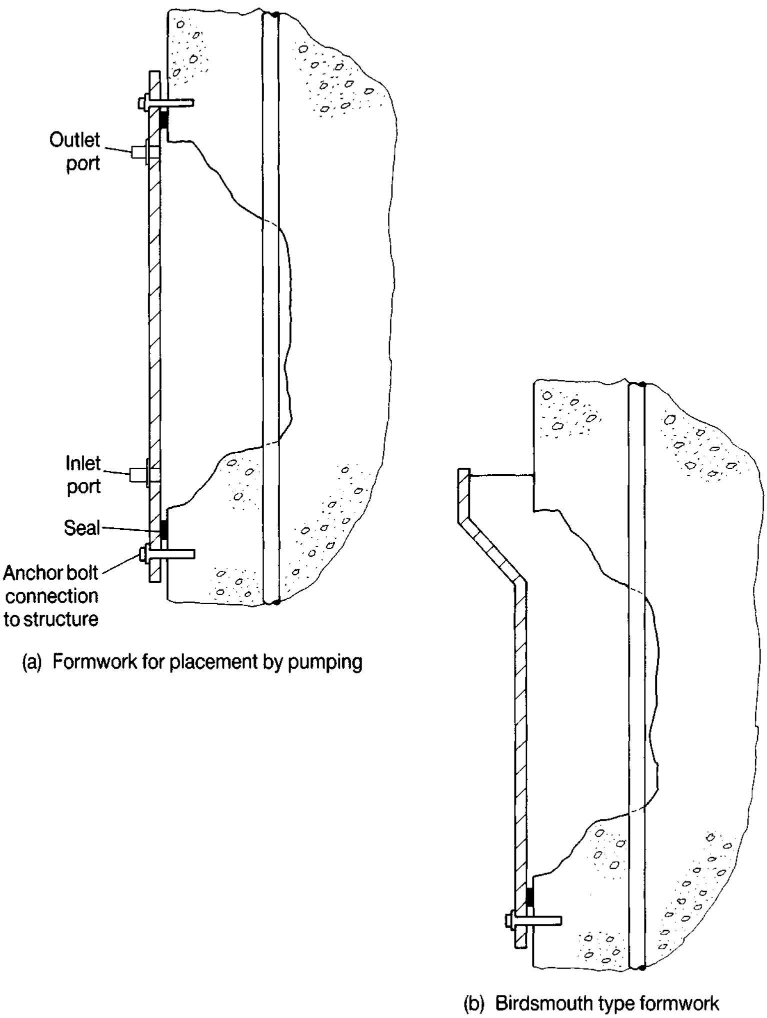



**Fig. 5.2** Formwork for patch repair

while the bonding coat is still tacky. If the bonding coat is allowed to harden then it will set to a smooth surface and provide a poor bond.

Two typical types of formwork for patch repairs are shown in Figure 5.2. These are usually bolted to the structure using rawlbolts or, in some cases, strapped around the member.

Where the repair material is to be pumped into the formwork, two openings are provided. Grout or mortar is pumped continuously in near the bottom of the damaged area, and displaces the water up and out near the top of the formwork. This minimizes any mixing between the repair material and the water, reducing washout of cement. The risk of trapping pockets of water within the repair is also reduced. The movement of

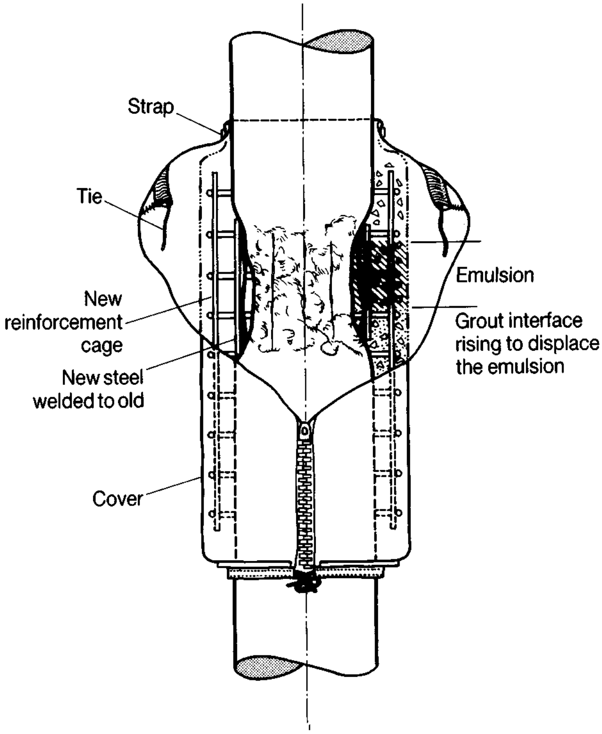



**Fig. 5.3** Details of a repair using flexible formwork

mortar into the formwork develops shear stresses at the concrete interface, forcing the mortar into the surface and improving the bond.

Where the repair material is poured into the formwork, a letter-box top opening must be provided. This results in a section of the repair which must be cut away after the formwork has been stripped. When selecting a repair mortar for pouring into a letter-box type formwork, a mix that will not readily suffer washout of cement must be selected. The mix must also be self-compacting as vibration would lead to intermixing with the water.

Flexible formwork can also be used for repair work. Figure 5.3 shows flexible formwork attached around a damaged column. Where reinforcement is provided care must be taken to ensure than adequate spacers and fixings are provided to control the cover to bars, particularly where strong currents may cause movement of the formwork.

A repair technique used by Rescon involves initially partially filling the formwork with a bonding agent emulsion (see Figure 5.3). The repair mortar or concrete is then introduced into the formwork, displacing the bonding agent which coats the prepared surface of the concrete.

## 5.5.2 Large-scale repair

The need for large-scale repair will generally have been brought about by structural overloading, fire damage, ship impact or, perhaps most commonly in the splash zone, reinforcement corrosion.

Where large areas are to be repaired the selection of repair material and methods are of critical importance if bleed or shrinkage is not to result in a leakage path at the top of the repair/parent concrete interface. In thick repairs excessive temperature rise in some repair materials may result in thermal cracking, although the heat sink effect of the surrounding water will reduce the temperature rise.

In many cases it may be necessary to undertake repairs to reinforcement because it either has been distorted or has corroded significantly. The repair of reinforcement is discussed in Section 5.6.

The general procedures for undertaking a large scale repair are as follows:

- Prepare the damaged area as discussed in Section 5.3. It will generally be necessary to cut away the concrete behind the reinforcement to ensure that the bars are protected from further corrosion. This will also ensure that the repair is well tied into the structure. The perimeter of the area should be saw cut to at least 20 mm to prevent a feather edge. At the top of the damaged area the concrete should be cut back at an inclined surface to ensure that water does not become trapped against the concrete and bleed water can escape.
- The reinforcing bars should be thoroughly cleaned and, if necessary, replaced or supplemented with additional bars. In the splash zone coating to the bars for corrosion protection should be applied. Under water this is not generally feasible, and in any case the risk of corrosion is not serious.
- The selection of the type of formwork will depend on the method to be used for placing the repair concrete. To minimize the effect of bleed and to ensure a good bond it is beneficial if pressure can be applied until the concrete has hardened. This will require robust formwork which is grout tight and tightly sealed to the structure. Where preplaced aggregate is to be used, then formwork must be modular such that it can be erected in sections as the aggregate placement proceeds. Types of formwork for use under water are discussed in Chapter 3.
- Immediately prior to placing the repair concrete, the formwork should be thoroughly flushed with fresh water to reduce contamination of the concrete with salts.
- Pumping is the most suitable method of concrete placement. Concrete is pumped in near the bottom of the form, displacing water out of the

top. Pumping can be continued to flush out the top layer of concrete which may have intermixed with water in the formwork. To minimize intermixing as the concrete flows around the reinforcement a slow rate of pumping should be adopted and vibration should only be carried out after the formwork is full of concrete. By locking off the upper opening, a pressure can be built up to counteract the effects of bleeding of the mix. The pressure also forces the repair concrete into the prepared surface, increasing the bond strength.

### 5.5.3 Preplaced aggregate concrete

Aggregate which has been graded to minimize the voids content is poured into the formwork and vibrated or rodded until well compacted. A suitable grout is then injected into the base of the formwork containing the compacted aggregate. The rise in the level of the dense grout displaces the water upwards and out of the top of the form as filling proceeds.

For successful injection of the grout, the formwork must be designed to be grout tight to prevent leakage and be tightly sealed to surrounding concrete. It must also be adequately vented at the top to enable air and water to escape. The use of a Perspex window in the form is useful to enable the movement of grout to be monitored as filling proceeds.

Prior to filling the form with grout, the aggregate should be vibrated into place and flushed through with fresh water to reduce sea water or silt contamination. Care must be taken to ensure that the formwork is filled with aggregate to the top of the damaged area, otherwise a region of aggregate-free grout will occur, resulting in high shrinkage and cracking. The grout is then injected through inlet pipes at the bottom of the form in a continuous process without interruption until grout flows out from the top of the form. The injection should then continue to ensure that the initial 'front' of grout that could have intermixed with water and suffer from washout of cement is flushed out of the form. It is recommended that the form is not vibrated during injection (as would be the case above water), as vibration will increase the risk of washout of cement.

For much larger or flat areas of concrete, such as replacing mass concrete pipe anchors or foundation slabs, it is common practice to use vertical grout pipes. These are usually 20 mm in diameter and spaced at 1.5 mm centres, the exact size and spacing depending on the size of the form and the aggregate characteristics. Grout is then pumped into the bottom of the form via these injection pipes, which are raised gradually as filling proceeds.

### 5.5.4 Injection

Cracks or voids in concrete under water can be repaired by injection of resin or cementitious grouts following similar procedures to those used in the dry. The choice of material is largely dependent on the size of crack or void to be injected and whether future movement is expected.

For cracks more than a few millimetres in width, cementitious grout will penetrate sufficiently; for thinner cracks, down to about 0.1 mm epoxy resin will be more suitable. It is generally not necessary to undertake crack injection when the crack width is less than 0.1 mm. The depth of penetration into cracks will also depend on the applied pressure and the time for which the pressure is maintained before the repair material solidifies.

Where there is evidence of corrosion at a crack it will be necessary to break out the concrete back to the reinforcement and carry out a full repair rather than merely to inject the crack.

The general procedure that should be adopted for crack injection is as follows:

- Prepare the concrete surface along the length of the crack.
- Attach inspection nipples at intervals along the crack using the rapid-setting cementitious or resin putty. The spacing of the nipples and their diameter will depend on the size and form of the crack to be injected and the material to be used. A spacing of between 100 and 300 mm between nipples and a diameter of 5 mm are typical. The nipples can either be bonded directly on to the surface over the crack or be inserted into a hole drilled into the crack.
- Seal the surface of the crack along its entire length. This can be achieved either by cutting a small groove along the crack and filling this with mortar or more simply by applying the mortar to the concrete surface.
- Flush the crack with fresh water to remove contaminants and ensure that the injection path is open. The use of coloured water will enable leakage points to be identified and sealed.
- Inject cementitious or resin grout into the crack through the nipples at one end of the crack. Continue injection through successive nipples until the crack is completely filled. Lock off each nipple after use.

Two methods of injection commonly used are gravity feed and pressure injection. Where pressure is used for resin injection, the two components of the resin can either be mixed on the surface and transported to the repair site in a pressurized contaaininer, or be mixed at a special intermixing nozzle immediately prior to injection.

Epoxy resins can have a density similar to that of water. Any material that leaks will therefore float around and cause a hazard to the divers undertaking the repair. All steps must be taken to minimize any leakage of resin during the injection operations.

Where a rigid epoxy resin is used then the tensile and shear strength of the section can be restored. Epoxy resin systems are thus capable of restoring some degree of strength to the concrete and are therefore important where structural integrity is critical.

Where the original cause of cracking recurs then further cracking will result. In these cases it is best to treat the crack as a movement joint by using lower modulus material to inject the crack or provide a seal on the surface. Suitable materials based on the more flexible polyurethane or polysulphide sealants will provide a barrier to the ingress of moisture and salts whilst still allowing some movement.

### 5.5.5 Gunite (shotcrete)

Where a large surface area is to be repaired or a column or beam is to be encased, then the use of gunite may be the best solution. The dry mix process where sand and cement are passed through the delivery hose and mixed with water at the nozzle is generally used.

Although gunite cannot be applied under water, the use of additives to promote very rapid setting can enable the method to be used in the splash/tidal zone. Products are available (e.g. Sigunite from Sika Inertol) that can produce an initial set within 30 s and a final set within 1 min.

The successful application of gunite is very dependent on the skill and experience of the nozzleman in adjusting the water supply and the pressure and ensuring uniformity of thickness. With careful application concrete strengths of 30 MPa can readily be achieved with good bonding to the parent concrete and high abrasion resistance.

The thickness of the gunite should generally be limited to a maximum of 50 mm, although second layers can be applied if an increased thickness is required.

A detailed discussion on the use of gunite for repairs has been given by the Concrete Society[5] and Heneghan,[6] the latter being directed specifically at repairs in marine environments.

### 5.5.6 Steel sleeve

A major operation in the repair of reinforced concrete piles or columns affected by reinforcement corrosion is the breaking out of damaged concrete, often to behind the bars. This operation will often require temporary supports to be provided for the remaining structure.

An alternative to this method of repair is the provision of a steel sleeve around the pile. The void between the sleeve and pile is then filled with concrete or mortar. The sleeve can be designed to accommodate further corrosion (and the resulting expansion) of the reinforcement. It must also be able to resist the forces in the pile in the event that the bars corrode sufficiently to become ineffective owing to either loss of area or loss of bond. To achieve this the sleeve wall must be sufficiently thick and it must

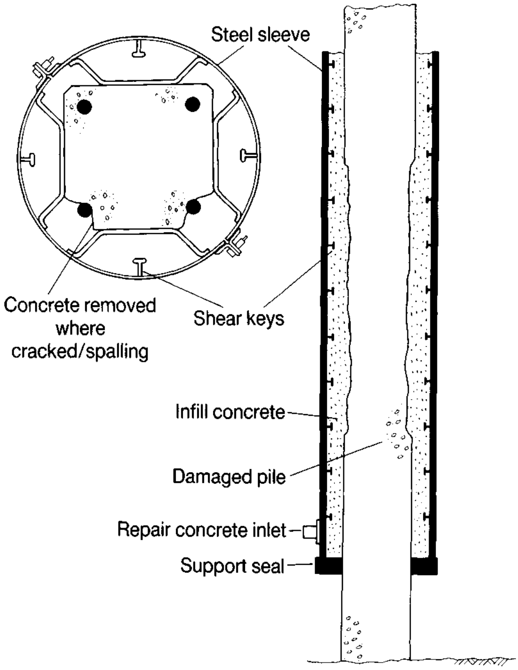



**Fig. 5.4** Steel sleeve repair

extend above and below the damaged length of pile. Load is then transferred to and from the sleeve by shear.

The general arrangement of a steel sleeve repair is shown in Figure 5.4 and the method of repair is summarized as follows:

- Prepare the damaged pile by removing marine growth and any loose sections of concrete.
- Clamp a temporary support/sealing ring around the pile below the damaged area. In general, corrosion damage will be limited to the splash/tidal zone. A sufficient length of undamaged pile is therefore present below the water.
- Bolt the two semi-circular sections of the sleeve together around the pile.
- Pump repair concrete/mortar in near the bottom of the sleeve to displace the water.
- Remove the temporary support/sealing ring and apply a corrosion protection coating to the steel sleeve.

## 5.6 Reinforcement repairs

### 5.6.1 General considerations

In cases of severe damage to reinforced concrete structure there is a possibility that the reinforcing bars will be broken or at least severely distorted. Where damage has been caused by impact the damage to reinforcement may be localized either at the point of impact or where rotations of columns or piers have occurred. In many cases it will be sufficient to force the distorted bars back into their original position to ensure adequate cover and to replace the damaged concrete.

Where the bars have been badly distorted, however, restraightening will affect the structural performance of the bars. The effect of restraightening will depend on the type of bar and its diameter. A reduction in the failure strain and the fatigue life may be expected although the tensile strength may not be greatly reduced.

In the splash zone in particular, general corrosion or local pitting can result in a significant loss of area of steel, necessitating the provision of additional bars. Link reinforcement, especially at corners, may be particularly badly affected.

When designing the method to be used for repairing damage to reinforcement, several problems must be considered:

- congested reinforcement may hamper access
- existing bars may be in bundles
- repairs may have to be carried out under water
- access may be from one side only.

Where prestressing tendons have been damaged it will be necessary not only to join in new tendons, but also to provide a method to restress the area.

If, however, the problems are recognized it is possible to design a suitable repair system to allow replacement reinforcement to be fixed and to provide a structurally sound repair.

### 5.6.2 Lap joints

The minimum length of lapped joints in tension, as specified in BS 8110, is 37 times the diameter of the bar (Type 2 bar in 30 N/mm$^2$ concrete). For a 20 mm diameter bar this is 740 mm. In reinstating damaged areas this would entail breaking out some 800 mm of concrete surrounding the immediate damage in order to form appropriate lap points in the reinforcement. Where the cover to the bar or the spacing between bars is small, or where the laps must be staggered, an even larger area of concrete must be broken out. In the USA, the requirements of ACI 318[7] should be followed.

Extensive breaking out of sound concrete in order to provide an adequate lap length for reinforcement would entail greater costs, not only in the breaking out but also in time and materials for replacing the concrete. In heavily reinforced sections provision of additional bars will greatly increase congestion and may result in difficulty in placing the repair material. This method must therefore be considered suitable only in cases where the damage to concrete is considerably more extensive than the damage to reinforcement.

### 5.6.3 Welding

The trend with modern bars is to produce them from steels having lower carbon equivalent values and thus making them more easily weldable. Connections can be made by welding either butt joints or lap joints. BS 8110 allows both form of joints but generally limits the strength of butt-welded joints to 80% of the strength of the bars. The required length of a lap-welded joint is of the order of only 80 mm for a 20 mm diameter bar, thus minimizing the amount of concrete needed to be broken out. ACI 318[7] permits the use of welded splices for large bars (No. 6 or larger) in main structural members, provided that the welded connections develop in tension at least 125% of the specified yield strength.

Welding can therefore be considered as a feasible method for joining replacement reinforcement to existing damaged reinforcement. It should be noted that welding has an adverse effect on the fatigue properties, and is not permitted on some reinforcement.

Trials of underwater welding should be carried out, and samples tested prior to undertaking the main repair.

### 5.6.4 Reinforcement couplers

These can be either mechanical or resin couplers. Mechanical couplers consist of a sleeve (a seamless tube) which is placed over the ends of the two bars to be joined. A hydraulic press is then used to swage the tube onto the bars. BS 8110 and ACI 318[7] allow the use of mechanical couplers subject to tests carried out using the exact type of reinforcement to determine the deformation after loading and the ultimate strength.

Where reinforcement is congested there may not be sufficient access for the hydraulic swaging tool. Where this method can be used, however, it results in a compact joint (typically 150 mm long for a 20 mm diameter bar). As the method relies on mechanical interlock between the sleeve and the bar, it can be made under a wide variety of site conditions since it is not affected by temperature, the surface condition of the steel or the presence of water.

Resin-grouted couplers are similar to mechanical couplers in that they consist of a sleeve placed over the ends of the bars to be joined. In this case, the sleeve is sealed at each end prior to being injected with a rapid-hardening resin to provide load transfer between the bars and the sleeve. Load transfer relies on the strength of the resin itself and on the bond/mechanical interlock between the resin and the bar and tube. The length of the coupler is typically the same as for a mechanical coupler.

Resin-grouted couplers can accommodate limited distortions of the bar and can be designed for multiple bars. One problem with grouted couplers is creep of the resin under high stresses. As, however, the replacement reinforcement is at least initially unstressed, creep may be less important than other factors.

### 5.6.5 External steel plates

In some repair situations it may be advantageous not to joint individual reinforcement bars but to provide extra reinforcement by means of a steel plate glued and bolted on to the face of the concrete section.

A considerable amount of research has been carried out on the performance of strengthened beams and bridges by the use of bonded steel plates and the main points relevant to the repair of offshore structures may be summarized as follows:

- Crack widths for a given load are reduced and the ultimate moment increased, particularly if rigid epoxy resins are used.
- Glue thicknesses of less than 1 mm are found to be detrimental although there appears to be no extra benefit in applying the epoxy resin in layers more than 2 mm thick. In the case of a steel plate bonded to an uneven concrete surface, variation in the thickness of resin would be needed to take up irregularities in the surface profile.
- Very fine cracks have been observed both in the concrete and the epoxy resin materials, enabling corrosion agents to reach the inner steel surface. Some corrosion has been noted after 2 years of exposure, although this did not affect the structural performance.
- The long-term effect on epoxy resin adhesive, of immersion in sea water can be a reduction in bond and shear strength.

Hence although there would appear to be sufficient experience in the use of external reinforcement to enable a workable system to be designed, the long-term performance of a resin-bonded plate has yet to be established. It is therefore essential to provide mechanical fixing in the form of concrete bolts or through ties in addition to the epoxy resin.

In most applications bonded steel plates have been used to strengthen underdamaged structures. Where the structure has been damaged the steel plate and its fixings must be designed not only to provide continuity of reinforcement, but also to resist concrete pressures during the repair operation.

For repairs to columns or piles, the use of a prefabricated steel tube clamped around the member may be considered. This method is described in Section 5.5.6.

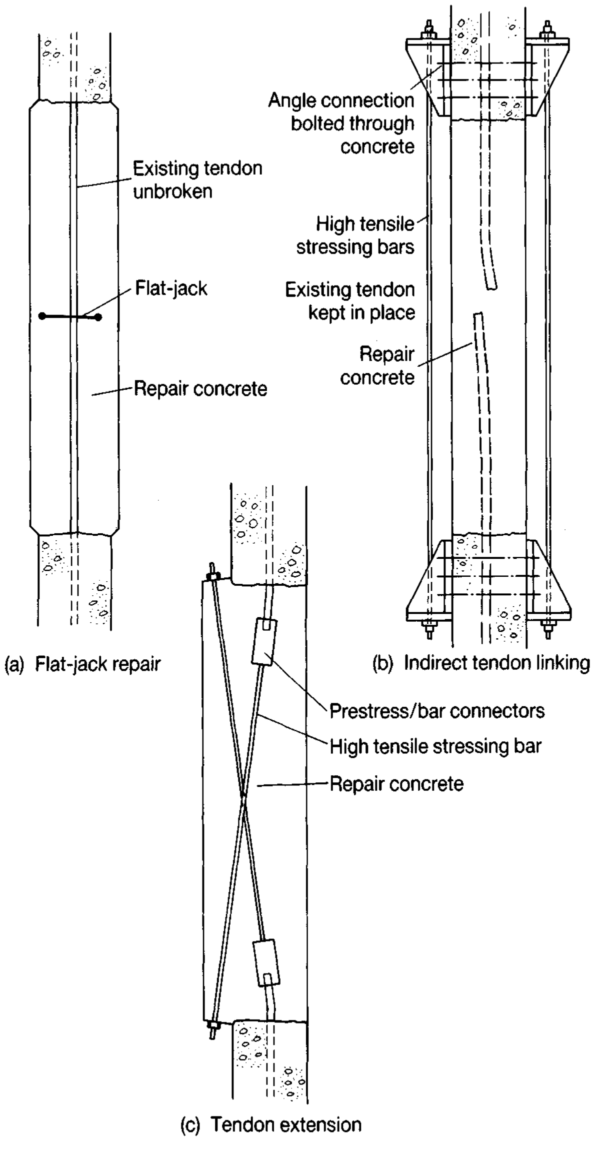



**Fig. 5.5** Prestress repairs

### 5.6.6. Repairs to prestressing tendons

Where prestressing tendons have been damaged or broken it may be required, in addition to repairing the tendons, to restress the damaged area. This may be necessary to induce compressive stresses in replacement concrete to prevent subsequent cracking under cyclic loading.

A number of options for reinstating prestress may be considered,[1] depending on the nature of the damage. Three approaches are illustrated in Figure 5.5:

- Flat-jack repair—where there are no tendons or the tendons are not damaged, flat-jacks can be used to induce compressive stress in the replacement concrete.
- Indirect tendon linking—steel brackets bolted to the structure above and below the damaged area are stressed together using Macalloy stressing bars.
- Tendons extension—the broken tendons are extended using Macalloy stressing bars which extend out through the face of the locally thickened structure.

## References


1 McLeish, A. (1984) Repairs to major damage of offshore concrete structures, in *Proceedings of 2nd European Community Symposium, Luxembourg*, pp. 308–319.
2 UEG (1983) *Handbook of Underwater Tools*, Report No. UR18, Underwater Engineering Group, CIRIA, London.
3 Keeney, C.A. (1987) *Procedures and Devices for Underwater Cleaning of Civil Works Structures*. Technical Report REMR-CS-8, US Army Engineer Waterways Experiment Station, Vicksburg, MS.
4 Waddell, J.J. (1974) Preplaced aggregate concrete, in *Concrete Construction Handbook*, Chapter 38. McGraw-Hill, New York.
5 Concrete Society (1978) *Assessment of Fire Damaged Concrete Structures and Repairs by Concrete*. Working Party Report No. 15, Concrete Society, London.
6 Heneghan, J.I. (1965) *Shotcrete Repairs of Concrete Structures in Marine Environments*. ACI Publication SP 65-28, American Concrete Institute, Detroit, MI.
7 American Concrete Institute (1989) *Building Code Requirements for Reinforced Concrete*. ACI 318-89, American Concrete Institute, Detroit, MI.
8 British Standards Institution (1985) *Structural use of concrete, Part 1 Code of Practice for Design and Contruction*, BS8110.

# 6 Durability of concrete underwater

P S Mangat

## 6.1 Introduction

Portland cement structural concrete is a popular material for marine applications. It has been used for the construction of sea defences, harbours, wharves, jetties, floating structures, defence forts and lighthouses, etc., for almost a century. In more recent times the development of the offshore oil industry has led to new marine applications such as concrete platforms and oil storage structures in the North Sea. These applications have extended the use of concrete to greater marine depths (100–150 m) than ever before.

The future potential for marine applications of the material is enormous with the inevitable growth in demand for the resources of the oceans and the possibility of locating cities, airports, nuclear power plants and various other facilities on offshore floating platforms in order to relieve land masses from urban congestion and pollution.

The popularity of concrete for marine construction is due to its economy for large structures and also to its generally excellent marine durability. The future applications of concrete in marine environments (coastal, offshore, estuaries) at great depths and in oceans with climatic extremes will stretch the durability of concrete to greater limits and, therefore, a sound understanding of the durability phenomenon is necessary to face this exciting challenge for the future successfully.

Considerable advances have been made in concrete technology in recent years and all these will assist in providing the right materials for specific marine applications of the future. The use of cement blends such as PFA, slag and micro silica has been proved to impart many desirable properties to concrete and, therefore, has become acceptable practice. The use of various admixtures such as superplasticizers, retarders and air entrainers has gained popularity in order to control the properties of fresh and hardened concrete. The ability to produce high-strength concretes with both normal- and lightweight aggregates will result in lighter and stronger marine structures which will be able to float if required. Concretes reinforced with fibres will have ductility, resistance to cracking and other superior properties which will make them suitable for aggressive environments which cause abrasion and impact.

The various types of deterioration processes to which concrete under water is exposed are described in this chapter. These include the chemical effects of deleterious substances and physical deterioration due to climatic extremes and aggressive exposure conditions. The properties of concrete and the composition of its hydration products, which control its durability under such conditions, are discussed. Prevention of deterioration by the choice of suitable composition of materials is outlined and the resulting properties of concrete are described.

# 6.2 Marine environment

## 6.2.1 Exposure zones

Concrete in a marine environment undergoes different deterioration processes depending on its position with respect to the tidal levels. From this standpoint, the marine environment can be divided into three principal zones as shown in Figure 6.1.[1]

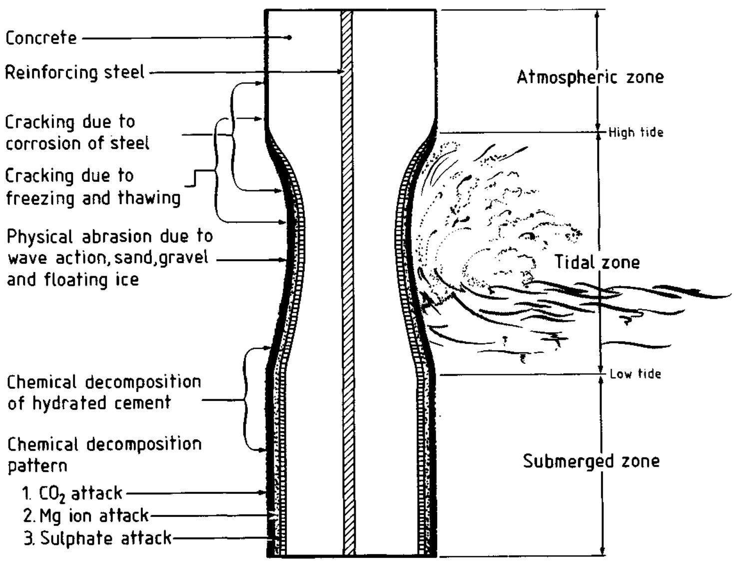



**Fig. 6.1** Marine exposure zones of concrete structures. (Mehta, P.K., American Concrete Institute, Durability of concrete in the marine environment)

- *Splash zone:*
  The zone exposed to the atmosphere above high tide level. The concrete is exposed to salt spray and cycles of wetting and drying. This zone is most prone to chloride-induced reinforcement corrosion resulting in cracking and spalling of the concrete cover. Freeze–thaw damage also occurs in this zone.
- *Tidal zone:*
  This is the zone between the low and high tide levels. The concrete undergoes wet and dry cycles of sea water exposure. Chloride-induced corrosion of reinforcement and frost action can cause cracking and spalling. Mechanical action of the waves with sand and gravel causes abrasion, resulting in loss of material. Chemical decomposition of Portland cement hydrate is also likely in this zone.
- *Submerged zone:*
  The part of the structure below the low tide level, which is always submerged in sea water, is primarily vulnerable to chemical attack by the salts in sea water on the products of hydration of Portland cement. This can result in strength reduction and loss of material. Sea water in this zone lacks dissolved oxygen to fuel corrosion of reinforcement and frost attack is also not a problem in this zone. The hydrostatic pressure caused by the depth of sea water, however, is likely to result in rapid diffusion of chloride into concrete.

### 6.2.2 Chemical composition of sea water

The salinity of sea water is generally about 3.5% of inorganic salts, the principal compounds being sodium chloride and magnesium sulphate. A summary of total salinity of the seas of the world is given in Table 6.1 and the ionic concentrations of the various salts in the seas are given in Table 6.2.[2]

The concentration of dissolved oxygen in sea water varies with temperature, depth and turbulence of the sea. It is approximately 10 ppm at the surface, decreasing to about 3 ppm at 100 m depth.[3] In comparison, the oxygen concentration in air is about 210 ppm. The low concentration of oxygen in sea water is the main reason for reinforcement corrosion being

**Table 6.1** Chloride content of sea water for various seas[2]

| Sea | Chloride content (ppm) | Total salinity (ppm) |
|---|---|---|
| North Sea | 16550 | 33060 |
| Atlantic Ocean | 20000 | 35537 |
| Mediterranean | 21380 | — |
| Arabian Gulf | 33660 | — |
| Persian Gulf | 23000 | 42750 |
| Baltic Sea | 3960 | 7110 |

**Table 6.2** Concentrations of soluble salts for various seas[2]

| | Concentration (g per 100 cm$^3$) | | | |
|---|---|---|---|---|
| Ion | North Sea | Atlantic Ocean | Baltic Sea | Persian Gulf |
| Sodium | 1.220 | 1.110 | 0.219 | 1.310 |
| Potassium | 0.055 | 0.040 | 0.007 | 0.067 |
| Calcium | 0.043 | 0.048 | 0.005 | 0.050 |
| Magnesium | 0.111 | 0.121 | 0.026 | 0.148 |
| Chloride | 1.655 | 2.000 | 0.396 | 2.300 |
| Sulphate | 0.222 | 0.218 | 0.058 | 0.400 |
| Total | 3.306 | 3.537 | 0.711 | 4.275 |

uncommon in submerged concrete. A typical case of unprotected steel corroding in sea water is shown in Figure 6.2.[4] This shows that the steel is most prone to corrosion in the splash zone where both $Cl^-$ and $O_2$ are freely available.

### 6.2.3 Temperature

The temperature of sea water has an important influence on the durability of concrete: high temperatures generally accelerate the deterioration process,[5] not only owing to their direct influence, but also to faster drying and salt accumulation caused by rapid evaporation in the splash zone. In cold oceans, such as the Arctic, surface temperatures of the sea water can be reduced due to atmospheric temperatures of below −50 °C, whereas water below the ice line remains at around 0 °C.[2] In temperate climates such as the North Sea, temperatures vary from about 16 °C near the surface in late summer to 6 °C in winter. In deeper water, below 50 m, the temperature variation is much smaller, in the range 6–9 °C for all seasons.[4]

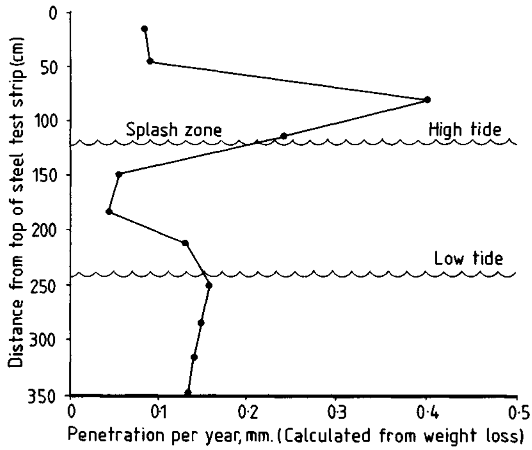



**Fig. 6.2** Corrosion rates of steel in marine exposure zones. (Sharpe, J.V., Review, The use of steel and concrete in the construction of north sea oil production platforms, Journal of Materials Science (1979) Chapman & Hall)

Warm oceans in the tropics can reach temperatures of nearly 30 °C in the surface zones, the temperature decreasing with depth to about 12 °C at 200 m depth.

### 6.2.4 Marine fouling

Concrete structures in a marine environment are normally covered with marine growth, the constitution and thickness of which depend on sea water composition, temperature and depth. Marine fouling causes degradation of the concrete surface and impedes subsequent repair to structure.[5] Marine organisms can be characterized into two broad categories, namely soft and hard fouling organisms.[5] Examples of soft organisms are sponges, sea squirts and sea weed. Hard fouling organisms include barnacles, mussels and tube worms. Growth of these organisms is influenced by light intensity, water temperature, dissolved oxygen concentration, depth, age of structural installation in the sea, geographical location, season, current velocities and surface characteristics of structures.[5]

In a marine durability survey of the 34-year-old Tongue Sands Tower in the English Channel, samples of marine growth were removed from both the tidal and submerged zones.[6] The distribution of growth was algae in the tidal zone, barnacles extending to below low tide level and mussels from the mid-tidal zone to the base of the tower at the sea level. The effect of marine growth on concrete durability appears to be insignificant. In fact, the sealing of the concrete surface by marine growth may act as a barrier to the diffusion of oxygen and chemical ions through the concrete cover, thereby preventing deterioration.[4,6]

## 6.3 Chemical attack

### 6.3.1 General

A reinforced concrete structure in the submerged zone of the marine environment is vulnerable to chemical reactions between sea water and the products of hydration of cement, which can result in material loss and strength reduction. Deterioration is also likely due to impact, abrasion and scour caused by floating ice and waves containing sand and gravel particles.

### 6.3.2 Hydrated Portland cement

The main products of hydration of Portland cement are vulnerable to decomposition by the aggressive components of sea water such as $CO_2$, $MgCl_2$ and $MgSO_4$. These products are formed by hydration of the dicalcium silicate ($C_2S$) and tricalcium silicate ($C_3S$) compounds of Portland cement which produce the two crystalline hydrates calcium hydroxide,

$Ca(OH)_2$, and tricalcium disilicate hydrate, $3CaO \cdot 2SiO_2 \cdot 3H_2O$ (or $C_3S_2H_3$).

Hydration of the tricalcium aluminate ($C_3A$) compound of ordinary Portland cement, in the presence of gypsum, produces a crystalline monosulphate hydrate, $3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 18H_2O$, which is responsible for the expansive reaction involving the formation of ettringite when hardened Portland cement comes in contact with sulphate-bearing waters. The monosulphate hydrate, of course, is not present in hydration products of sulphate-resisting Portland cements owing to the low $C_3A$ content of less than 3.5% permitted for such cements. Cements containing more than 10% by weight of $C_3A$, such as ordinary Portland cement, are particularly vulnerable to sulphate attack.

### 6.3.3 Carbon dioxide

The pH of sea water is normally about 8 and very small amounts of $CO_2$ dissolved from the atmosphere are present.[1] In the presence of decaying organic matter, however, $CO_2$ concentrations become high and the sea water becomes acidic with pH values of about 7 or less. In these conditions carbonation reactions with all the hydrated cement products can result in deterioration of concrete. Carbonation reactions can also occur in concrete exposed to underground waters or flowing and percolating waters with high $CO_2$ concentrations and low pH.

The possible chemical reactions of carbonation which may take place are as follows:[1,7]

$$CO_2 + Ca(OH)_2 \rightarrow \underset{\text{aragonite}}{CaCO_3} + H_2O \qquad (1)$$

In sea water containing large amounts of $CO_2$, aragonite is converted into calcium hydrogencarbonate, $Ca(HCO_3)_2$:

$$CO_2 + \{Ca(OH)_2 + 3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 18H_2O\} \rightarrow 3CaO \cdot Al_2O_3 \cdot CaCO_3 \cdot xH_2O + \underset{\text{gypsum}}{CaSO_4 \cdot 2H_2O} \qquad (2)$$

$$3CO_2 + 3CaO \cdot 2SiO_2 \cdot 3H_2O \rightarrow \underset{\text{aragonite}}{3CaCO_3} + 2SiO_2 \cdot H_2O \qquad (3)$$

As in reaction (1), aragonite is converted into calcium hydrogencarbonate in sea water containing sufficient $CO_2$.

Calcium hydrogencarbonate and gypsum produced by the above reactions are both soluble in water and, therefore, can be leached out of concrete. In consequence, loss of material, strength reduction or mushiness can result.

### 6.3.4 Magnesium salts

The typical $MgCl_2$ content of sea water is 3200 ppm, which is sufficient to cause deterioration of Portland cement hydrates due to $Mg^{2+}$ ion attack.[1] The calcium hydroxide hydrate reacts as follows to yield dense precipitates of brucite, $Mg(OH)_2$:

$$MgCl_2 + Ca(OH)_2 \rightarrow \underset{\text{brucite}}{Mg(OH)_2} + CaCl_2 \tag{4}$$

$CaCl_2$ is leached out of concrete, resulting in material loss and softening.

### 6.3.5 Sulphate attack

Solutions such as natural and polluted ground waters and sea water can cause sulphate attack on concrete. The sulphate ions from sea water react with the hydrates of Portland cement[1,7] resulting in deterioration of concrete. The following chemical reactions are possible during sulphate attack on cement hydrates in sea water:[1]

$$MgSO_4 + Ca(OH)_2 \rightarrow \underset{\text{brucite}}{Mg(OH)_2} + \underset{\text{gypsum}}{CaSO_4 \cdot 2H_2O} \tag{5}$$

The high concentration of NaCl in sea water increases the solubility of gypsum and prevents its rapid crystallization. It also increases the solubility of $Ca(OH)_2$ and $Mg(OH)_2$. The result is leaching of these compounds which makes the concrete weak.

$$MgSO_4 + \{Ca(OH)_2 + 3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 18H_2O\} \rightarrow Mg(OH)_2 + \underset{\text{ettringite}}{3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O} \tag{6}$$

$$MgSO_4 + \{Ca(OH)_2 + 3CaO \cdot 2SiO_2 \cdot 3H_2O\} \rightarrow 4MgO \cdot SiO_2 \cdot 8H_2O + \underset{\text{gypsum}}{CaSO_4 \cdot 2H_20} \tag{7}$$

The formation of ettringite can cause expansion and cracking, especially in land-based concrete and in laboratory experiment.[7] In marine environments, however, expansion and cracking are normally prevented owing to the solubility of ettringite in sea water. Ettringite together with gypsum can, therefore, be leached out of concrete. Another difference relative to land-based sulphate attack is that the presence of magnesium sulphate in sea water breaks down the structure of Portland cement paste by attacking the tricalcium disilicate hydrate of the cement.[8] As a result, marine sulphate attack leaves the concrete soft and brittle. The leaching of compounds and softening of concrete due to sulphate attack are further aggravated by the presence of chlorides in sea water. Sulphate attack in the submerged zone of marine concrete is slower than the higher zones where alternate wetting and drying accelerate the deterioration process.

Some of the chemical reactions which tend to weaken concrete in the marine environment have also been reported. $Mg^{2+}$ ion from magnesium sulphate can replace $Ca^{2+}$ ion from the tricalcium disilicate hydrate, ultimately forming magnesium silicate[9] which increases the porosity of concrete. A modified form of ettringite has also been observed in marine concrete which may cause deterioration in concrete.[10] This ettringite contains up to 5% $SiO_2$ and 0.2% chloride.

### 6.3.6 Acid attack by bacterial action

In the North Sea, concrete structures are used to contain stagnant saline waters in storage cells and platform legs of offshore structures. The crude oil in the storage tanks rests on the stagnant sea water. The temperature of the oil issuing from the production well is often between 70 and 100 °C and the temperature at the interface of the stagnant sea water and oil ranges between 45 and 50 °C.[11] At these high temperatures microbial activity occurs in the stagnant sea water, which is also contaminated by some organic matter.

Both aerobic and anaerobic conditions are considered to be necessary for this microbial activity, which results in the generation of substantial quantities of hydrogen sulphide at the oil/water interface.[12] Consumption of oxygen by the growth of aerobic bacteria in stagnant water generates carbon dioxide and reduces pH. This produces the conditions for sulphate-reducing bacteria (anaerobic bacteria) to grow and produce $H_2S$. Results of a two-stage experiment are given in Figure 6.3, which show a fall in pH during the first stage due to aerobic microbial growth and an increase in $H_2S$ production in the second stage of the experiment due to anaerobic sulphate-reducing bacterial activity.[12] Both low pH and $H_2S$ are potentially aggressive to reinforced concrete. $H_2S$ dissolves in the moisture present on the surface of concrete, producing sulphuric acid. The cement mortar

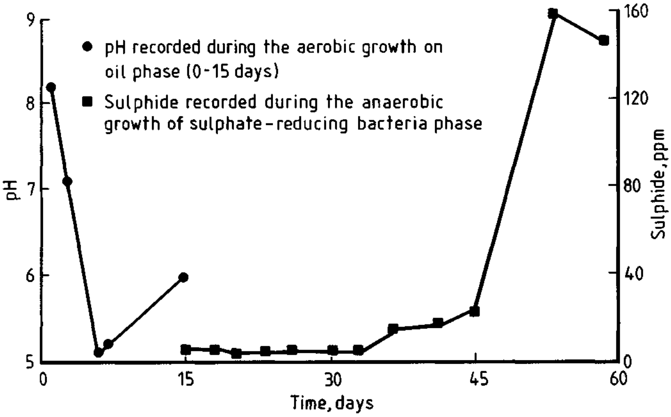



**Fig. 6.3** Loss of pH and increases in $H_2S$ due to aerobic and anaerobic bacterial activity.

$[Ca(OH)_2]$ is gradually dissolved by the $H_2SO_4$ resulting in progressive deterioration and mushiness of concrete.[13]

Production of hydrogen sulphide by sulphate-reducing bacteria and consequent concrete deterioration have also been reported in other oil–water systems such as crude oil carriers which contain only a small amount of water, resulting in pH values of 2–3.[11] The phenomenon of sulphate attack-reducing bacterial attack was first observed in concrete sewers, especially at fairly high temperatures, where the ultimate production of $H_2SO_4$ led to serious deterioration of concrete.[14] Cases have also been reported in the Middle East of structures retaining relatively stagnant waters where deterioration due to bacterial attack was evident. Resistance to such chemical attack can be enhanced by the use of Portland blast-furnace slag cement, addition of pozzolanic materials in concrete and using some recommended methods to fix $Ca(OH)_2$ in concrete.[15] Surface coatings can also be used to provide resistance to such attack.

## 6.4 Prevention of chemical attack

### 6.4.1 Influence of cement composition

The sulphate resistance of concrete can be improved by the use of sulphate-resisting cement which has a low $C_3A$ content. This is due to the absence of the monosulphate hydrate, $3CaO \cdot Al_2O_3 \cdot Ca \cdot SO_4 \cdot 18H_2O$, which is produced by the hydration of $C_3A$. It has been found in practice that a $C_3A$ content of about 7% is a rough boundary between cement of poor and good performance in sulphate-inducing environments. Sulphate resistance tests based on expansion measurements of mortar samples have been conducted which confirm the correlation between $C_3A$ content of cement and deterioration. Portland cements with a $C_3A$ content of less than 10% satisfactorily resist expansion under exposure to a sulphate solution[16] and to sea water.[17] Results of an investigation on French Portland cement immersed in sea water for 10 years are given in Figure 6.4, which relate the durability of mortar cubes to $C_3A$ content.[17] For $C_3A$ contents of approximately 10% or less most of the individual test results in Figure 6.4 achieve a durability ranking greater than 70, which indicates good to excellent performance. At higher $C_3A$ contents Figure 6.4 indicates poor marine durability of the concrete. $C_3A$ contents of 10–14% normally occur in ordinary Portland cements; in sulphate-resisting Portland cements the $C_3A$ content is normally limited to 3.5% or less, which on the basis of Figure 6.4 is satisfactory for marine applications.

A combination of $C_3A$ in excess of 13% together with high $C_3S$ contents leads to lower sulphate resistance, as shown by the expansion results in Figure 6.5.[18] This is likely to be due to correspondingly larger amounts of hydration products of $C_3S$, particularly $Ca(OH)_2$, which are liberated resulting in the formation of brucite, gypsum and ettringite which causes

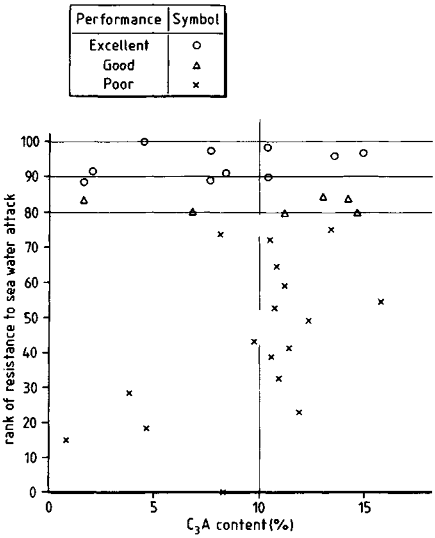



**Fig. 6.4** Influence of $C_3A$ content of cement on sea water attack. (Peltier, R., Resultats des essais de longue duree de la resistance des ciments a la mer, au laboratoire maritime de la Rochelle, La Revue des Materiaux (1970), Septima)

expansion. The need to limit the $(C_3A + C_3S)$ content is, therefore, apparent and the following empirical rule[18] can be adopted when $C_3S > 50\%$:

$$C_3A + 0.27C_3S \leq 23.5\%$$

The results in Figure 6.5 of nine different cements containing $C_3A$ between 0.7 and 10% and $(C_2S + C_3S)$ between 65 and 76% show small expansions with age. In these cements, the variation of $C_4AF$ between 13.5 and 23%

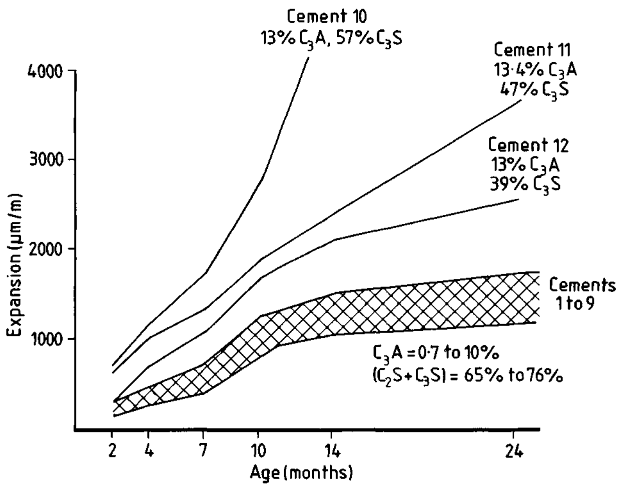



**Fig. 6.5** Influence of $(C_3A + C_3S)$ content of cement on expansion caused by sea water attack. (Regourd, M., American Concrete Institute, Physical-chemical studies of cement pastes, mortars and concretes exposed to sea water)

appears to have no significant influence on expansion. The ($C_3A + C_4AF$) content, however, was kept constant at 21–23.7%.

The fineness of cement influences the optimum gypsum content for cement to be used in a marine environment.[18] Cements with excessive amounts of gysum are prone to expansion and cracking in sea water. More finely ground cements can tolerate higher contents of gypsum since ettringite, which causes expansion and, therefore, disruption is distributed more homogeneously and hence stresses can be more easily absorbed.[18]

## 6.4.2 Influence of blended cements

Cement blending materials such as PFA and ground granulated blast-furnace slag (GGBS) have been used in marine applications and generally improve the marine durability of concrete. More recently condensed micro silica (silica fume) as a cement blend has been used in concrete and its marine durability needs to be established.

### 6.4.2.1 PFA cements

The resistance of concrete to sulphate attack in a marine environment can be increased by the introduction of a pozzolanic material such as PFA. Most low-calcium PFAs are particularly beneficial against sulphate attack. The PFA can be additional to the cement or it may be used as a partial replacement for cement. The principal compounds of PFA are silica ($SiO_2$), alumina ($Al_2O_3$), iron oxide and calcium oxide.

The $Ca(OH)_2$ liberated during the hydration of Portland cement reacts with the pozzolana to yield cementitious hydrates similar to those produced by Portland cement. $Ca(OH)_2$ is consumed at different rates by different pozzolana, PFA is slower and significant reactivity starts only after about 28 days of curing.[18] Satisfactory early curing, therefore, is important. This consumption of $Ca(OH)_2$ leads to improved sulphate resistance, which can be particularly high when pozzolana is used with sulphate-resisting cement.[19] However, an adequate amount of pozzolana is required in this case otherwise a reaction with sulphates is possible in the longer term. Consequently, the use of PFA with sulphate-resisting Portland cement is not permitted in some Codes of Practice. Performance tests must be conducted to determine the suitability and amount of PFA to be used with cement.

Heavy damage due to alkali–aggregate reactions has been observed in some marine structures. The reaction is affected by the alkali content of cement, the reactivity of aggregates and the curing and environmental conditions. The penetration of chloride from sea water also accelerates the alkali–aggregate reaction. Replacement of cement with PFA generally prevents the deleterious alkali–silica reactions; the amount of PFA required increases with the amount of reactive aggregate. Low-calcium PFAs are effective in reducing expansion due to alkali–silica reactions at cement replacement levels of 25–30%;[20] higher levels of cement replacement are needed with high-calcium PFAs.

### 6.4.2.2 Blast-furnace slag cements

In Western Europe, especially Germany, The Netherlands and Belgium, Portland cement blended with ground granulated blast furnace slag contents greater than 65% is more commonly used than sulphate-resisting cement for marine applications.[21] Portland blast-furnace slag cement is richer in $Al_2O_3$ and contains less CaO than Portland cement. In slag-blended cements, the Portland cement is the first to hydrate, releasing $Ca(OH)_2$ by the hydration of $C_2S$ and $C_3S$. This activates the slag to produce similar hydrates as Portland cement.[18] The best sulphate resistance is achieved at slag contents greater than about 70%. At such slag contents, the formation of ettringite is impossible owing to the low content of free $Ca(OH)_2$ in the cement paste;[21] no $Ca(OH)_2$ can exist in a blended cement containing more than 80% slag, as shown in X-ray diffraction diagrams.[18] While the sulphate resistance increases with increasing slag content, the effect on mechanical strength can be the opposite. A finer grinding of the blended cement will partly overcome poor mechanical properties, but the main solution is to ensure the use of slags which are in a vitreous state. This stage is achieved by quenching the molten slag running from the blast furnace.[21]

A study has recently been reported on a range of concretes containing Portland cements and ground glassy blast furnace slags which are exposed to tidal and submerged environments at a marine exposure site.[22] The most durable concrete in both tidal and full immersion zones was that containing ordinary Portland cement of medium $C_3A$ content (8.8%). The GGBFS concretes at 60% and 70% replacement levels also had good chemical resistance to sea water attack. They were, however, vulnerable to frost attack in the tidal zone, but frost attack was completely absent in submerged concrete. Concretes made with sulphate-resisting Portland cement with low $C_3A$ content (0.57%) and with ordinary Portland cement of high $C_3A$ content (14.1%) suffered greater chemical attack under submerged conditions. These results generally support the recommendations in BS 6349 Part 1, 1989,[23] which limit the $C_3A$ content of cements placed in UK waters to less than 10%. In the USA, ASTM C-150, Type II cement or Type III cement with the optional 8% $C_3A$ invoked is generally specified for sea water exposure. However, the 8% limit on $C_3A$ may be increased to 10% if the water/cement ratio of the concrete is kept below 0.45 and the concrete will be permanently submerged in sea water. A further limit to the minimum $C_3A$ content of not less than 4% is also recommended to prevent chloride-induced corrosion of reinforcement. The use of cement replacement materials is also recommended either with 25% replacement of Portland cement with PFA or with at least 70% replacement of Portland cement with GGBS.

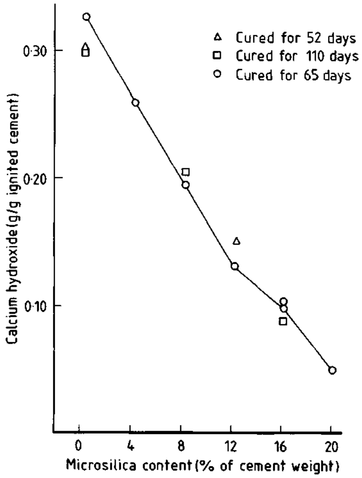



**Fig. 6.6** Effect of microsilica content on alkalinity ($CaOH_2$ content) of hydrated cement. (Sellevold, E.J. *et al.*, Silica fume-content pastes: hydration and pore structure, Norwegian Institute of Technology)

### 6.4.2.3 Micro silica concrete

Condensed micro silica (or silica fume) is a by-product of the production process for silicon metal and ferrosilicon alloys. It can be used in concrete as a cement replacement or as an addition to improve properties of the material. Micro silica generally shows high pozzolanic reactivity.[24] The $Ca(OH)_2$ content of hydrated cement decreases with increasing dosage of micro silica as shown in Figure 6.6, elimination of $Ca(OH)_2$ occurring at 24% micro silica content. Micro silica also acts as a filler and distributes the hydration products in a homogeneous manner. This results in a refined microstructure so that gel particles are not distributed individually as in ordinary cement paste but are combined a dense structure which extends to the aggregate boundary.[25] There is contradictory evidence, however, that the pore structure can be coarser in concrete blended with micro silica under different curing conditions.[26]

The sulphate resistance of concrete is improved by the addition of micro silica. Results of a field study[27] are given in Figure 6.7 which show a marked increase in sulphate resistance with 15% micro silica. In addition, micro silica can also be used to control alkali–aggregate reactions in concrete,[28] which can occur, for example, when reactive aggregates are used in marine structures. Reasons for the good durability performance of concretes containing micro silica are their low permeability, which reduces the diffusion rates of harmful ions,[29] the lower $Ca(OH)_2$ content that can be attacked and leached out by harmful chemicals and the increased amount of aluminium, which reduces the amount of alumina available for the production of ettringite.

The alkalinity of pore fluid in cement pastes decreases with increasing

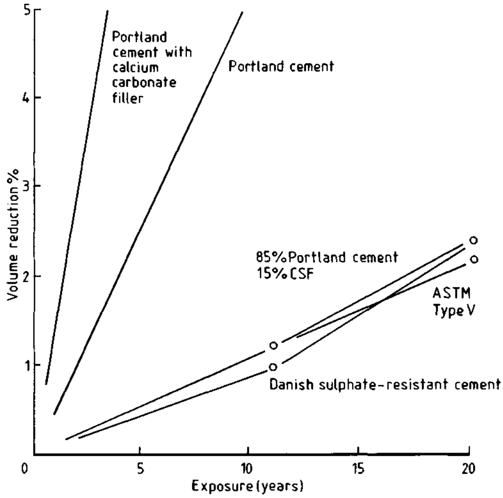



**Fig. 6.7** Influence of microsilica on sulphate resistance of concrete. (Fiskea, O.M., Betong i alkunskifer, Norwegian Geotechnical Institute)

micro silica content, as shown in Figure 6.8,[30] and stabilizes after about 3 months.[31] This could increase the risk of carbonation and reduce the protective effect of the alkaline electrolyte against reinforcement corrosion. However, the impermeable microstructure of micro silica concrete will resists the diffusion processes of $CO_2$ and $Cl^-$ and, therefore, the net effect on corrosion due to micro silica addition may not be harmful.

## 6.4.3 Influence of initial curing

Initial curing provided to concrete soon after casting has a profound influence on the porosity and pore structure of the hydrated cement matrix.[26] Although the pore volume and pore structure appear to bear no

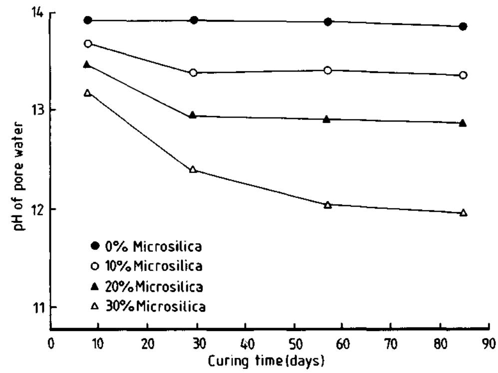



**Fig. 6.8** pH versus curing time at different microsilica contents. (Page, C.L., Vennesland. Pore solution composition and chloride binding capacity of silica fume cement pastes, Materials & Structures, 1983, DUNOD)

direct relationship with the sulphate resistance of concrete,[32] their influence on other aspects of durability is likely to be important.

The sulphate resistance of concrete as affected by initial curing at different temperatures and relative humidities (RH) has been investigated, including curing conditions simulating hot, arid climates such as in the Middle East.[32] Initial moist curing of concrete (approximately 100% RH) results in lower sulphate resistance than initial air curing at low (25% and 55%) RH. This effect of initial curing is equally evident in plain concrete and in concretes blended with PFA, GGBFS and silica fume. The presence of a carbonated layer on the concrete surface is generally accompanied by superior sulphate resistance.[22] A greater extent of carbonation, however, is not necessarily followed by greater sulphate resistance. For example, low-humidity curing at high temperature (45 °C) results in higher depths of carbonation but the sulphate resistance is lower than with similar curing at 20 °C.[32]

The sulphate resistance of concrete increases with the replacement of cement with 22% PFA, 9% silica fume and 80% GGBFS. The sulphate resistance also increases owing to drying out of concrete during early curing at low relative humidity and to carbonation.[32] The possible common factor which results in improved sulphate resistance due to all these different causes is the reduced $Ca(OH)_2$ content in the surface layers. It is well established that $Ca(OH)_2$ is an essential ingredient of the sulphate attack reaction which produces the expansive product, ettringite, which is mainly responsible for deterioration.[1,7] Lower $Ca(OH)_2$, therefore, promises higher sulphate resistance.

The use of PFA (22%), silica fume (9%) and GGBFS (80%), for partial replacement of ordinary Portland cement, results in higher sulphate resistance than for plain concrete under initial curing conditions simulating hot, arid climates such as in the Middle East. The effect of initial curing at high temperature (45 °C) is significantly harmful to the sulphate resistance of plain concrete but much less harmful to these blended cement concretes.[32]

## 6.5 Resistance to penetration of deleterious substances

### 6.5.1 General

Although products of hydration of hydraulic cements and cement blends are vulnerable to chemical attack by sea water, this only becomes a reality if the aggressive ions from the sea water can penetrate the concrete. High-density and impermeable concretes can be made with a suitable choice of materials and mix design, which can control durability to a significant extent and overshadow the importance of chemical factors. An appreciation of the properties of concrete which control the penetration of the environment is, therefore, important in a durability study. The deterioration processes which take place in underwater concrete are due to the presence of sulphate ions, $CO_2$ and, to a lesser extent, chloride ions. The factors affecting the penetration of these ions in concrete are considered in the following sections.

## 6.5.2 Diffusion of carbon dioxide

The rate of diffusion of gases in concrete is strongly dependent on the degree of moisture saturation,[33] the rate of diffusion being lower with increasing relative humidity of curing. The diffusion of $CO_2$ through water is about $10^4$ times slower than in air and the rate for $O_2$ is about $10^5$ times slower. Maximum rates of carbonation occur under relative humidity ranging between 30% and 90%.[34] The depth of carbonation ($x$), which is commonly measured by the phenolphthalein indicator test, is related to the exposure time ($t$) roughly by a parabolic relationship of the form $dx/dt = Kt^{0.5}$ (Ref. 35). Various other relationships have also been proposed[34,36] which are of the form $x = D\sqrt{t}$.

The proportionality constant, $K$, or the diffusion coefficient, $D$, depends on parameters associated with concrete quality such as water/cement ratio and environmental conditions such as relative humidity and temperature.[36] A limitation of the phenolphthalein indicator test is that it only detects fully carbonated concrete which has a pH of less than 9 and does not indicate the degree of partial carbonation. In concrete exposed to tidal and submerged zones of the marine environment, high moisture saturation prevents to a significant extent full carbonation. However, partial carbonation can occur which, together with leaching of $Ca(OH)_2$, lowers the pH of the pore fluid near the concrete surface as shown in Figure 6.9.[37] The

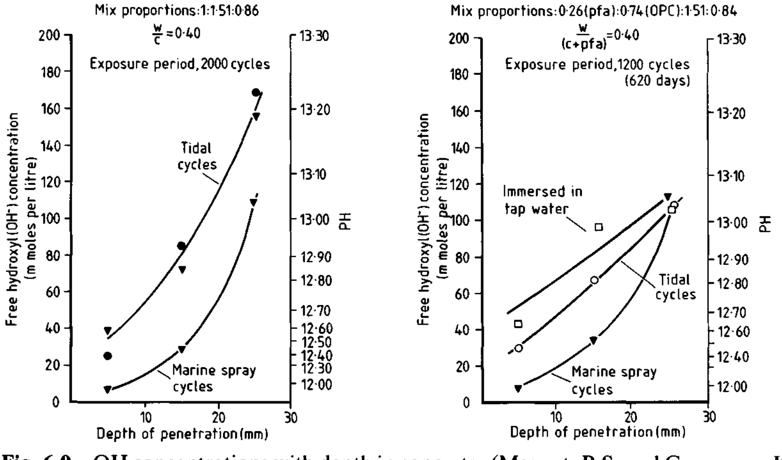



**Fig. 6.9** OH concentrations with depth in concrete. (Mangat, P.S. and Gurusamy, K. Pore fluid composition under marine exposure of steel fibre reinforced concrete, Cement and Concrete Research, Pergamon Press Ltd.)

depression of pH is aggravated by higher concentrations of chloride ions in the concrete. For example, for concrete mixes represented in Figure 6.9, the $Cl^-$ concentration was maximum in concrete exposed to simulated marine cycles, less under tidal zone exposure and minimum (zero) in specimens cured in laboratory water;[37] the corresponding reduction in pH is maximum in samples with greatest $Cl^-$ concentration.

### 6.5.3 Diffusion of chloride

The rate of $Cl^-$ penetration in concrete is represented by Fick's law of diffusion,[38] which can be expressed as

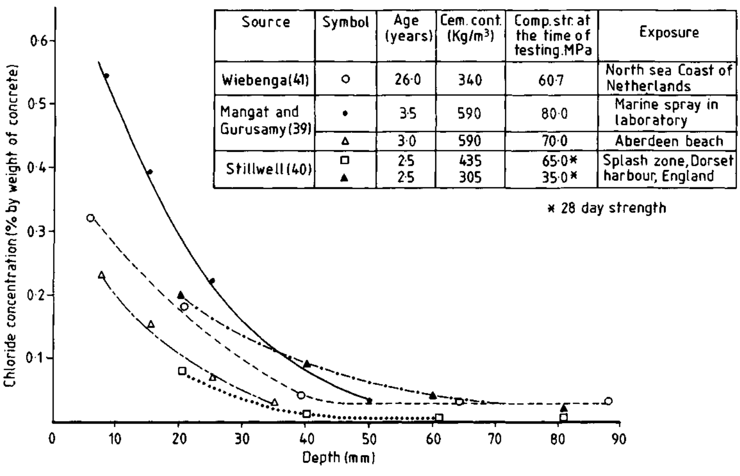
$$\frac{\partial c}{\partial t} = D_c \frac{\partial^2 c}{\partial x^2}$$

where $c$ is the $Cl^-$ concentration at a depth $x$ after time $t$ and $D_c$ is the diffusion coefficient. A standard solution of this partial differential equation is

$$C_{(x,t)} = C_o\left[1 - \operatorname{erf}\left(\frac{x}{2\sqrt{D_c t}}\right)\right]$$

where $C_o$ = equilibrium $Cl^-$ concentration of concrete surface and erf = error function.

Typical chloride diffusion curves under marine exposure obtained by various researchers are shown in Figure 6.10.[39] There is considerable similarity between these diffusion curves despite the different ages of exposure, $Cl^-$ extraction procedure and mix proportions used by the different investigators. The curve representing simulated marine exposure




| Source | Symbol | Age (years) | Cem. cont. (Kg/m³) | Comp. str. at the time of testing. MPa | Exposure |
|---|---|---|---|---|---|
| Wiebenga (41) | ○ | 26·0 | 340 | 60·7 | North sea Coast of Netherlands |
| Mangat and Gurusamy (39) | • | 3·5 | 590 | 80·0 | Marine spray in laboratory |
| | △ | 3·0 | 590 | 70·0 | Aberdeen beach |
| Stillwell (40) | □ | 2·5 | 435 | 65·0* | Splash zone, Dorset harbour, England |
| | ▲ | 2·5 | 305 | 35·0* | |

**Fig. 6.10** Cl-diffusion profiles in concrete under marine exposure. (Mangat, P.S. and Gurusamy, K. Chloride diffusion in steel fibre reinforced marine concrete, Cement and Concrete Research, Pergamon Press Ltd.)

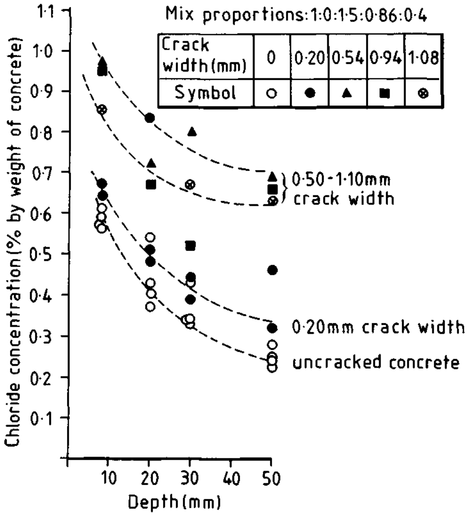



**Fig. 6.11** Cl-diffusion profiles in cracked concrete. (Mangat, P.S. and Gurusamy, K. Chloride diffusion in steel fibre reinforced concrete containing pfa, Cement and Concrete Research, Pergamon Press Ltd.)

in the laboratory, however, shows significantly higher $Cl^-$ concentrations owing to evaporation in laboratory air resulting in concentrated sea water.

In cracked concrete, $Cl^-$ concentrations at different depths increase with increasing crack widths, as shown in Figure 6.11.[42] In submerged concrete, the $Cl^-$ diffusion process is further influenced by the hydrostatic pressure and the rate of sea water penetration is given by the following expression:[43]

$$K = \frac{d^2 v}{2ht}$$

where $K$ = permeability coefficient, $d$ = depth of saturation, $v$ = void content of concrete, $h$ = hydrostatic head and $t$ = time.

Chloride penetration into concrete occurs quickly even in high cement content, low water/cement ratio concretes, most of the penetration occurring within 6 months of exposure.[39,40] The values of the diffusion coefficients, $D_c$, at early ages are high and reduce sharply with longer term exposure.[39] Values ranging between $0.1 \times 10^{-8}$ and $10.0 \times 10^{-8}$ cm$^2$/s have been reported for a wide range of marine concretes of different materials (also including PFA), mix proportions and age.[39,42,44,45]

Partial replacement of cement with PFA reduces chloride diffusion rates into concrete. Laboratory-based tests using diffusion cell experiments on Portland cement/PFA pastes which were exposed to extended initial curing in fresh water or $Ca(OH)_2$ solutions showed very large reductions in chloride ingress due to PFA addition.[31,33] The reductions were, however, much more modest under practical conditions of initial curing provided to concrete specimens which were subsequently exposed to a marine environment.[42] Partial replacement of cement with GGBS also results in a

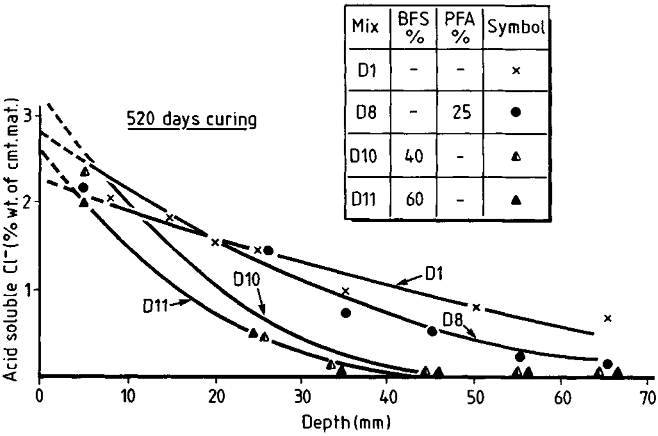



**Fig. 6.12** Cl-diffusion profiles in concretes containing cement replacement materials pfa, slag (Molloy, B.T. Steel fibre and rebar corrosion in concrete under marine curing, Aberdeen University)

significant reduction in chloride penetration, especially in the longer term and at depths of several centimetres into concrete.[46]

Chloride concentration profiles with depth from the surface of concrete are shown in Figure 6.12 for concretes made with ordinary Portland cement and with cement replacement by PFA or slag. The water/cementitious materials ratio is constant at 0.58.[47] It is clear that both PFA and slag are effective in reducing chloride penetration at greater depths in concrete but are prone to higher chloride concentrations near the surface. The need to provide adequate cover to steel reinforcement in PFA and slag concretes is obvious.

The great resistance to $Cl^-$ diffusion imparted by micro silica concrete is evident from the results in Figure 6.13.[48] The effectiveness of micro silica is

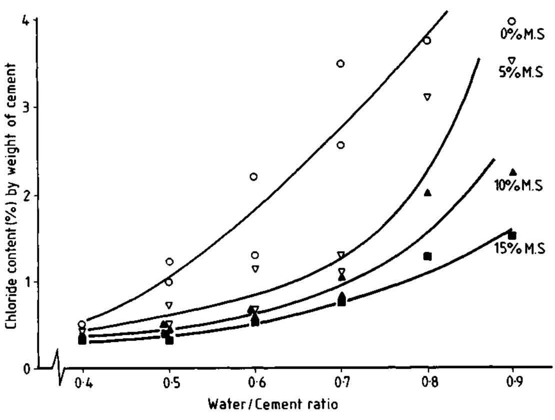



**Fig. 6.13** Relationship between chloride content and water/cement ratio for concrete containing 0, 5, 10 and 15% microsilica.

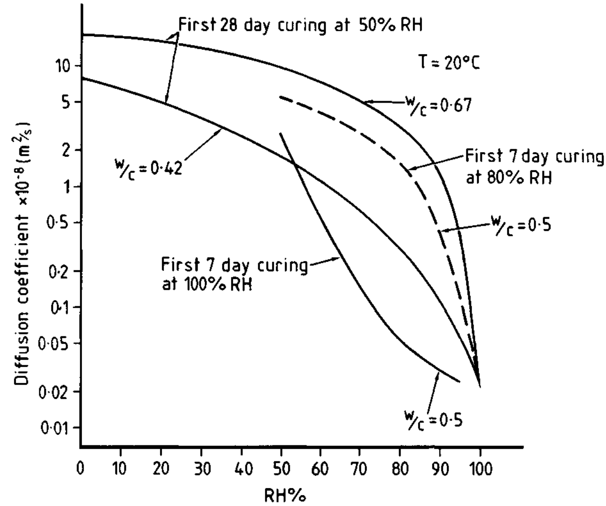



**Fig. 6.14** Effect of moisture on $O_2$ diffusion in concrete. (Tuuti, K. Corrosion of steel in concrete, Swedish Cement & Concrete Institute)

more pronounced at low cement contents and becomes relatively insignificant at cement contents of 450 kg/m$^3$.

## 6.5.4 Diffusion of oxygen

Observations on reinforced concrete marine structures indicate that for the continuously submerged parts of a structure, steel corrosion is almost negligible.[6,40] Since the concentrations of chlorides in concrete submerged in sea water will exceed critical levels and low electrical resistivity of saturated concrete is conducive to corrosion, this absence of corrosion can be attributed to the fact that diffusion of oxygen in saturated concrete is an extremely slow process.[33] The influence of moisture state on the rate of oxygen diffusion is shown in Figure 6.14 for concrete specimens which were conditioned to different relative humidity environments for at least 6 months.[33] The results show that changing the moisture state from 50% to 100% relative humidity reduces the permeability of concrete by at least a factor of 100. Figure 6.14 also shows that moist curing during the first 7 days is of critical importance for oxygen penetration in concrete. High water/cement ratio concretes have higher oxygen diffusion rates; in Figure 6.14, at a relative humidity of 50%, a minimum of a fivefold increase in permeability is observed when the water/cement ratio is increased from 0.42 to 0.67. The importance of the near-surface zone for controlling durability is evident in Figure 6.15, which shows a reduction in oxygen diffusion with increasing depth of cover.[33] This will be due to the drier moisture state of the surface layers of concrete due to the dry curing environment of 20 °C and 50% RH.[33]

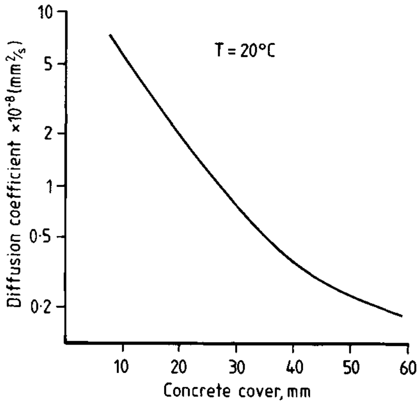



**Fig. 6.15** Relation between oxygen diffusion coefficient and depth of concrete. (Tuuti, K. Corrosion of steel in concrete, Swedish Cement & Concrete Institute)

The immunity imparted to steel corrosion in concrete submerged in sea water does not extend to stainless steel.

## 6.5.5 Permeability of concrete

Permeability of concrete is possibly the most important property which determines its long-term marine durability. Impermeable concrete can be produced by adopting mix design recommendations for marine concrete, using high cement contents of about 400 kg/m$^3$ and a low water/cement ratio of about 0.4.[49] A low water/cement ratio is a more important parameter than a high cement content and its influence on the permeability coefficient is shown in Figure 6.16,[50] which combines the permeability coefficient data from various sources.

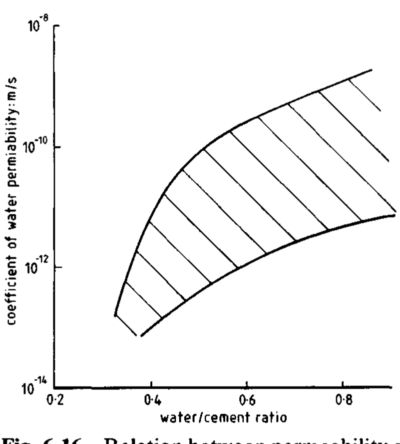



**Fig. 6.16** Relation between permeability and w/c ratio. (Bamforth, P.B. The water permeability of concrete and its relationship with strength, Magazine of Concrete Research, Thomas Telford Publications)

High-quality aggregates of maximum size 20 mm are desirable since the permeability of concrete is dependent primarily on the permeability of cement paste and aggregate, the paste being more dominant since it embeds the aggregate particles in a fully compacted concrete. High-quality granite aggregate, for example, has a similar permeability coefficient to mature cement paste with a water/cement ratio of 0.7. A water-reducing admixture is desirable to provide good workability for efficient compaction and a blending material such as PFA or slag can be used to control thermal cracking and to provide resistance to sulphate attack. Such mixture design results in high-strength concrete of grade 60 N/mm$^2$ or more, which has an extremely low permeability coefficient of less than $10^{-11}$ m/s.[51] With such low permeability, the chemistry of hydration products of cement is likely to have relatively little effect on marine durability.

The permeability of marine concrete in the submerged (underwater) zone is further reduced under long-term exposure to sea water owing to the deposition in the voids and microcracks of products of chemical interaction between sea water and Portland cement hydrates.[52] Examples of these products include brucite, aragonite and ettringite, which are formed by chemical reactions outlined earlier. The chlorides which diffuse into concrete also fill small pores in the concrete, thereby further reducing permeability.

Proper curing is essential to achieve low permeability of concrete and this requirement becomes even more critical if blends such as slag and PFA are used.[42,50] An appreciation of the minimum initial moist curing period necessary can be gained from the oxygen permeability results given in Figure 6.17 for concrete disc specimens which were wet cured for up to 72 h followed by curing at 50% RH and 20 °C for several months until a steady state of hydration had been reached.[53] Concretes based on ordinary Portland cement only and blends with PFA and slag were used and the results in Figure 6.17 show that short wet curing periods for blended

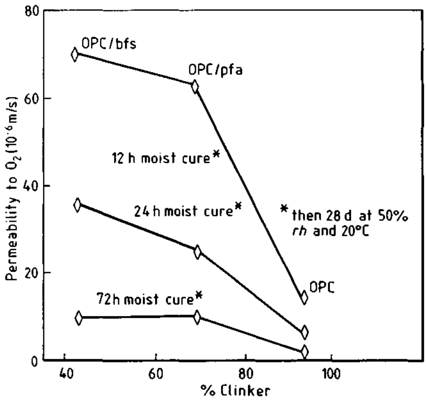



**Fig. 6.17** Relation between oxygen permeability and clinker content (%) after different periods of initial moist curing. (Pomeroy, D., American Concrete Institute. From basic research to practical reality)

cements caused unacceptably large increases in oxygen permeability. Ordinary Portland cement-based concretes are less affected by shorter periods of initial moist curing. For underwater concrete, the need for moist curing should be satisfied more easily and, therefore, limitations on the use of blended cements are unlikely on these grounds.

Owing to the difficulties associated with testing the permeability of concrete, limited data are available on this property. A recent report gives the following typical values of the cofficient of water permeability for high, average and low permeability concretes:[54]

| | |
|---|---|
| High permeability | $>10^{-10}$ m/s |
| Average permeability | $10^{-12}$–$10^{-10}$ m/s |
| Low permeability | $<10^{-12}$ m/s |

## 6.6 Corrosion

The principles of reinforcement corrosion in concrete are dealt with in considerable detail in recent publications and are covered comprehensively in a recent Rilem report[36] and an ACI report.[55] The pore fluid in a concrete matrix is the electrolyte for reinforcement corrosion. The high alkalinity of the pore fluid in normal concretes provides passivity to steel reinforcement against corrosion. This passivity, however, can be broken if the concrete surrounding the reinforcement undergoes carbonation and thereby loses its high pH. Under marine exposure, passivity of reinforcement can also be lost by the ingress of high concentrations of chloride ions at the steel surface, which can initiate pitting corrosion. Following the breakdown of passivity, the corrosion reaction is fuelled by a continuing supply of oxygen and moisture at the corrosion cell, which consists of a cathodic and anodic region on a reinforcement bar. The rate of corrosion is higher in concretes of low resistivity. In reinforced concrete marine structures, the splash and tidal zones are particularly vulnerable to reinforcement corrosion owing to the high concentrations of chloride ions and free availability of oxygen and moisture. In permanently submerged structures, the general corrosion rates can become negligible owing to the lack of dissolved oxygen in deep water. Interaction of reinforcement in the submerged and tidal zones of concrete structures, however, can result in accelerated localized corrosion at cracks or other damaged areas. This is due to the sea water in fully submerged concrete providing an extremely low resistance electrolyte which allows the formation of a macro-cell corrosion activity between a damaged area and the extensive cathode of reinforcing steel in the undamaged concrete.[56] Protection against such corrosion can be provided by connecting the reinforcement in the submerged concrete to a system providing cathodic protection for any underwater steelwork attached to the concrete structure. Such cathodic protection is commonly provided for most offshore structures in the North Sea.[57]

Reinforced concrete structures submerged in stagnant water are also vulnerable to reinforcement corrosion owing to the action of sulphate-reducing bacteria as described in Section 3.5. The sulphate-reducing bacteria cause a reduction in pH and produce $H_2S$, both of which are aggressive to reinforced concrete. The resulting $H_2SO_4$ attack on the concrete produces a porous and disintegrated matrix. Corrosion of steel embedded in highly porous concrete has been observed under such exposure conditions[56,58,59] and approximate estimates suggest significant corrosion rates of up to 0.75 mm/year. In addition to stagnant water conditions, such corrosion can also be caused by marine fouling, bacterial activity in sediments, dumped cuttings of drilling and discharged production water.[60]

## 6.7 Physical deterioration

### 6.7.1 General

The physical actions which can cause deterioration of marine concrete in the underwater/splash zone are abrasion/erosion, impact and freeze–thaw cycles. Abrasion/erosion is caused by the tidal action of waves carrying gravel and sand in suspension. In addition, abrasion due to ice is a serious problem for concrete structures in cold oceans. Impact damage can be caused by floating ice and by accidental impact with ships, dropping objects and debris. Frost damage together with ice abrasion is likely in the cold oceans.

### 6.7.2 Abrasion, erosion and cavitation

The abrasion resistance of concrete is greatly influenced by the pore structure of the cement matrix in the outermost few millimetres of the surface zone.[61] The pore structure is influenced by the water/cement ratio and surface finishing procedures. Marine concrete exposed to abrasion attack requires hard and strong aggregates with a relatively high coarse/fine aggregate ratio such as 2:1. Since in the surface zone the coarse aggregate is covered by the mortar matrix, the quality of fine aggregate also needs to be sharp and strong. In conditions of severe abrasion/erosion, the interfacial bond between coarse aggregate and mortar matrix usually fails, causing the particles to pull out.[52] A very high strength concrete mix incorporating jasper or quartz diorite aggregate, micro silica and superplasticizers shows the same order of abrasion resistance as granite stone.[62]

The abrasion resistance of different concrete materials has been measured[63] by the abrasive action of steel balls which were mobilized by a jet of water on the concrete face at a velocity of 1.8 m/s. The results in Figure 6.18 show that after 72 h of exposure the weight loss of conventional

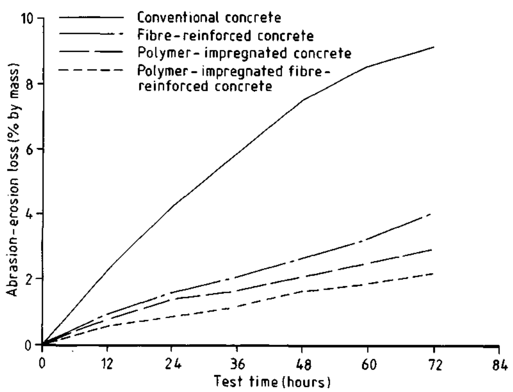



**Fig. 6.18** Abrasion resistance of different types of concretes. (Scanlon, J.M., Jnr, American Concrete Institute, Applications of concrete polymer materials in hydrotechnical construction)

concrete was about 9% compared to 4–2% for steel fibre-reinforced and polymer-impregnated concrete.[64]

A comprehensive review on the resistance of concrete to ice abrasion and impact[65] gives the latest information on test methods and abrasion resistance of various concrete mixes which are suitable for marine applications. These mixes include lightweight concretes which would be suitable for floating structures and concretes with fibre reinforcement, cement blending materials and air entrainment. The results of laboratory abrasion tests are given in Figure 6.19 for these mixes, for both normal- and lightweight aggregate concretes of compressive strengths exceeding 44 N/mm$^2$.[66]

## 6.7.3 Frost action

The degree of saturation has a critical influence on the frost resistance of concrete. Dry concrete can withstand freezing and thawing indefinitely, but concrete saturated with water can be severely damaged by a few cycles of freezing and thawing, even when it contains entrained air. Disruption is caused by the repeated freezing and, therefore, expansion of water in the pores of concrete. The limiting volume of water in the concrete pore structure which on freezing causes bursting pressure within the concrete is the critical saturation point. Below the 'critical' saturation point, a concrete is highly resistant to frost;[67] the critical saturation point depends on the size of concrete mass, its homogeneity and the rate of freezing.

Beyond the critical saturation point freezing water in the cavities expands and causes disruption unless space is available close to the cavity to relieve the pressure; this is the basis of air-entrained concrete which provides enhanced frost resistance to concrete. Air-entraining agents are used to distribute uniformly small bubbles of air, approximately 0.5 mm in size, in the concrete. The spacing between the bubbles is the critical factor

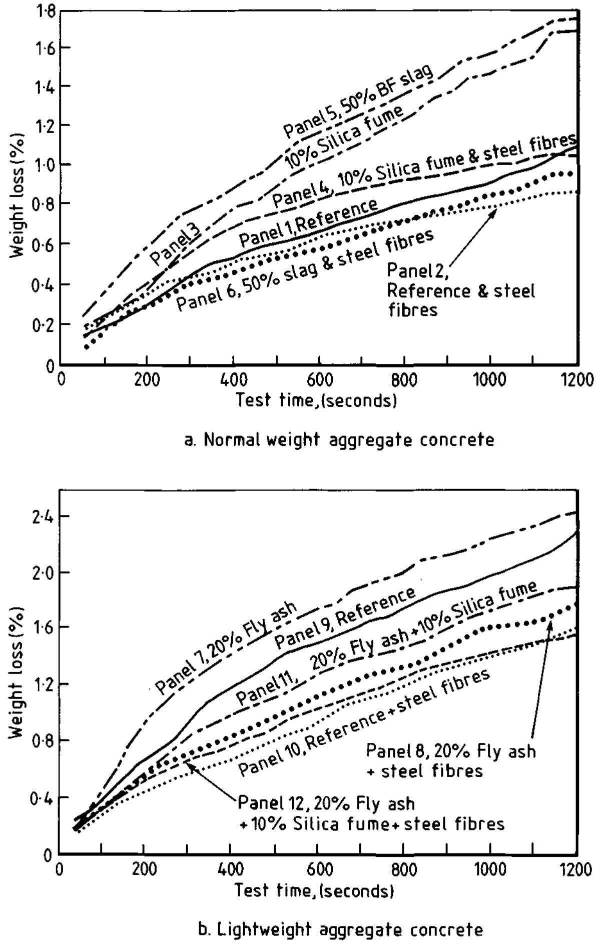



**Fig. 6.19** Abrasion resistance of different types of concrete. (CANMET Contract File No. 23440-6-9003)

which provides freeze–thaw resistance; a spacing of 0.2–0.25 mm is considered to provide full protection from frost damage.[68] The air entrainment content normally recommended to achieve satisfactory frost resistance ranges between 4 and 7%.

The second primary factor which controls frost resistance is the pore structure of cement paste. Frost resistance can be enhanced by mixes of low water/cement ratio which have small capillaries and little freezable water.[15] This is important for marine concrete where high-strength mixes and high cement content and low water/cement ratio are normally used. For such low water/cement ratio concretes air bubble spacing due to air entrainment can be increased to achieve the same durability factor as high water/cement ratio concretes, as shown in Figure 6.20.[69] Frost resistance studies on high-strength concretes containing micro silica[70] show that after

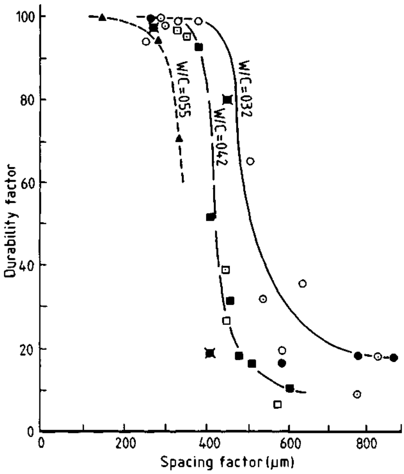



**Fig. 6.20** Freeze-thaw durability of concrete as affected by w/c ratio and air void spacing factor. (Philleo, American Concrete Institute, Frost susceptibility of high-strength concrete)

proper curing such concretes can develop a pore structure with the absence of freezable water, resistance to saturation and consequent immunity to frost attack. At early ages, however, the addition of micro silica hinders durability. The freeze–thaw resistance of non air-entrained concretes incorporating PFA or GGBS is generally lower than the reference Portland cement concretes. With the provision of air entrainment, however, the blended cement concretes show better frost resistance.

Another important factor with respect to marine concrete is the effect of chlorides on frost resistance. The results in Figure 6.21 show significant reduction of freeze–thaw resistance with increasing chloride concentration in concrete.[71]

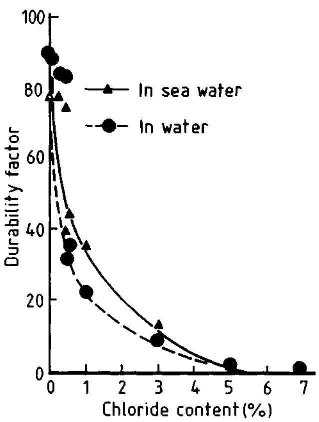



**Fig. 6.21** Influence of chloride concentration in concrete on freeze-thaw resistance. (Yamato, T., Emoto, Y. and Soeda, M., American Concrete Institute, Freezing and thawing resistance of concrete containing chloride)

# References


1 Mehta, P.K. (1980) *Durability of Concrete in the Marine Environment—a Review*. ACI Publication SP-65, pp. 1–20, American Concrete Institute, Detroit, MI.
2 Sverdrop, H.V., Johnson, M.W. and Fleming, R.H. (1946) *The Oceans, Their Physics, Chemistry and General Biology*. Prentice Hall, Englewood Cliffs, NJ.
3 Myers, J.J., Holm, C.H. and McAllister, R.F. (Eds) (1969) *The Handbook of Ocean and Underwater Engineering*. McGraw-Hill, London.
4 Sharp, J.V. (1979) Review, the use of steel and concrete in the construction of North Sea oil production platforms. *J. Mater. Sci.*, **14**, 1773–1799.
5 Haywood, M.G. (1980) Marine growth and its removal, in *Conference on the Inspection, Maintenance and Repair of Concrete Under Water*, p. 5. Cement and Concrete Association, London.
6 Taywood Laboratories (1980) *Marine Durability Survey of the Tongue Sands Tower*. Technical Report No. 5, Concrete in the Oceans, Taywood Laboratories.
7 Taylor, H.F.W. (1990) *Cement Chemistry*, pp. 396–398. Academic Press, New York.
8 Mather, B. (1966) *Highway Res. Rec.*, No. 113.
9 Regourd, M. (1975) *Ann. Inst. Tech. Batim. Trace Publics.*, **329**, 86.
10 Conjeaud, M.L. (1980) Mechanism of sea water attack on cement mortar, in *Performance of Concrete in the Marine Environment*, ACI Special Publication SP-65, pp. 39–62. American Concrete Institute, Detroit, MI.
11 Onabolu, O.A., Khoury, G.A., Sullivan, P.J.E. and Sterritt, R. (1985) Inorganic acid content of NS oil: effect of anaerobic bacterial activity in tanks. *Petr. Rev. J. Inst. Petr.*, January, 42–45.
12 Morgan, T.D.B., Steele, A.D. and Gilbert, P.D. (1983) Concrete durability in acidic stagnant environments, in *Microbial Corrosion Conference, Teddington, National Physics Laboratory*, pp. 66–73. Metals Society, London
13 Thornton, H.T. (1978) Acid attack on concrete caused by sulphur bacteria action, *ACI J.*, 557–584.
14 Forrester, J.A. (1959) *An Unusual Example of Concrete Corrosion Induced by Sulphur Bacteria in Sewers*. Report TRA/320, Cement and Concrete Association, London.
15 Neville, A.M. (1985) *Properties of Concrete*, 3rd Edition, pp. 454. Pitman Press, London.
16 Wolschow, D. (1952) *Proc. ASPM*, **52**, 250–266.
17 Peltier, R. (1970) Résultats des essais de longue durée de la résistance des ciments à la mer, au Laboratoire Maritime de la Rochelle. *Revu. Matér.*, No. 680.
18 Regourd, M. (1980) Physical–chemical studies of cement pastes, mortars and concretes exposed to sea water, in *Performance of Concrete in the Marine Environment*, ACI Special Publication SP-65, pp. 63–82. American Concrete Institute, Detroit, MI.
19 Kalousck, G.L., Porter, L.C. and Benton, E.J. (1972) Concrete for long-time service in sulphate environment, *Cement and Concr. Res.*, **2**, No. 1, 79–89.
20 Mehta, P.K. (1989) Pozzolanic and cementitious by-products in concrete—another look, in *Proceedings of International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Trondheim, 1989*. ACI Special Publication SP-114, Vol. 1, pp. 1–44. American Concrete Institute, Detroit, MI.
21 Vanden Bosch, V.D. (1980) Performance of mortar specimens in chemical and accelerated marine exposure, in *Performance of Concrete in the Marine

*Environment*, ACI Special Publication SP-65, pp. 487–507. American Concrete Institute, Detroit, MI.

22 Osborne, G.J. (1992) The performance of Portland and blast furnace slag cement concretes in marine environenments, *Proceedings of 4th CANMET/ACI International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Istanbul*, Vol. 2, pp. 1303–1324. ACI Special Publication SP-132. American Concrete Institute, Detroit, MI.

23 British Standards Institution (1989) *BS 6349, Part 1, 1989 Amendment No. 4, Code of Practice for Maritime Structures*. AMD 6159, British Standards Institution, London.

24 Sellevold, E.J. *et al.* (1982) *Silica Fume–Cement Pastes: Hydration and Pore Structure*, pp. 19–50. Report BML, 82.610, Norwegian Institute of Technology, Trondheim.

25 Diamond, S. (1984) Scientific basis for the use of microsilica in concrete, presented at *First Seminar on Elkem Microsilica Technology, Elkem Chemicals, Sao Paulo, Sept. 1984.*

26 Mangat, P.S. and El-Khatib, J. (1992) Influence of initial curing on porosity and pore structure of blended cement concretes, in *Proceedings of 4th CANMET/ACI International Conference on Fly Ash, Silica Fume, Slag and Other Natural Pozzolans in Concrete, Istanbul*, Vol. 1, p. 813. ACI Special Report SP-132, American Concrete Institute, Detroit, MI.

27 Fiskea, O.M. (1973) *Betong i Alkunskifer*. Publication No. 101, Norwegian Geotechnical Institute, Oslo.

28 Asgeirsson, H. and Gundmundson, G. (1979) Pozzolanic activity of silica dust, *Cement Concr. Res.*, **9**, 249–252.

29 Mehta, P.K. (1981) Sulphate resistance of blended Portland cements containing pozzolans and granulated blast furnace slag, presented at the *5th International Symposium on Concrete Technology, Monterrey, March 1981*.

30 Page, C.L. and Vennesland, O. (1983) Pore solution composition and chloride binding capacity of silica fume cement pastes, *Mater. Struct.*, **16**, 19–25.

31 Glasser, F.P. and Marr, J. (1984) The effect of mineral additives on the composition of cement pore fluids. *Proc. Br. Ceram. Soc.*, **35**, 419–425.

32 Mangat, P.S. and El-Khatib, J. (1992) Influence of initial curing on sulphate resistance of blended cement concrete, *Cement Concr. Res.*, in press.

33 Tuuti, K. (1982) *Corrosion of Steel in Concrete*. Swedish Cement and Concrete Association, Stockholm.

34 Marshall, A.L. (1989) *Marine Concrete*. Blackie (Van Nostrand Reinhold).

35 Roberts, M.H. (1981) *Carbonation of Concrete Made with Dense Natural Aggregates*. BRE Information Paper, IP6/81.

36 Schiessl (Ed.) (1988) *Corrosion of Steel in Concrete*. Rilem Report TC60-CSC, Chapman and Hall, London.

37 Mangat, P.S. and Gurusamy, K. (1987) Pore fluid composition under marine exposure of steel fibre reinforced concrete, *Cement Concr. Res.*, **17**, 734–742.

38 Crank, J. (1975) *The Mathematics of Diffusion*, p. 414. Oxford University Press, London.

39 Mangat, P.S. and Gurusamy, K. (1987) Chloride diffusion in steel fibre reinforced marine concrete, *Cement Concr. Res.*, **17**, 385–396.

40 Stillwell, J.A. (1983) *Concrete in the Oceans*. Technical Report No. 8, p. 59.

41 Wiebenga, J.G. (1980) Durability of concrete structures along the North Sea coast of the Netherlands, in *Performance of Concrete in the Marine Environment*, ACI Special Publication SP-65. American Concrete Institute, Detroit, MI.

42 Mangat, P.S. and Gurusamy, K. (1987) Chloride diffusion in steel reinforced concrete containing pfa, *Cement Concr. Res.*, **17**, 640–650.

43 Valenta, O. (1969) Paper presented at the *International Symposium on the Durability of Concrete, Prague*.
44 Browne, R.D. (1980) in *Performance of Concrete in the Marine Environment*, ACI Special Publication SP-65, p. 169. American Concrete Institute, Detroit, MI.
45 Sorensen, A. and Maahu, E. (1982) *Nordic Concr. Res.*, No. 1, Paper No. 24.
46 Dector, M.H., Short, N.R., Page, C.L. and Higgins, D.D. (1989) Chloride ion penetration into blended cement pastes and concretes, in *Proceedings of 3rd International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolanas in Concrete, Trondheim, Norway, 1989* (Ed. Malhotra, V.M.), ACI Special Publication SP-114, Vol. 2, pp. 1399–1411. American Concrete Institute, Detroit, MI.
47 Molloy, B.T. (1990) Steel fibre and rebar corrosion in concrete under marine curing. *PhD Thesis*. Aberdeen University, Aberdeen.
48 Azari, M.M., Mangat, P.S. and Tu, S.C. (1992) Chloride ingress in microsilica concrete, *Int. J. Cement Composites*, in press.
49 FIP (1974) *Recommendations for the Design and Construction of Concrete Sea Structure*, 2nd edn. FIP.
50 Bamforth, P.B. (1991) The water permeability of concrete and its relationship with strength, *Mag. Concr. Res.*, **43**, No. 157, 233–241.
51 Browne, R.D. and Domone, P.L. (1975) Permeability and fatigue properties of structural marine concrete at continental shelf depths, in *International Conference on Underwater Construction Technology, Cardiff, April 1975*, p. 39.
52 Mehta, P.K. (1988) Durability of concrete exposed to the marine environment, in *Concrete in the Marine Environment, Proceedings of Second International Conference, Canada, 1988*, ACI Special Publication SP-109, pp. 1–30. American Concrete Institute, Detroit, MI.
53 Grube, H. and Lawerence, C.D. (1984) Permeability of concrete to oxygen, in *Proceedings of Rilem Seminar on Durability of Concrete Structures Under Normal Outdoor Exposure, Hanover, March 1984*, pp. 68–79.
54 Concrete Society (1991) *Permeability Testing of Site Concrete*. Technical Report 31, Concrete Society, London.
55 American Concrete Institute (1991) *Corrosion of Metals in Concrete*. ACI 22R-89, American Concrete Institute, Detroit, MI.
56 Wilkins, N.J.M. and Lawrence P.F. (1985) The corrosion of steel reinforcements in concrete immersed in sea water, in *Corrosion of Reinforcement in Concrete Construction* (Ed. Crane, A.P.). Ellis Horwood, Chichester.
57 Wilkins, N.J.M. (1990) Localised corrosion of reinforcement in marine concrete, in *Corrosion of Reinforcement in Concrete* (Eds Page, C.L., Treadaway, K.W.J. and Bamforth, P.B.), pp. 3–18. Elsevier Applied Science, Barking.
58 Hamilton, W.A. and Sanders, P.F. (1984) Sulphate-reducing bacteria and aerobic corrosion, in *Corrosion and Marine Growth on Offshore Structures* (Eds Lewis, J.R. and Mercer, A.D.). Ellis Horword, Chichester.
59 Wilkins, N.J.M. and Lawrence, P.F. (1980) *Fundamental Mechanisms of Corrosion of Steel Reinforcements in Concrete Immersed in Sea-Water*. Concrete in the Oceans Technical Report No. 6, Cement and Concrete Association, Slough.
60 Wilkinson, T.G. (1984) Biological mechanisms leading to potential corrosion problems, in *Corrosion and Marine Growth of Offshore Structures* (Eds Lewis, J.R. and Mercer, A.D.). Ellis Horwood, Chichester.
61 Sadegzadeh, M., Page, C.L. and Kettle, R.J. (1987) Surface microstructure and abrasion resistance of concrete, *Cement Concr. Res.*, **17**, 581–590.

62 Gjorv, O.E., Baerland, T. and Ronning, H.R. (1987) High strength concrete for highway pavements and bridge decks, in *Proceedings of Utilisation of High Strength Concrete, Stavanger, Norway, 1987*, pp. 111–122.
63 Liu, T.C. (1981) Abrasion resistance of concrete, *Proc. Am. Concr. Inst.*, **78**, 341–350.
64 Scanlon, J.M., Jr (1980) Applications of concrete polymer materials in hydrotechnical construction, in *Applications of Polymer Concrete*, ACI Special Publication SP-69, pp. 45–62. American Concrete Institute, Detroit, MI.
65 Hoff, G.C. (1988) Resistance of concrete to ice abrasion—a review, in *Concrete in the Marine Environment*, ACI Special Publication SP-109, pp. 427–456. American Concrete Institute, Detroit, MI.
66 *Canmet Contract File No. 23440-6-9003*. Available from Energy, Mines and Resources, CANMET, Ottawa, Ontario, Canada.
67 Centre d'Information de l'Industrie Cimentière Belge (1957) *Le Beton et le Gel*, Bull. No. 61, 62, 63, 64.
68 Powers, T.C. (1954) Void spacing as basis for producing air-entrained concrete, *J. Am. Concr. Inst.*, **50**, 741–760; Discussion, 760.
69 Philleo, R. (1987) Frost susceptibility of high-strength concrete in *Concrete Durability*, Vol. 1, ACI Special Publication SP-100, pp. 819–842. American Concrete Institute, Detroit, MI.
70 Sorenson, E.V. (1983) Freezing and thawing resistance of condensed silica fume (microsilica) concrete exposed to deicing chemicals, ACI Special Publication SP-79, pp. 709–718. American Concrete Institute, Detroit, MI.
71 Yamato, T., Emoto, Y. and Soeda, M. (1987) Freezing and thawing resistance of concrete containing chloride, in *Concrete Durability*, Vol. 1, ACI Special Publication SP-100, pp. 901–918. American Concrete Institute, Detroit, MI.

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