

CHAPTER 75

DURABILITY OF CONCRETE IN COAST PROTECTION WORKS

by

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ABSTRACT

Frost action and biological attack are not important causes of damage to concrete in coast protection works in the British Isles and no cases of alkali-aggregate reaction have been positively identified. Chemical attack is usually confined to structures containing concrete of indifferent quality. Damage to exposed concrete structures due to abrasion by wave driven shingle is, however, extensive at many sites in the United Kingdom and gives rise to a considerable maintenance problem. A series of experimental panels was laid in the apron of a sea wall at Fleetwood in 1961 in order to compare the performance of different concrete mixes when exposed to attack by the sea. Some provisional conclusions have been drawn from a study of the results of this experiment, and a survey of coast defence works in England and Wales has provided additional information. Suggestions are made for further research into factors affecting the durability of concrete in coast protection works.

SCOPE OF PAPER

The deterioration of concrete structures as a result of chemical attack, repeated freezing and thawing, and alkali-aggregate reaction has been extensively documented, but relatively little information has been published about the effects of abrasion on concrete structures in marine environments. This paper deals in some detail with this problem as well as considering other, better known, causes of deterioration of concrete structures in the coastal waters of the British Isles. It also touches briefly on some aspects of design and construction that have a bearing on durability of marine structures and makes some suggestions for further research.

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CAUSES OF DETERIORATION

The main causes of deterioration of concrete in marine conditions are: repeated freezing and thawing; chemical attack; biological attack; mechanical damage, including abrasion; crystal growth; corrosion of reinforcement.

FREEZING AND THAWING

Severe frost is rare in the coastal waters of the British Isles and freezing and thawing is therefore not an important cause of damage to concrete in marine structures. It may be a contributory factor in the deterioration of any concrete that is of indifferent quality, especially in places such as river estuaries where the salinity of the water is reduced.

CHEMICAL ATTACK

Most marine structures on the coast of Great Britain have been built with concrete containing ordinary or rapid-hardening Portland cement complying with BS 12. These cements correspond approximately to ASTM Types 1 and 111 cements, but the alkali and magnesia contents of British Portland cements are usually rather lower than the corresponding American products, the maximum weight of magnesia being limited by the Standard to 4%. In a few cases, sulphate-resisting Portland cement has been used. This is approximately equivalent to ASTM Type V cement; the content of tricalcium aluminate is limited to a maximum of 3.5% and magnesia to 4%.

Experimental work has indicated that in some circumstances special cements may be preferable to ordinary Portland cement in marine structures, particularly where they are subjected to alternate wetting and drying but, in spite of this, numerous structures built with ordinary Portland cement have given satisfactory service. The only cases that the authors have observed in which chemical attack has clearly been a major factor in the deterioration of concrete in sea-water have been structures in which the concrete was of indifferent quality: sometimes the cement content was inadequate and usually the concrete had not been properly compacted with the result that sea-water had permeated the full thickness of the concrete. Some engineers consider that concrete made with sulphate-resisting Portland cement is more resistant to abrasion than concrete made with ordinary Portland cement, but it is possible that some instances of deterioration of ordinary Portland cement concrete which at first sight appear to be due to abrasion may have been aggravated by sulphate attack affecting the surface of the concrete and rendering it more vulnerable to damage by the action of waves and shingle. The additional cost of sulphate-resisting Portland cement is low (currently 32s.6d. per ton), and its use may be a worth-while insurance where satisfactory repairs would be difficult or impossible.

Alkali-aggregate reaction is not considered in this paper because very few reactive aggregates are known to occur in the British Isles.

BIOLOGICAL ATTACK

Biological attack is not likely to occur in marine structures as stagnant conditions are generally required for the growth of the microbes. It is possible however that microbes may contribute to chemical attack on porous or honeycombed concrete where the conditions necessary for their growth may obtain in the pores of the concrete. Chemical secretions from the growth of seaweed on concrete structures may also contribute to surface deterioration (Plate I).

ABRASION

At many places on the coast of Great Britain, abrasion of sea walls and revetments by wave-driven shingle gives rise to a considerable maintenance problem. A particular case is Fleetwood in Lancashire where the foundations of the sea walls along the west shore are protected by sloping aprons with wearing faces of granite blocks and concrete which require continual repair.

In order to compare the performance of different materials a series of experimental panels was constructed in 1961 (Plate II) and records have been kept of the rate of loss by abrasion. Twenty eight concrete panels each 15 ft. x $6\frac{1}{2}$ ft. x 6 in. thick were constructed over a 200 ft. length of apron, the mix designs being all based on the same water/cement ratio and workability.

A control panel using rapid hardening Portland cement, with aggregate/cement ratio 6.6, and $1\frac{1}{2}$ in. and $\frac{3}{4}$ in. single size Shap granite aggregate and beach sand, was included in each day's work so that account could be taken of variations in conditions from day to day during construction and differences in beach level from one end of the experimental length to the other. Interpretation of the results is complicated since each deliberate change involved other variations. The average wear suffered by the six control panels has varied with position from $\frac{1}{2}$ in. in 7 years at one end of the length to $1\frac{3}{4}$ in. at the other. This difference has been taken into account by adjusting all quoted figures so as to be comparable with the most severely abraded control panel.

The loss sustained by the six panels with different cements, which otherwise were of the same proportions and contained the same aggregates as the controls, has varied between $\frac{1}{2}$ in. of wear on average for high alumina cement concrete to $3\frac{1}{2}$ in. for super-sulphated cement concrete. The high alumina cement concrete has decayed in patches, forming isolated cavities several inches deep; this may be due to "conversion", or to alkali attack from Portland cement concrete below the panels. It seems that neither of these panels will have a life much in excess of ten years. Of

the other panels, sulphate-resisting cement concrete appears so far to be the most durable (average wear $1\frac{1}{2}$ in.) followed by ordinary Portland and rapid hardening Portland cement concrete (average wear $1\frac{3}{4}$ in.) and extra rapid hardening Portland cement concrete which has lost $2\frac{1}{2}$ in. on average. Extra rapid hardening cement contains calcium chloride which is incorporated during manufacture and its relative performance as shown by these results is confirmed by one of the control panels in which an admixture containing calcium chloride was used.

The results obtained from changes in grading and type of aggregate are more difficult to compare since changes in aggregate/cement ratio were required in order to maintain the same water/cement ratio and workability. With the same aggregates as used in the panels already discussed, but aggregate/cement ratios (a/c) from 6.2 to 7.0 and sand contents from 30% to 38%, the average loss in 7 years has varied from 2 in. to $2\frac{3}{4}$ in. At a/c = 7.0 a reduction in sand content from 38% to 32% gave no improvement in performance (average wear $2\frac{1}{2}$ in.) but with a/c = 6.2 the amount of wear has been slightly less on concrete with 30% of sand than on that with 36% (2 in. and $2\frac{1}{4}$ in. respectively). The control panel concrete with a/c = 6.6 is apparently more durable than the best of this group and it may be concluded that with $1\frac{1}{2}$ in. maximum size angular aggregate little advantage is to be gained by using mixes richer than about $6\frac{1}{2}$ to 1. A better performance was obtained by the substitution of a crushed granite fine aggregate for the beach sand, but this required a richer mix to maintain workability (a/c = 6.0; wear in 7 years - 1 in. average).

Three panels were cast using a continuously graded coarse aggregate ($1\frac{1}{2}$ in. to $3/16$ in.) and a panel was made with $\frac{3}{4}$ in. single size granite aggregate which performed marginally better than the $1\frac{1}{2}$ in. and $\frac{3}{4}$ in. single size or $1\frac{1}{2}$ in. to $3/16$ in. graded aggregates but required a richer mix (a/c = 5.8). The differences between these and corresponding panels using $1\frac{1}{2}$ in. and $\frac{3}{4}$ in. single size aggregates are too small for conclusions to be drawn at this stage.

Several different types of aggregates were included in the experiment and from observation to date they may be arranged in the following order from most to least durable: (1) Flint gravel (2) Fine-grained granite (3) Coarse-grained granite (4) Hard blue limestone (5) Limestone (6) Pit gravel containing mainly sandstone (millstone grit) but also some limestone and igneous rock.

In addition to the usual laboratory tests on the concrete and concrete materials, a series of abrasion tests was carried out by sand-blasting 12 in. cubes of some of the concretes used on site. In these tests, the super-sulphated cement concrete and rapid hardening cement concrete with high sand content (38%) were significantly less resistant, and two specimens of flint aggregate concrete significantly more resistant, than the others. Apart from these four cases, no correlation has been found between the abrasion test results and the properties of the concrete, such as strength and density, or the performance in the field.

A full-scale experiment, but more limited than the trials at Fleetwood, is being carried out by the Kent River Authority at Dymchurch, where a sloping apron was reconstructed in 1963. The work consists mainly of panels of precast concrete blocks surrounded by ribs of in-situ concrete, but some panels of granite blocks have been included for purposes of comparison (Plate III). In order to compare the performance of different types of concrete, some of the precast concrete blocks were made with gap-graded aggregate and some with continuously graded aggregate. Flint coarse aggregate of $1\frac{1}{2}$ in. maximum size was used and in the gap-graded concrete was single size. The aggregate/cement ratio was 5 and the cement ordinary Portland. Concrete test cubes made during the course of the work had crushing strengths of about 6,000 lb./in.² at 7 days and 7,300 lb./in.² at 28 days. So far there has not been enough abrasion for any significant differences to be seen between the granite and the various concretes.

Parts of another apron at Littlestone were reconstructed by the Kent River Authority in 1960 and 1966 with ragstone slabs set in fine concrete. The fine concrete jointing material used had an aggregate/cement ratio of $2\frac{1}{2}$ using flint aggregate from Dungeness graded from $\frac{1}{2}$ in. down to no. 100 BS sieve, in accordance with the requirements for all-in aggregates of BS 1201. Sulphate-resisting Portland cement was used with 4% entrained air, and 2% calcium chloride was added as an accelerator. This jointing material appears to be very resistant to abrasion and the ragstone slabs in the 1960 work are showing signs of greater wear than the jointing concrete.

An interesting example of a concrete sea wall exposed to very severe abrasive conditions is at Sheringham on the coast of Norfolk. The beach material consists of a very hard flint shingle and the exposure is severe. The sea wall was built in the summer of 1967 using concrete containing a coarse aggregate consisting of irregular flint gravel ($1\frac{1}{2}$ in. maximum) similar to the beach material and sand from the same source. The proportion of sand in the total aggregate was 34% and the aggregate/cement ratio of the concrete was 7.1 with ordinary Portland cement. Precast reinforced concrete retaining wall units that were part of a manufacturer's standard range were used as permanent formwork. These units were not designed to withstand such severe exposure and in places they have almost completely disappeared (Plate IV). The concrete in the main structure is showing some signs of abrasion but is wearing more slowly.

Coast protection works at a number of places on the coasts of Wales and the West of England where concrete has suffered from abrasion have been examined. Plate V showing a sea wall at Porthcawl in Glamorgan is a typical example. In most cases the work was constructed fifteen or more years ago, and in much of it concrete having nominal mix proportions 1:2:4 by volume was used and the concrete was compacted by hand. Although the performance of structures in different situations cannot be compared, owing to differences in exposure, a general inspection does suggest that the older structures, in which the concrete was probably not very well compacted and in which little quality control was exercised, have not proved as durable as more modern structures incorporating carefully designed concrete mixes with better standards of quality control and compaction.

CRYSTAL GROWTH

Salt crystals growing in cracks in concrete structures may exert enough pressure to cause spalling of the concrete. This is particularly likely near high water level and in the splash zone. It has been suggested to the authors that sodium sulphate formed through the double decomposition of sodium chloride and gypsum is more likely to cause this trouble than crystals of sodium chloride. One of the authors has seen some concrete sea walls in Lincolnshire on the east coast of England in which spalling of concrete has started from fine shrinkage cracks (Plate VI) and this may well have been due to crystal growth.

CORROSION OF REINFORCEMENT

It is considered that there should be at least 2 in. (5 cm.) of dense concrete cover to all reinforcement in marine structures in order to prevent corrosion, and an additional allowance should be made for abrasion when the necessary concrete cover is assessed.

In the past structures have been built at a number of coastal sites in which inadequate compaction of the concrete surrounding the reinforcement has led to corrosion of the steel and cracking of the concrete. Many of these structures were built before mechanical vibrators came into general use, and in some cases the trouble had been aggravated by detailing of reinforcement that made compaction of the concrete difficult. This applies particularly near the lower edges of heavily reinforced beams.

JOINTS

A really satisfactory method of sealing expansion joints does not appear to have been developed. Both hot-poured and cold-poured sealants are often dislodged by wave action and the authors have seen a number of sea walls in which the expansion joints are inoperative because the sealant has been torn out and stones have become wedged in the joints (Plate VII). Extruded plastics sections cast into the concrete have given promising results and

it is suggested that ribbed neoprene strips of the type sometimes used in sealing joints in concrete roads might be tried for this purpose. Atmospheric humidity near the sea varies relatively little and the annual temperature range is considerably less in coastal areas than it is inland, so that movements of joints should be relatively small; it appears that an unnecessary number of expansion joints are often provided. Since coast protection works usually involve fairly massive structures, it is difficult to provide enough reinforcement to control cracking and fairly frequent contraction joints are therefore necessary.

CONSTRUCTION

A problem in the execution of coast protection works is to prevent the sea from washing out the surface of freshly placed concrete. Formed surfaces are protected if the formwork is water-tight and rigidly fixed in position, but protection of the upper surface of a lift of concrete or a sloping apron often presents difficulties. Some of the experimental panels at Fleetwood, for instance, suffered superficial damage by the sea during the first few hours after placing, and subsequently had to be cut out and re-cast. In parts of the work carried out by the Kent River Authority at Dymchurch, the precast concrete blocks have proved more durable than the in-situ concrete surrounding them. This may be due to better compaction of the precast concrete, but it is also possible that the surface layers of the in-situ concrete may have been affected by the sea soon after placing.

FURTHER RESEARCH

The most useful information will be gained from full-scale trials in which different materials and structures are exposed to similar conditions for a number of years. Owing to differences in exposure conditions at various places round the coast, it is not possible to draw detailed comparisons between the behaviour of different materials and structures at different places, so that full-scale trials such as those at Fleetwood are the most valuable source of information. The trials at Fleetwood are the most elaborate of which the authors are aware, but it is hoped that other authorities will undertake similar work since definite conclusions cannot be drawn from one set of results. It has been established practice for a number of years in various parts of the world to carry out full-scale trials in connexion with highway design and construction, and the authors hope that similar trials will be carried out for coast protection works.

It would be useful to develop an accelerated wear test for use under laboratory conditions, preferably using larger particles of abrasive material than those used in sand-blasting. The 'rattler test' for abrasion resistance of precast concrete paving flags, described in BS 368 might be suitable for development for this purpose. The apparatus used in this test consists basically of a

steel box measuring about 1 x 1 x 2 ft. A paving flag is fixed to the inside of each 1 x 2 ft. side and 1,000 steel balls $\frac{1}{2}$ in. in diameter are placed in the box. The ends of the box are sealed and it is rotated at 60 revolutions per minute about the long axis for 48 hours. The loss in weight of each paving flag is measured after this period.

In view of the unsatisfactory performance of many types of joint sealing material, there is also a need for experimental work on this subject. The performance of a number of poured sealants is very susceptible to inadequate preparation of the surface of the joint and, because of the difficulty of obtaining clean dry joint surfaces in concrete structures between tide levels, a sealant material that would adhere satisfactorily to a damp surface would be very valuable. Apart from this, the authors would like to see some trials carried out with plastics or neoprene sections as suggested above.



Plate I - Surface deterioration possibly
due to seaweed growth



Plate II - Experimental wearing panels at
Fleetwood



Plate III - Aprons at Dymchurch with granite and concrete blocks as wearing surfaces



Plate IV - Sea Wall at Sheringham constructed with precast concrete retaining wall units as permanent formwork



Plate V
Abrasion at base of
sea wall at Porthcawl



Plate VI
Spalling of stepped
concrete apron possibly
initiated by crystal
growth in hair cracks

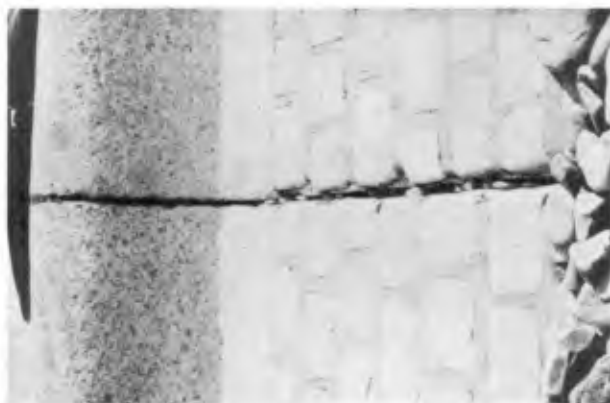


Plate VII
Stones wedged in
expansion joint from
which sealant has been
torn out