FERRO CEMENT
FOR
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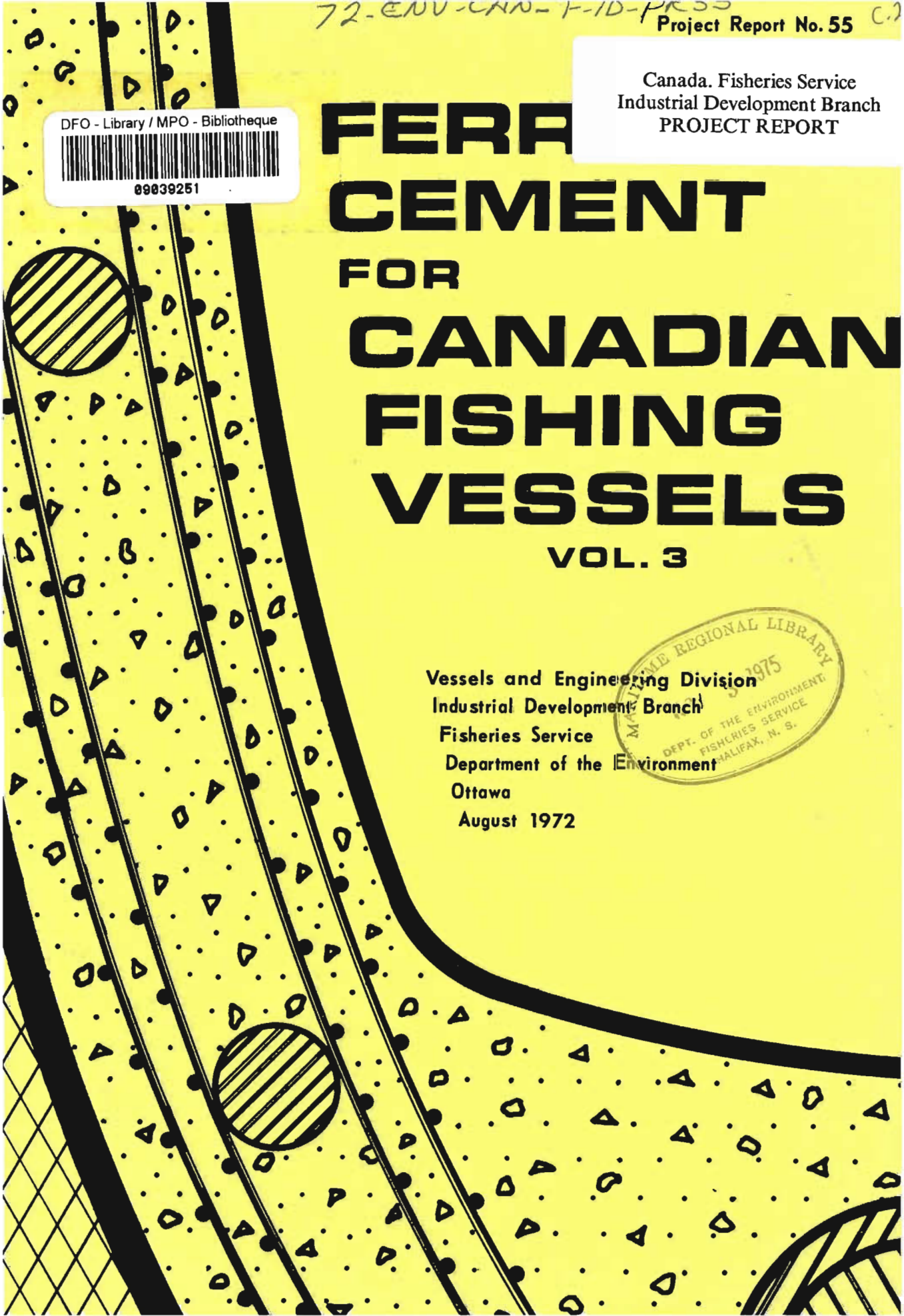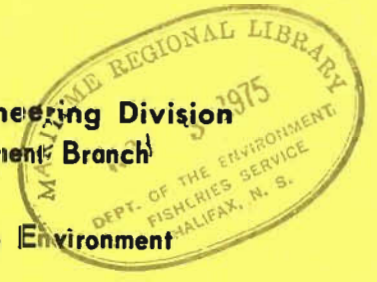
Project Report No.55

FERRO-CEMENT

FOR

FISHING VESSEL CONSTRUCTION III

Prepared by

THE BRITISH COLUMBIA RESEARCH COUNCIL

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A. INTRODUCTION.

This report covers new work on Ferrocement for Canadian Fishing Vessels carried out in the 1971-72 program under the sponsorship of the Fisheries Service, Canada Department of the Environment. It has been the intent of the study over the past several years to obtain engineering data on "typical" ferrocement construction which will aid those undertaking to build a ferrocement boat. Previous studies (1,2,3) have considered the influence of several kinds of wire mesh, reinforcing rods, and mortar, on strength mainly in flexure and impact, and durability to freeze-thaw and marine exposure. A mathematical model for a steel/mortar composite was posed.

The present report looks into such areas as the behaviour of ferrocement under cyclic flexural loads and under the stress imposed by bolted fastenings, the resistance of a variety of paint coatings under various exposure conditions, and the need for control of internal quality. The literature of ferrocement and peripheral fields has also been examined. The bibliography specific to ferrocement has been maintained.

Although most of the program time has been spent directly on the tests described above, some part of the program time has been consumed in answering the many enquiries by phone, letter, and personal visits to this laboratory on the use of ferrocement construction in such applications as septic tanks and boat construction. In addition, participation in the work of the Task Group HS-6-4 Hull Structure - Ferrocement of the Society of Naval Architects and Engineers has occupied some time.

B. SUMMARY

An apparatus constructed to apply cyclic loads to beam specimens of ferrocement has provided some preliminary values for fatique strengths in flexure. Greater replication and verification is necessary to provide results which can be used with reasonable assurance. The preliminary tests indicate that the fibre stress should not exceed about 1,000 psi if a satisfactorily long service life is to be obtained in ferrocement of the kind and construction described herein.

Bolting tests on specimens of 4-inch and 8-inch wide specimens indicate that shear/bearing bolt loads in ferrocement of the kind and construction described should be tentatively limited to about 500 lb/in.

A number of types of paint coatings, e.g., inorganic ethyl silicate zinc-rich paint, polyester-based, epoxy-based, polyamide resin-based, chlorinated rubber-based, and polyvinyl chloride-based paints were evaluated on the basis of visual appearance (flaking), spalling resistance, etc., after exposure to 600 wet/dry seawater cycles, 84 days in a tidal environment with subjection to any marine organisms present, and to 1,500 hours (6 years equivalent) in a Weather-Ometer. The inorganic ethyl silicate zinc-rich paint, the highly modified polyester resin, and a two-component pigmented epoxy resin performed best. The chlorinated rubber-based, the polyvinyl chloride-based, and two-component polyamide resin paints were judged to be good. The test results were not entirely consistent.

The potential of controlling internal quality of ferro-cement composites using ultrasonic techniques is high but a considerable research program is deemed necessary to adapt present techniques to this material, especially in the uncured stage.

Newly acquired literature on ferrocement has been briefly reviewed and the ferrocement bibliography included in previous programs has been up-dated.

Finally, recommendations for much needed work are briefly stated.

C. BEHAVIOUR UNDER CYCLIC BENDING STRESSES.

1. Background.

The behaviour of concrete under cyclic loading has been under investigation for over 50 years. Much of the work has been concerned with repeated loading of concrete cylinders in compression and concrete beams, both reinforced and unreinforced, in bending. On the basis of early work by Clemmer[4], Older[5], and Hall[6], workers generally agreed that repeated stresses higher than about 50 percent of the ultimate strength would eventually cause failure by fatigue. Stresses up to 75 percent of the modulus of rupture were considered acceptable if they were applied only a few times per year. Billig[7] discusses the general problem of the strength of concrete under repeated loadings as it pertains to design in structural concrete. More recent work by Hansen[8], using sonic techniques to indicate crack initiation and propagation, showed that cracks once formed in concrete subjected to pulsating compressive loads never stabilize but continue to grow until ultimate failure.

Yoshimoto et al[9] and Hilsdorf and Kesler[10] have studied the effect of repeated loading and microcracking on the flexural strength of concrete beam specimens. Their work shows that microcracking can increase the flexural strength of the concrete.

The effect of cyclic stresses on the behaviour of fibre-reinforced concrete has been studied to limited extent. Romualdi et al[11], in an admittedly limited study, showed that the presence of an internal mechanism to reduce extensional stresses in the vicinity of the internal flaw edges will enhance the fatigue resistance of a concrete composite. Results of tests showed that beams containing short wire fibre reinforcement may have endurance limits as high as 90 to 95 percent of the static strength whereas the endurance limit of conventional concrete is about 55 percent of the static strength.

The cracking of concrete slabs reinforced with welded wire fabric with wire spacings of 3 inches or greater has also been studied[9,10].

2. Experimental Work.

(a) Apparatus.

An apparatus was constructed to simultaneously load eight 4 x 12-inch ferrocement specimens in bending. Four pairs of roller-cams attached to a central rotating shaft apply cyclic unilateral bending loads to up to four pairs of specimens at about 30 cycles per minute. The roller cams are fixed to the shaft at 45-degree intervals to load only one pair of specimens at a time thereby allowing the use of a small motor and light construction. Third-point loading of the 4 x 12-inch specimens was accomplished by means of a strain-gauged and calibrated 4-inch channel-section load cell attached to each specimen. A six-channel quick-response Visicorder monitored the applied loads. The deflection of specimens under load was measured by a dial gauge held in a saddle support over a 9-inch span. The number of load cycles applied was recorded by a cam-activated electric counter. The apparatus and load test set-up are shown in Fig. 1 and 2. Fig. 3 shows a portion of the six-channel record of the stress.

The apparatus has the advantage of simplicity of construction and the ability to test up to four pairs of specimens at once. However, it suffers the disadvantage of being a constant-strain device rather than a constant-load device. The apparatus, after some preliminary runs, has provided some insight into the behaviour of ferrocement under repeated bending loads. Only three pairs of specimens were tested at a time because the available recorder had only six channels.

(b) Tests.

(i)   Run #1.

Six 4 x 12-inch specimens were prepared from Panel 53
which contained two layers of 1/2-19 ga. galvanized hardware cloth above
transverse 0.225 in. high-tensile double drawn rods on 2-inch centres
and three layers below longitudinal rods. These specimens were used
to develop loading, measuring, and recording techniques. Quantitative
test results are not presented in this report. The loads were maintained
at about 100 lb (a fibre stress of about 300 psi) and no specimen showed
any cracking after 110,000 load cycles.

(The expression F = 2.8P may be used to determine the
fibre stress for the imposed loads in this and subsequent test runs.)

(ii)   Run #2.

Eight 4 x 12-inch specimens were prepared from Panel 50.
This panel contained two layers of 1/2-19 ga. galvanized hardware cloth
above transverse 0.25 in. bright nail rods on 2-inch centres and three
layers below longitudinal rods. The fibre stress at first crack and
the modulus of rupture obtained from flexural testing a 12 x 24-inch
specimen from this panel were 4,050 psi and 7,350 psi, respectively.

The following initial loads were applied:

Test Specimen	Load Cell	Initial Load, lb	Initial Fibre Stress, psi
50-1	1	500	1,400
50-2	2	500	1,400
50-3	3	750	2,100
50-4	4	750	2,100
50-5	5	1,000	2,800
50-6	6	1,000	2,800
50-7	5	900	2,550
50-8	6	900	2,550

The loading mechanism was periodically tightened to
maintain a fairly constant load throughout the run but this was not
an entirely successful procedure.

Specimens 50-5 and 50-6 developed fine cracks under
transverse rods within 50 load cycles during the initial setting-up
process. By 250 cycles, 50-6 had developed a large diagonal crack
from an outside load support to a third-point load location. Speci-
ment unsoundness in this location undoubtedly influenced the diagonal

cracking. Specimen 50-5 delaminated with a diagonal crack in a similar manner within 6,000 cycles. It was not possible to maintain the initial load on specimens 50-5 and 50-6 and they were removed from the apparatus after 6,000 load cycles. Fig. 4 and 5 show the crack condition at that time.

Specimens 50-7 and 50-8 were installed in their place and loaded to 900 lb (fibre stress = 2,550 psi). These specimens showed severe cracks before 14,000 load cycles had been applied and it was not possible to maintain a constant load. Fig. 6 shows the bottom surface before and after 100,000 load cycles.

Specimens 50-2 and 50-3 exhibited bottom cracks before 20,000 load cycles. Fig. 7 and 8 show the transverse bottom crack and the deviation towards a transverse rod as the crack progresses through the specimen.

Specimens 50-1 and 50-4 were still uncracked after 106,000 load cycles.

The loads and cycles to first observed cracking are summarized in Table 1.

(iii)   Run #3.

Four specimens, 4 x 12 inches, were cut from Panel 51 which contained two layers of 1/2-19 ga. galvanized hardware cloth on transverse 0.25 in. hot-rolled steel rods on 2-inch centres and three layers below longitudinal rods. The fibre stress at first crack in the static flexure test on a 12 x 24-inch specimen from this panel was 2,620 psi. The modulus of rupture at maximum load was 6,310 psi.

The specimens were inserted into the cycle testing apparatus and loaded to impose initial fibre stresses of 1,930 to 2,600 psi, i.e., from about 75 to 100 percent of the maximum fibre stress at first crack obtained in static bending, as follows:

Test Specimen	Load Cell	Initial Load, lb	Initial Fibre Stress, psi
51-3	3	760	2,130
51-4	4	930	2,600
51-5	5	710	1,990
51-6	6	690	1,930

The cyclic load apparatus was run to 200,000 cycles. The average loads over about 20 cycles and the static deflections were

periodically measured during the run. The fibre stress of specimen 51-3 which developed fine cracks during initial loading leveled off at about 400 psi. The stress on the other specimens ranged from 600 to nearly 1,000 psi.

The fibre stresses in the specimen under load and the characteristic load-deflection curves are presented in Table 2 and Fig. 9.

(iv)   Run #4.

Four specimens from Panel 49 (reinforced with two layers of 1/2-19 ga. hardware cloth above transverse 0.225 in. high tensile double-drawn rods on 2-in. centres and three layers of 1/2-19 ga. hardware cloth below longitudinal high tensile rods), one specimen from Panel 201 (containing 2 percent of steel fibre), and one from Panel 205 (containing 1 percent of steel fibre) were tested in this test run. Static bend tests on a 12 x 24-in. specimen from Panel 49 showed a fibre stress of 2,300 psi and a modulus of rupture at maximum load of 4,900 psi. The specimens were statically loaded to selected cyclic loads to calibrate the load cell and to measure the initial deflection. The specimens were then inserted into the test apparatus and loaded to the selected loads.

The following initial loads were applied:

Test Specimen	Load Cell	Initial Load, lb	Initial Fibre Stress, psi
205-2	1	75	210
201-2	2	250	700
49-7	3	910	2,550
49-3	4	800*	2,240
49-2	5	380	1,060
49-1	6	480	1,340

(*A fine transverse crack had developed at 600 lb in static loading.)

The cyclic load apparatus was run to nearly 300,000 load cycles. The average loads over about 20-cycle increments and the static deflections of the beam specimens were determined periodically throughout the run. In this series of tests the deflections at n cycles were measured as the deflections from an initial no-load base line and from the no-load base line after n cycles. The average applied loads and deflections are presented in Table 3. Fig. 10 and 11 show how the fibre stress and deflections change with the number of imposed cycles.

The specimens 201-2 (containing 2 percent steel fibre) and 205-2 (containing 1 percent steel fibre) were also tested to nearly 300,000 cycles, but at much lower loads. Specimen 201-2 was loaded to an initial fibre stress of 700 psi. Specimen 205-2 developed a crack on initial loading and was loaded only to 210 psi. The loads on the two specimens leveled off at about 600 psi and 140 psi, respectively, as shown in Table 3. The limited testing does not permit conclusions to be drawn.

The deflections, especially dynamic deflections were difficult to measure accurately and consistently. However, certain trends were observed as shown in Fig. 11. The permanent set, i.e., the "no-load" displacement of the midpoint of the beam specimen with respect to its ends at the start of the test run, n = 0, compared to the "no-load" displacement after n cycles of load, increased until it reached a plateau of about 80,000 load cycles. The presence of a crack in the specimen appeared to delay the point where the permanent set became constant. The "no-load" load deflection at n = n decreased in a complementary manner until a plateau was reached after about 80,000 cycles. The total deflection from the original "base-line" is essentially constant since constant strain or deflection is characteristic of the cyclic loading machine used.

(c) Discussion of Test Results.

The results of Run #2 in which an effort was made to maintain the bending load (and fibre stress) at or near its original value indicate that the fibre stress should not exceed 1,400 psi for a reasonable fatigue life.

The results of Run #3 in which the high initial loads were allowed to decrease with time as cracks developed or as a creep mechanism (possibly microcracking) occurred indicated that fibre stresses as high as 1,000 psi could be carried for many cycles provided cracking did not occur early in the test.

Probably the most interesting results were obtained from Run #3. The load on one of the two specimens with high initial loads (greater than 2,000 psi fibre stress) decreased rapidly to about 1,000 psi fibre stress. The other, which developed an early crack, decreased to about 600 psi. However, the loads on the two specimens with only moderate initial loads (1,070 and 1,340 psi) dropped only slightly to about 1,000 psi.

The original crack in specimen 49-3 opened and closed during each loading and unloading cycle but even after nearly 300,000 cycles had an at-rest crack width of less than 0.01 in. The crack

in specimen 49-3 had progressed through to the compression surface as a fine almost invisible line. The mortar was carefully chipped away from the crack at the tensile side of the specimen. All longitudinal wires in the outside layer of mesh were broken. None of the broken wires showed the elongation or ductility observed in mesh wires broken in tensile loading. The breaks in the wires are characteristic of those resulting from a fatigue mechanism.

Reinforced concrete design presupposes that the tensile strain in the concrete near the steel must exceed the extensibility of the concrete if the useful strength of the steel reinforcement is to be approached. In many applications, cracking can be tolerated. However, open cracks cannot be tolerated in corrosive environments, such as the saltwater marine environment to which ferrocement vessels are exposed. Various researchers, e.g., Nawy[12], are investigating crack control and the allowable crack width in concrete slabs reinforced with welded wire fabric in terms of crack spacing and concrete cover. The ultimate criterion of failure of ferrocement under static and repeated tensile and flexural loads is a crack which allows ingress of a corrodent and consequent deterioration of the fine reinforcement. Future work in this area would be a valuable contribution to the practical application of ferrocement to the construction of fishing vessels.

(d) Conclusions.

The tests which need greater replication and verification indicate that ferrocement specimens of the type of construction described can, in the absence of macrocracks, withstand cyclic (fatigue) fibre stresses of about 1,000 psi. Further, it is concluded that highly-loaded ferrocement can plastically deform through creep, microcracking, or other mechanism, to diminish the bending load to a tolerable level.

D. FASTENINGS.

1. Background.

Ferrocement, in common with other "brittle" materials (arbitrarily described as those materials with failure strains less than about 5 percent), shows a considerable difference between its ultimate strength in tension and its strength in compression. The distribution of stress at a bolt hole is, therefore, quite different from that obtaining at a bolt hole in a steel plate. It follows that the practice for bolting attachments to ferrocement will differ from the practice for bolting to steel plate which, in the case of single- or double-bolted lap joints, are practically solved by "shop rules"

as described in various books on strength of materials and machine design.

Bolted lap joints in steel may fail from any one of several causes: (a) shearing of the bolt, (b) tearing of the plate through the bolt holes, (c) crushing of the plate back of the bolt in bearing, (d) crushing of the bolt in bearing, (e) tearing out the plate in front of the bolt, and (f) splitting of the plate in front of the bolt. Several of these will not apply to ferrocement applications.

This report has made a preliminary study of the behaviour of ferrocement when attachments are bolted to it. The study has been confined to the case of double- and single-lap joints carrying planar loads (such as might be applied to the gunwale portion of a hull by a chain plate attachment). The relationship between the marginal pitch (the distance from the edge of the ferrocement plate to the centre of the bolt-hole) and the inter-bolt-hole spacing has been considered.

2. Experimental Work.

A series of tests was undertaken to determine the bolting characteristics of ferrocement. The tests were confined to loads parallel to the surface plane of the ferrocement test panel specimens. No tests were undertaken in which the bolts and panel were loaded normal to the panel surface, i.e., with the panel acting as a membrane. The tests provided a measure of the resistance of the ferrocement in bearing and in tensile shear.

Specimens were all cut from panels of generally similar construction. All specimens had 0.225-in. double drawn steel reinforcement (80,000 psi) on 2-in. centres in both directions and approximately the same weight of mesh reinforcement per square foot of panel area. Specimens containing 1/2-16 ga. galvanized welded square mesh reinforcement contained two layers, one layer on each side of the two-layer rod reinforcement. Those containing 1/2-19 ga. galvanized hardware cloth contained five layers, two layers on one side and three layers on the other side of the two-layer rod reinforcement. Those containing the 1/2-22 ga. galvanized hexagonal mesh reinforcement had ten layers, five layers on each side of the two-layer rod reinforcement. All specimens were approximately one inch thick.

In the present tests, four types of ferrocement specimens were tested, these representing four bolt locations with respect to the distance from the free edge and to the distance from the free sides (in effect, the bolt spacing in a multi-bolt assembly). The four types were:

- Type A, 4 x 8-inch specimens with a bolt hole 2-inches from each end and each side;

- Type B, 4 x 12-inch specimens with a bolt hole 4-inches from each end and 4-inches from each side;

- Type C, 8 x 8-inch specimens with a bolt hole 2-inches from each end and 4 inches from each side;

- Type D, 8 x 12-inch specimens with a bolt hole 4-inches from each end and each side.

Specimen types A and B approximately simulate a 4-inch bolt-hole distance between adjacent bolts and types C and D an 8-inch bolt-hole distance between adjacent bolts. All bolt holes were drilled in the centre of the space bounded by the 0.225-inch reinforcing rods on 2-inch centres. The holes were drilled through the ferrocement test specimens with a carbide-tipped spade masonry drill. The bolting strength tests approximately simulate a condition such as a chain plate bolted to the hull gunwale.

Bolting of the ferrocement specimens was accomplished in the following manner:

High strength steel bolts, 1/2-inch diameter, were passed through a steel bar (5/16-inch thick by 3-inch wide), the ferrocement specimen, a 4 x 4-inch piece of 3/4-inch fir plywood, a 1/2-inch washer, and another 5/16 x 3-inch steel bar. The assembly was tightened until the 1/2-inch washer was impressed into the fir plywood. The tensile loading mainly was transmitted to the ferrocement test specimen through the 5/16 x 3-inch steel bars in essentially a co-planar manner such as c chain plate loads would be transmitted to the hull gunwale. Sketches of the specimens and loading configurations are shown in Fig. 12.

One type A and one type B specimen (1/2-19 ga. hardware cloth) were tested with the loading configuration as shown in the lower portion of Fig. 12. This set-up uses a steel bar on only one side of the ferrocement specimen and consequently induces higher bending stresses in the specimen. The tests are identified in Table 4 as E(A) and F(B) to indicate the modified loading configuration for specimens of types A and B.

The tensile loading was applied by means of a Tinius Olsen Universal Testing Machine (60,000-lb capacity) using the 12,000-lb range. The maximum load held was recorded. Also, the load was recorded when a crack in line with one of the holes (either a transverse or longitudinal crack) was first observed or when a momentary dropping of the load indicator needle indicated a crack. Vertical cracks started at the end of the specimen and progressed towards the hole. The vertical cracks, although they were sometimes visible before any transverse cracks were seen, probably started after transverse cracks originating at the hole, since the transverse cracks were obscured by the steel bar until a crack had become at least an inch long.

The test results presented in Table 4 show that a hole distance of two inches from the free edge is adequate to prevent failure by edge breakout when the specimen width (bolt hole spacing) is four inches. A hole distance of four inches from the free edge is necessary to prevent edge breakout when the specimen width (bolt hole spacing) is eight inches. Fig. 13 to 16 show typical failures.

Since 4-inch wide specimen types A and B failed in a similar mode, i.e., with generally transverse cracking through one bolt hole, it is reasonable to treat the two types jointly when comparing the bolt-hole strength of specimens from 1/2-16 ga. welded square mesh, 1/2-19 ga. hardware cloth, and 1/2-22 ga. hexagonal mesh. Since 8-inch wide type C specimens in general behaved in a different manner from type D specimens, they are treated separately.

The average maximum loads held are:

	Type A, B		Type C		Type D	
	lb	lb/in.	lb	lb/in.	lb	lb/in.
1/2-16 ga. welded square mesh	3,350	840	4,200	525	6,000	750
1/2-19 ga. hardware cloth	3,030	760	3,600	450	5,700	710
1/2-22 ga. hexagonal mesh	2,570	640	3,600	450	5,200	650

3. Discussion and Conclusions.

The above summary indicates that the closer bolt hole spacing (4-inch vs 8-inch) obtains higher unit loads, lb per inch of width. The load values at first visible crack, as shown in Table 4, indicate that unit loads much below the maximum loads given above will cause cracking at the hole. Since the hole crack is not visible until it has attained a crack length of at least one inch, the loads at crack initiation are only approximate and bolt loadings considerably lower than one half of the maximum loads should be used. In view of the above uncertainty, the unknown distribution of loads carried by adjacent bolts, the high possibility of a non-linear loading different from the essentially planar loading used in these tests which will induce bending stresses in the ferrocement plate, and the possible cyclic nature of the loadings it would seem unreasonable to use bolting arrangements which exert more than about 500 lb/in. in ferrocement panels of the kind and construction described herein.

E. COATINGS FOR FERROCEMENT.

1. Background.

Although history has shown that many concrete structures have failed from environmental causes, there is abundant evidence of many structures which have lasted decades and even centuries. In most natural environments well-made concrete structures do not deteriorate. The concrete (mortar) remains sound and the steel reinforcement is protected by a passivating film of iron oxide that forms in the alkaline concrete environment during placement, setting, and curing. In more aggressive environments, reinforced concrete (and ferrocement) can deteriorate from physical-chemical changes in the mortar itself or from chemical attack on the steel reinforcement. Descriptive information on the deterioration of mortar and reinforcement and on the coatings which may be applied lies chiefly in the literature of concrete structures other than ferrocement vessels. The literature provides a good understanding of the mechanisms which cause mortar deterioration and reinforcement corrosion.

(a) Deterioration of Mortar.

Concrete and mortar may fail from a number of causes. Sulphate attack is a common cause of concrete (mortar) deterioration. Sulphate attack occurs when sulphate ions in the environment react with hydrated lime and tricalcium aluminate in the cement mortar to

form a complex calcium sulfo-aluminate compound. This compound can combine with water to form crystals which increase in volume and disrupt the cement mortar. Cement Types II and V, containing low contents of tricalcium aluminate, are generally used in ferrocement work and, in general, deterioration of ferrocement by sulphate attack in a marine salt water environment should not be a serious problem. The vulnerability of concrete (mortar) to chemical attack is affected by several factors, especially the permeability of the concrete (mortar) to liquids and gases. The permeability of concretes and mortars can vary by a factor of 10^6.

It has been pointed out in earlier studies that disintegration of concrete by weathering is caused mainly by the disruptive action of freezing and thawing and, of course, by expansion and contraction under restraint both from temperature changes and alternate wetting and drying. The resistance of concrete and mortar to disintegration from these causes is also related to permeability and to the size distribution and shape of air voids.

Concrete and mortar may also suffer deterioration from high expansion forces exerted within the structure by corrosion products formed when steel reinforcement is attacked by corrosive agents such as salt water.

(b)   Corrosion of Steel Reinforcement.

Corrosion of reinforcing steel in conrete exposed to a salt water environment is attributed [13,14] to unequal concentrations of sea salts which produce corrosion cells of the differential concentrations in the mortar result from a variable permeability - which investigators [15] have shown is the single most important physical characteristic of concrete (or cement paste) influencing the corrosion of reinforcing steel.

Oxygen is necessary for the corrosion process and it has been postulated that oxygen diffuses through the permeable concrete and establishes differential oxygen concentrations in two ways: differences in oxygen solubility in solutions of different dissolved solids content and greater availability of oxygen in those areas of lower water saturation. Variations in the dissolved solids content (salts from the seawater) of water in the capillaries in the concrete or mortar arise from wetting and evaporation which concentrate and redissolve salts in the water in the pores. The variations will be greatest at the water line and in the splash zone. Galvanic cells form with anodic areas either high salt/low oxygen or high water-saturation/low oxygen and the cathodic areas either low salt/high oxygen or low water-saturation/high oxygen. Potential differences of about 0.5 volts were found in one structure examined.

The presence of salts in the pores of the cement mortar paste also has the effect of decreasing the resistivity of the mortar which results in higher galvanic currents. It has been shown that corrosion of reinforcing steel below the waterline is virtually absent. This is possibly explained by the absence of oxygen, probably permitting the establishment of a highly protective film which prevents ferrous ions passing into solution.

Steel reinforcement completely encased in normal concrete requires a minimum coverage of only 1/2 in. in most natural environments, whereas structures exposed to corrosive water environments may require a cover of 2 to 3 in. For thin concrete elements such as building panels and ferrocement boat hulls, where the encasement material is a denser mortar but where the cover provided is very thin, the reinforcement, the mortar, or both, may require a protective coating. The effectiveness of galvanized coatings on the steel reinforcement to protect the steel has been studied by many investigators[16].

(c) Coatings for Concrete (Mortar).

It is considered that coatings applied to the concrete (mortar) may act as a waterproofing membrane to prevent entry of moisture, dissolved salts and oxygen into the structure and may also provide a dielectric barrier against stray currents. However, very little recorded experience is available on the efficacy of paint in preventing corrosion of steel reinforcements in concrete structures. One investigation of 40 circular wire-wound prestressed concrete water reservoirs reported that all reservoirs showed corrosion except two that had been painted with a polyvinyl acetate latex coating over a flood coat of raw linseed oil[17]. The unpainted tanks showed significant correlations between corrosion damage and thickness of mortar coat, wet areas, and mortar/wire bond. The investigator concludes that corrosion of stress wires may be reduced by sealing the exterior wall surface to reduce and equalize the oxygen supply to the wires. The investigator claimed that a coat of paint over mortar helps maintain equilibrium by reducing the availability of oxygen, restricting the access of positive ions, and preventing the leaching action of water. He explained that although imperfections in the coating will permit some air to reach the wire and create a local cathode area, most of the wire is anodic and the favourable ratio of anode to cathode areas makes minor imperfections of small consequence. If the cement mortar is uncoated, the wires will be in a highly cathodic state. An area where oxygen replenishment is restricted, such as a wet spot, will result in a local anode surrounded by a large local cathodic area. Accelerated corrosion at the unfavourable anode-to-cathode ratio site is likely. He

considered that a breathing type of polyvinyl acetate latex coating
was satisfactory for new reservoirs but that a more impermeable
coating may be required for older reservoirs where some corrosion
damage and some loss of alkalinity had already occurred. He further
concluded that a 3/4-inch layer of mortar is required over the
wires if the reservoir is to be painted.

The only "recent" article related solely to coatings for
concrete is the comprehensive guide prepared by the American Concrete
Institute in 1966 [18]. The article claims that there are only
20 basic film-forming materials available for use in coatings manu-
facture and that these vehicles largely determine the properties and
class name of coating. However, minor variations in formulations
can affect the performance of the coating. The survey points out
that the performance of any coating will be affected by the surface
preparation, the method of application, environmental conditions
at the time of application, and the film thickness.

Two up-to-date books, Technical Manual No. TM 5-618,
Paints and Protective Coatings and Organic Coatings, Properties,
Selection, and Uses [19,20], provide a broad survey of the many
conventional and more recently developed organic coatings and a
comprehensive guide to the selection and application of an adequate
coating system for most materials, including concrete.

In general, oil-base paints must be avoided because the
alkali in the mortar saponifies the oil-base paint unless long aging
periods permit the concrete to dry out, the surface alkalinity to be
neutralized, and the free, inner alkalinity to migrate out and be
gradually neutralized. The limited resistance of oil-based paints,
and their permeability to moisture, limits their applications in
structures where moisture behind the film can cause flaking and
blistering.

Latex (water-emulsion) paints of the styrene-butadiene,
polyvinyl acetate, and acrylic types are relatively insensitive to
water and resist alkalinity well. The nature of the water-base
latex paints allows application over damp (not wet) and unaged (not
uncured) mortar. However, the low resistance of latex paints to
corrosive environments generally restricts the use to general purpose
applications.

The synthetic resin coatings which have been used
successfully on concrete intended for severe service include the
epoxy resins, polyesters, polyurethanes, and various synthetic
rubbers. A number of other resins have shown some promise and paint
manufacturers are undoubtedly investigating formulations of these
and the above-mentioned types.

Ferrocement boat-building authorities have not put much emphasis on the need to paint ferrocement boats. Jackson and Sutherland [21] claim that painting can be eliminated if 0.05 inch of high quality mortar can be guaranteed over the steel mesh but that since plastering control is not always adequate, painting is recommended. They state that professional New Zealand practice has been to coat the outside of the hull with an epoxy resin coating to provide an additional water barrier between the sea and reinforcement near the surface and to bind any surface movement of the fine particles of sand thereby improving resistance to abrasion. They claim that anti-fouling paints are as useful for underwater sections of concrete hulls as for wood or steel hulls. Samson and Wellens [22] advise the use of two coats of thiocol-based epoxy resin to seal any stray ends of mesh, followed by anti-fouling paint on the hull bottom. Whitener [23] recommends that hulls for use in salt water be coated with epoxy resin finishes. He includes a list of substances which attack concrete, a list of protective treatments, and a helpful list of manufacturers of coatings and components of coatings extracted from "Effect of Various Substances on Concrete and Protective Treatments", published by the Portland Cement Association in 1968. Benford and Husen [24] offer no advice on painting.

The chief binders or vehicles which appear to have application for coating ferrocement hulls are described as follows:

(i) Inorganic binders.

Inorganic binders composed of sodium, lithium or ethyl silicates react with zinc metal dust to form very hard films which are extremely resistant to corrosion in humid or marine atmospheres. Tests have shown that zinc-rich paints containing high percentages of zinc dust (92-95 percent of dry film weight) afford some sacrificial anodic protection to steel until the build-up of corrosion products stifles the galvanic action.

(ii) Polyester resin-based coatings.

Polyesters are synthetic alkyd resins made by the esterification of a polyhydric alcohol with a polybasic acid. The polyester resin coating used in this study is a highly modified resin containing chlorinated paraffins and low molecular weight polystyrenes, hydraulic cement, red lead, and aluminum flake. Polyesters show considerable promise as a durable finish for concrete as well as for metal in marine environments.

(iii) Vinyl resin-based coatings.

The vinyl resins include chiefly the polymers and copolymers of vinyl acetate, vinyl chlorides, and vinylidene chloride, and the polyvinyl acetates and alcohols derived from polyvinyl acetate. Polyvinyl chloride resin can be modified for use in solvent-type coatings by copolymerization with vinyl acetate. The copolymers can be formulated into solution-type air-drying coatings. The coatings have alkali resistance which makes them suitable for use over concrete and also a resistance to weather and alternate or continuous immersion in salt water which makes them suitable for marine applications. Because of their generally low solids content, vinyl finishes require several coats for adequate dry film thickness.

(iv) Epoxy-amide based coating.

The epoxy-polyamide resins are obtained by blending epoxy resins with reactive polyamide resins which act as both curing agent and modifier. Coatings from the resulting blend are reported to have good hardness, flexibility, toughness, resistance to alkali, resistance to water, and good adhesion to concrete and other surfaces. They are considered to have better resistance to continuous immersion in water than do straight epoxies. Epoxy-polyamide coatings are recommended for marine applications.

(v) Chlorinated rubber-based coatings.

Chlorinated rubber-based paints are made from chlorinated natural rubber dissolved in chlorinated aromatic hydrocarbons. The paints adhere well to concrete and provide good resistance to water and many chemicals. They have been widely used for exterior applications and for humid applications such as swimming pools.

2. Experimental Work.

(a) Preparation of Mortar Substrate.

Although the above-mentioned article and books list over fifty classes of thermoplastic and thermosetting coatings, the present study was limited to those kinds available and recommended as potentially useful by the industrial representatives of two international and national paint manufacturers.

Half-panel portions of 30 x 30-in. ferrocement panels made some two years earlier were used for the application of various paint coatings. All panels had been made with Type II cement, with similar water/cement and cement/sand ratios, and with similar trowelling techniques. The wire mesh reinforcement was not the same in all panels but all reinforcement was covered with a 1/4-in. layer of mortar. The mesh pattern could be seen in the back (mould) side of all panels.

The mortar surface of the half-panels were prepared for painting as follows:

(i)   The surface was ground with a flexible abrasive disc (F885F Aloxite 50 Fastcut resin disc) to smooth the surface and remove any cement film.

(ii)   The surface was etched by scrubbing with 200 cc of 30 percent (or 15-percent where specified) muriatic acid until all bubbling ceased, then re-etching for 30 seconds. (Panels 13 and 14 were not etched.)

(iii)   The panels were washed copiously with water and thoroughly dried at 100 F overnight.

(iv)   The surface was dusted with a clean cloth before the paint was applied by brushing.

The paint coatings included an inorganic two-component self-curing ethyl silicate zinc-rich primer (as primer and top coat), a polyvinyl chloride-based enamel (as primer and top coat), a two-component chemical-curing polyamide resin over a zinc-rich primer, a chlorinated rubber-based enamel, a highly modified polyester resin coat, and various top coat primer combinations. The average film thickness of all systems as measured with a calibrated microscope ranged from 4 to 6 mils.

The complete list and description of the 12 systems prepared are provided in Table 5. Exposure tests began after a 14-day cure at the normal room conditions existing in the laboratory. The original appearance of the samples is recorded in Fig. 17.

(b)   Exposure Testing.

(i)   Control Specimens.

Specimens of each kind were allowed to remain in the normal atmospheric environment of the laboratory for

control examination. The assessment of the coatings after 4 months is presented in Table 6.

(ii)   Marine Tidal Exposures.

Duplicate 3 x 3-in. specimens from the 12 panels were prepared for exposure in a marine environment subject to marine organisms. The 12 specimens, one from each pair, were set into a shallow pan of paraffin wax and enclosed in a 1/2-in. mesh screen cage. The set of specimens was exposed at the Vancouver Kitsilano Station of the Canadian Coast Guard at mean tide level to give two cycles of seawater immersion and drying per day. The other specimen of each pair from the 12 panels was similarly prepared. This set of specimens was placed at the level of low low tide for complete immersion at all times. The appearance of the specimens was observed at regular intervals. The observations of the specimens after 84 days (168 tide cycles) are recorded in Table 7. The specimens exposed at mean tide and at low low tide were identical except that the former contained a slightly heavier slime coat. The appearance of the specimens before exposure and after 84 days is shown in Fig. 17 and 18.

(iii)   Laboratory Seawater Exposures.

Duplicate 3 x 3-in. specimens were sawn from the 12 painted half-panels for exposure in a cyclic exposure apparatus. One of each pair of specimens was dipped in hot paraffin wax to a depth of about 5/8-in., enough to cover exposed ends of the reinforcing mesh wires. The 24 specimens set in the apparatus were subjected to 600 exposure cycles (one hour of wetting in filtered seawater and three hours of air-drying in front of an air fan). The specimens were observed at regular intervals and their appearance after 600 cycles is recorded in Table 8 and Fig. 19.

(iv)   Weather-Ometer.

Duplicate 3 x 6-in. specimens were exposed in a standard Weather-Ometer apparatus. Each cycle consists of 102 min of ultraviolet light plus 18 min of ultraviolet light and water spray. The temperature is maintained at 140 F. An exposure period of 250 hr is equivalent to one year of outside mid-temperate climate. The exposure period at the time of writing was 1500 hr, equivalent to 6 years. The appearance was observed and recorded at

regular intervals. Flaking of the coating in one of the
specimens from panel 16B was observed after an equivalent
exposure of 5 years, Fig. 20 and 21. The observations
after 1500 hr (6 years) are recorded in Table 9.

3. Rating of Paint Systems Tested.

The condition of the coated specimens after 120 days in a
normal indoor environment, 600 cycles of wetting and drying in a seawater
cycling apparatus, after 84 days in a marine environment, and after
1500 hr (equivalent to 6 years) in the Weather-Ometer apparatus, was
rated on the basis of: visual appearance (craze-cracking and flaking;
ease of separating coating from substrate with a razor blade; spall
resistance to two close scores with a penknife; and resistance to gouging
with a pointed probe. The composite rating was based on: Good 2, Fair 1,
and Poor 0, for the four test conditions and the four evaluation tests.

The specimen-paint systems obtaining over 80 percent of
the maximum points include:

Specimen No. 3A   2 coats inorganic ethyl silicate zinc rich
                   primer.

"   No. 9A   1 coat highly modified polyester resin
                   containing portland cement, red lead, and
                   aluminum flake over mortar pretreated with
                   zinc silico-fluoride.

"   No. 9B   1 coat highly modified polyester resin
                   containing portland cement, red lead, and
                   aluminum flake over mortar with no zinc
                   silico-fluoride pretreatment.

"   No. 11B   2 coats two-component pigmented epoxy
                   resin.

Those obtaining 60 to 80 percent include:

Specimen No. 7   1 prime and 2 top coats of chlorinated
                   rubber-based paint.

"   No. 11A   1 prime and 2 top coats of polyvinyl
                   chloride-based enamel.

"   No. 16A   1 prime and 1 top coat of two-component
                   polyamide resin paint.

Those obtaining 40 to 60 percent include:

Specimen No. 6 1 clear epoxy seal coat and 2 top coats of vinyl resin anti-fouling paint.

" No. 12B 1 coat of inorganic ethyl silicate zinc-rich primer and 1 top coat of polyvinyl chloride based enamel.

" No. 14 1 prime and 2 top coats of chlorinated rubber-based paint (no acid etch treatment).

Those obtaining less than 40 percent include:

Specimen No. 12A 1 coat of inorganic ethyl silicate zinc-rich primer and 1 top coat of two-component polyamide resin.

" No. 16B 1 prime and 2 top coats of chlorinated rubber-based paint.

   The zinc-rich primer and metal pigmented polyester resin coatings appear to provide the best "serviceability" but these coatings have poor aesthetic chracteristics. The two-component epoxy resin, chlorinated rubber-based, polyvinyl chloride-based, and polyamide resin-based paints all appear to give reasonably good performance but the relatively poor performance of the polyamide and of another formulation of chlorinated rubber-based paint on other specimens is somewhat disconcerting. It is not known whether or not small differences in the condition of the substrate, e.g., smoothness, is responsible.

F. CONTROL OF INTERNAL QUALITY.

   The preparation of specimens for the several tests in this and previous studies has exposed many internal defects in apparently sound panels. The defects are chiefly of two kinds. Unsoundness from incomplete penetration of the mortar when trowelling from both sides is probably the most serious defect. If this occurs the hollow centre of the panel fails to develop a mortar/rod bond, and the structure delaminates readily under flexural and impact stresses, greatly reducing the stiffness and strength. Unsoundness from internal gas voids is also a serious defect. Hydrogen evolves at galvanic cells set up between galvanized mesh and ungalvanized rod reinforcement in the presence of wet mortar. The gas produces voids along the ungalvanized rods which may or may not manifest themselves in external blisters or eruptions. It has been shown that gas voids markedly reduce the mortar/rod bond strength with

a resultant decrease in stiffness and modulus of rupture.

There is a great need to assess the mortared quality of vessel hulls in situ and preferably before the mortar has set. Operational control of the raw materials and conformance to cement/sand and water/ cement ratios cannot guarantee the quality of the finished product, i.e., the plastered hull. Compaction, penetration, and curing affect both the strength of the mortar and of the hull structure. Non-destructive testing must become part of any inspection and certification program.

Radiography could most usefully reveal internal defects, incomplete penetration and voids but complete radiographic inspection would be an extremely costly procedure. However, it might be used at stem, rib, and other corner sections which are difficult to penetrate with mortar.

The success attained by ultrasonics as a non-destructive tool for revealing internal defects in forgings, castings, and machinery parts suggests that it might be useful as a tool for detecting flaws in ferrocement structures. It has been used to detect flaws and to determine the strength of concrete both in massive structures and in reinforced beams although most strength tests have been performed on laboratory samples.

Special instruments have been designed for testing concrete in highways and more massive structures. One of these is a pulse flaw-detector called the Betonoskop (or concrete-scope). Others are Model UCT designed by the Road Research Laboratory, Hammondsworth, Middlesex, U.K. and built by A.E. Cawkell Electronic Engineers, Southall, U.K.; a model built by Mullard Ltd. Equipment Division, London, U.K.; and Ausculteur Dynamique marketed by Laboratoire Electro-Acoustique (L.E.A.) Rueil (S. & O.) in France.

Although ferrocement has a much finer grain size than normal concrete containing coarse aggregate, it does contain moderately coarse rods and several layers of finely divided mesh which may cause considerable scatter. Preliminary tests using an ultrasonic instrument with probes and frequencies normally used for the detection of cracks and flaws in machine parts failed to produce suitably sharp reflections.

The development of techniques for appraising the quality, both in inherent mortar strength and in structural soundness of ferrocement during plastering and after setting and curing, will require a research program of considerable size. Most standard texts on concrete discuss the use of ultrasonics in quality control. Several books[25-27] on ultrasonic testing, which provide very useful basic data on the kinds of transmitters and receivers, the use of attenuation and velocity techniques,

and applicable frequencies for concrete testing procedures, can be used
as an initial basis for a research program.

G. REVIEW OF LITERATURE.

Much literature has been examined during the course of
this study. The literature related to the behaviour of concrete under
repeated stresses and that related to paint coatings has been cited
in the relevant sections.

Only a few articles on the specific subject of ferro-
cement have appeared in print. Shah and Key[28] have explored the
influence of different mortar compositions, yield strength of mesh, and
different surface areas of reinforcement on the impact resistance of
ferrocement plates. Their findings are as follows:

1. Replacement of natural sand with lightweight framed
   sand did not influence the tensile strength of
   ferrocement.

2. Increasing the ductility of steel increased the
   width of cracks formed under impact loads and hence
   the rate at which water flowed through impacted
   plates.

3. Superior cracking performance was obtained in tensile
   and impact specimens of ferrocement with reinforce-
   ment of higher specific surface.

Naaman and Shah[29] examined the influence of mesh, size,
and fraction volume of reinforcement on cracking and ultimate behaviour
of ferrocement specimens subjected to uniaxial tension. Their findings
show the relationship between mesh parameters and crack spacing. However,
the following relationships related to the "first crack" condition appear
to be of greater interest for the design of ferrocement vessels:

(a) For one type of mesh, higher volumes of the mesh
    reinforcment showed first cracking at higher stresses.

(b) Increasing the specific surface of reinforcement
    increases the stress at the onset of cracking.

(c) The modulus of elasticity of the composite at first
    crack increases with the specific surface of the
    reinforcement.

(d)   The lower values of the modulus of elasticity at
first crack can be predicted with a knowledge of
moduli of mesh and the volume fraction of longi-
tudinal reinforcement.

The width and distribution of cracking under stress
has always been an important feature of ferrocement construction.
Greater emphasis is being placed on the onset of cracking as a criterion
of failure. Walkus[30,31] has reported on experimental investigations
into the main factors which influence the cracking and elongation of
ferrocement. Other workers, already cited in another part of the report,
have also considered the cracking behaviour of reinforced concrete under
flexural and impact loadings.

The work by Christensen and Williamson[32] has explained
the galvanic cell/hydrogen evolution problem encountered by this
investigation and others when fresh portland cement mortar is applied
to galvanized mesh on ungalvanized (black or bright) steel rods.
Christensen and Williamson found the problem of hydrogen bubble eruptions
during horizontal fabrication of panels. This investigator found a
serious eruption problem which required subsequent slicking and trowelling
even in panels made in an upright position. The authors verified and
quantized the chemical remedy of using chromium trioxide (CrO_3) known
by Bresler and Cornet and recommended[33] for the preservation of
galvanized reinforcement in concrete cast against ungalvanized steel forms.
They showed that CrO_3 added in very dilute concentrations, 100 to 300 ppm
by weight to the mix water, would prevent hydrogen evolution and increase
the rod/mortar bond strength and the stiffness and ultimate strength of
ferrocement in bending.

Significant work especially in the mathematical analysis
of ferrocement composites continues to be undertaken in universities
such as Laval[34] and Sir George Williams[35].

The properties of short steel fibre-reinforced mortars
continue to be examined by researchers such as Kar and Pal[36]. The
American Concrete Institute has recently prepared a state-of-the-art
report[37] on the subject.

The use of concrete and ferrocement for offshore and under
seas applications has been discussed by Haynes et al[38] and Shah et al[39].

Note: The bibliography of books and papers directly related to
ferrocement has been maintained in a manner consistent
with the listings included in previous reports on this
subject for the Fisheries Service. It is consecutively
numbered from that in the previous report and is located
at the end of section References. All articles and
books in the bibliography are on file at B.C. Research.

H. FERROCEMENT RESEARCH NEEDS.

Many millions of dollars have been spent on studies related to the behaviour and development of the more conventional materials for hull construction. These and other studies on ferrocement have just scratched the surface of the requirements.

The development and improvement of the component parts of the ferrocement composite, i.e., mortar and reinforcement, requires much study. The study of polymer mortars and of admixtures to mortars are important areas. The possibility of using other types of steel mesh and steel rod reinforcement, of glass and other non-metallic reinforcements, and of short metallic fibres, requires much study.

The areas in which the present studies have been chiefly concerned are the engineering properties of "typical" ferrocement construction and the practical aspects of making and ensuring sound construction. Much work remains to be done, e.g.:

1. Engineering Data.

These studies have examined the behaviour of a few typical ferrocement composites under impact and bending loads and very briefly under cyclic bending. Much more work is required to obtain additional engineering data and to develop a mathematical model which will allow more general application of the determined test results to many constructions. A rational development of standardized test specimens is required. For example, the minimum length of specimen for a bend test must be determined so that the load/deflection characteristic is not affected by premature slippage of the reinforcing rods.

2. Hull Quality.

Sonic, ultrasonic, and other techniques should be studied to assess applicability as means to indicate the presence of internal air and gas pockets, incomplete penetration, and other evidence of unsoundness both in simple hull sections and at T- and L-section joints where penetration is particularly difficult. For maximum benefit to the builder, the techniques should be able to be used before the mortar sets. Techniques for hardened mortar will aid the inspectors who must grant certification.

3. Hull Protection.

Some useful comparative data have been obtained on how various coatings resist marine environmental exposures. Work is required

to determine how well the coatings protect the mortar and reinforcement by studying:

- the penetration of salt water through the coating membrane under pressure,
- the behaviour of both mortar and reinforcement after exposure,
- the effect of defects, pinholes, cracks, scrapes in the coating,
- the effect of crack width on ingress of corrodent,
- galvanic attack.

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79	Whitener, J.R.	Ferro-Cement Boat Construction.	Book. Cornell Maritime Press, Inc., Cambridge, Maryland, 1971.	x	x	x		x	x		x
80	Shah, S.P., and W.H. Key, Jr.	Impact Resistance of Ferro-Cement.	J. Structural Division, Proc. American Society of Civil Engineers, Vol 98, No. ST1, Jan. 1972, 111-123.	x	x	x		x	x		x
81	Naaman, A.E., and S.P. Shah.	Tensile Tests of Ferro-Cement	J. American Concrete Institute, Proc. Vol 68, No. 9, Sept. 1971, 693-698.	x	x	x		x			x
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TABLE 1. LOAD AND FIBRE STRESS IN BEAMS FROM PANEL 50 IN REPEATED BENDING

| Specimen No. | Approximate Bending Load | | No. of Cycles to First Observed Crack |
	Load lb	Fibre Stress psi	
50-1	500	1400	Not broken after 106,000 cycles.
2	500	1400	>12,000 <20,000
3	750	2100	>12,000 <20,000
4	750	2100	Not broken after 106,000 cycles.
5	1000	2800	<50
6	1000	2800	<50
7	900	2500	>6,000 <14,000
8	900	2500	>6,000 <14,000

TABLE 2. FIBRE STRESS AND DEFLECTION IN BEAM SPECIMENS FROM PANEL 51.

No. of Cycles	51-3 Fibre Stress psi	51-3 Defln. mils	51-4 Fibre Stress psi	51-4 Defln. mils	51-5 Fibre Stress psi	51-5 Defln. mils	51-6 Fibre Stress psi	51-6 Defln. mils	Remarks
0	2,130	14	2,610	9	1,990	8	1,930	9	Fine cracks in 51-3
2,000	1,620	14	2,210	9	1,930	8	1,820	9	
3,600	1,620	14	2,210	9	1,930	8	1,930	8	
8,000	1,620	11	2,180	9	1,790	8	1,600	9	
18,000	1,320		1,910	9	1,340	6	1,400	9	
32,000	950	13	1,600	8	1,260	4	1,450	8	Fine crack in 51-4
46,000	810	13	1,480	8	1,150	6	1,340	7	
75,000	670		1,180	9	1,060	5	1,230	7	
80,000	640	8	1,120	9	840	7	980	7	
110,000	500	9	1,070	10	730	4	930	6	
154,000	480	10	980	10	730	6	950	5	
200,000	420	10	950	10	620	5	810	6	

TABLE 3. FIBRE STRESS AND DEFLECTION IN BEAM SPECIMENS FROM PANELS 205, 201, AND 49.

No. of Cycles	205-2 Fibre Stress psi	Defln. mils	201-2 Fibre Stress psi	Defln. mils	49-7 Fibre Stress psi	Defln. mils	49-3 Fibre Stress psi	Defln. mils	49-2 Fibre Stress psi	Defln. mils	49-1 Fibre Stress psi	Defln. mils
0	210		700		2550		2240		1070		1340	
3,600	140		590		2120		1900		870		950	
9,200	140	19	640	3	1940	12	1850	13	1000	5	1280	7
26,000	140	18	620	3	1400	12	980	13	950	4	1340	5
38,000	140		620		1180		760		980		1250	
44,000	140		630		1320		840		920		1120	
64,000	140	18	640	3	1200	8	760	11	920	5	1200	6
94,000	140	22	590	3	1150	8	790	11	670	5	1160	5
120,000	140	22	630	3	1200	7	730	11	840	5	840	4
150,000	140	16	590	3	1120	10	640	10	840	5	1060	3
170,000	140	16	590	3	1000	9	590	11	840	4	920	4
200,000	140	17	590	3	960	10	560	11	840	5	870	6
250,000	140	24	590	3	1040	10	560	12	840	5	1000	4
280,000	140	15	560	3	1120	5	640	11	810	6	1060	7

TABLE 4. RESULTS OF BOLTING TESTS.

(single 1/2-in. bolts in 4-in. wide (A,B) and 8-in. wide (C,D) specimens)

Panel Construction	Specimen Type	Load at First Visible Crack, lb	Max. load held, lb	Mode of Failure
1/2-16 ga welded square mesh (2 layers)	A	(2400)	3300	transverse, vertical
	B	2600	2800	transverse
	B	(2800)	3400	transverse
	B	3000	3900	transverse
	C	3600	4200	vertical, transverse
	D	4100	6000	transverse, vertical
1/2-19 ga hardware cloth (5 layers)	A	(2200)	2700	transverse, vertical
	A	2800	3300	transverse
	A	(2300)	2750	transverse
	A	2100	2600	transverse
	B	2800	3200	transverse
	B	(2600)	2800	transverse
	B	(3000)	3850	transverse
	C	(3200)	3600	vertical, transverse
	C	3300	3600	vertical, transverse
	D	(5100)	6000	transverse, vertical
	D	5000	5400	transverse, vertical
	E(A)	1600	2630	transverse
	F(B)	1500	2450	transverse
1/2-22 ga hexagonal mesh	A*	800*	1200*	transverse
	A	(2000)	2600	transverse vertical
	B	(1700)	2500	transverse
	B	1800	2600	transverse
	C	3000	3600	transverse, vertical
	D	4000	5200	transverse, vertical

* tensile loading transverse to direction of mesh.

TABLE 5. PAINT SYSTEMS EXPOSED

Half-Panel No.	Etched in HCl	Description of Paint System Used	Total Coating Thickness mils
3A	Yes	Primer - Inorganic two-component self-curing ethyl silicate zinc-rich primer. Dried 6 hr in normal laboratory environment. Top coat - same as primer coat.	6
6	Yes	Primer seal coat - Two-component clear epoxy finish. Dried 8 hr in normal laboratory environment. Top coats - vinyl resin base anti-fouling paint. Dried 4 hr. - vinyl resin base anti-fouling paint.	4
7	Yes	Primer seal coat - Chlorinated rubber-based paint thinned with 15 percent thinner. Dried 8 hr. Top coats - Chlorinated rubber-based paint. - Chlorinated rubber-based paint.	4
9A	Yes	Primer seal coat - Zinc silico-fluoride solution. Dried 16 hr. Top coat - Steelmate* - a highly-modified polyester resin vehicle containing metal and metallic oxide fillers.	4
9B	Yes	Primer seal coat - None. Top coat - Steelmate (as in 9A).	4
11A	Yes	Primer seal coat - Polyvinyl chloride-based enamel coating thinner with 15-percent vinyl thinner. Dried 2 1/2 hr. Top coats - Polyvinyl chloride-based enamel. Dried 2 1/2 hr. - Polyvinyl chloride-based enamel coating.	5
11B	Yes	Primer seal coat - None. Top coats - Two-component pigmented epoxy-resin coating. Dried 5 1/2 hr. - Two-component pigmented epoxy-resin coating.	4

*Steelmate is a proprietary formulation of B.C. Research
 developed especially for underwater application on steel.

TABLE 5. (cont'd).

Half-Panel No.	Etched in HCl	Description of Paint System Used	Total Coating Thickness mils
12A	Yes	Primer seal coat – Inorganic two-component self-curing ethyl silicate zinc-rich primer. Dried 6 hr. Top coat – Two-component clear chemical curing polyamide resin.	6
12B	Yes	Primer seal coat – Inorganic two-component self-curing ethyl silicate zinc-rich primer. Dried 6 hr. Top coat – Polyvinyl chloride-based enamel thinned with 30-percent vinyl thinner.	6
13	No.	Primer seal coat) – As in panel No. 6. However top coats broke badly and specimens were Top coats) not exposed to environment tests.	
14	No	Primer seal coat) – As in panel No. 7. Top coats)	4
16A	Yes	Primer seal coat – Clear chemical curing polyamide resin. Dried 24 hr. Top coat – Two-component clear chemical curing polyamide resin.	5
16B	Yes	Primer seal coat – Chlorinated rubber-based enamel thinned with 15-percent compatible thinner. Dried 24 hr. Top coats – Chlorinated rubber-based enamel. Dried 24 hr. – Chlorinated rubber-based enamel.	4

TABLE **6.**   ASSESSMENT OF COATINGS AFTER 4 MONTHS IN ROOM ENVIRONMENT
(not exposed to weather or service environment)

Specimen No.	Visual Appearance	Separation Resistance (Lifting with razor blade)	Spall Resistance (Scoring with penknife)	Gouge Resistance (Pointed tool)
3	Fine crazing.	Separates easily. Brittle flakes.	Spalls and crumbles.	Fair
6	No defects.	No separation at interfaces.	Spalls.	Fair
7	" "	No separation at interfaces.	No spalling.	Good
9A	" "	No separation at interfaces.	No spalling.	Good
9B	" "	No separation at interface.	No spalling. Debris tenacious.	Good
11A	" "	Separates with difficulty.	No spalling. Debris tenacious.	Good
11B	" "	Separates with difficulty.	No spalling. Debris tenacious.	Good
12A	" "	Separates fairly easily. Brittle.	Spalling. Debris tenacious.	Good
12B	" "	Top coat separates from primer. Tough.	Spalls. Debris tenacious.	Fair
14	" "	Separates with difficulty.	Spalls slightly.	Fair.
16A	" "	Separates with difficulty.	No spalling. Debris tenacious.	Good
16B	" "	Top coat separates from primer. Brittle.	Spalling. Debris brittle.	Poor

TABLE 7. ASSESSMENT OF COATINGS AFTER 84 DAYS MARINE EXPOSURE AT MEAN TIDE
(Vancouver - Kitsilano)

Specimen No.	Visual Appearance	Separation Resistance (Lifting with razor blade)	Spall Resistance (Scoring with penknife)	Gouge Resistance (Pointed tool)
3	Slime layer – otherwise clean as original.	No separation at interface.	Slight spalling. Debris crumbly.	Good
6	"	Spalls with difficulty.	Spalls. Debris slightly crumbly.	Fair
7	"	No separation at interface.	No spalling. Debris slightly crumbly.	Good
9A	"	"	"	"
9B	"	"	Slight spalling. Debris slightly crumbly.	"
11A	"	Separates as tough film.	Slight separation. Debris tenacious.	Good
11B	"	Separates only with difficulty.	"	"
12A	"	Spalls at interface.	Spalls fairly readily.	"
12B	"	Top coat separates from primer as tough film.	Spalls fairly readily. Debris crumbly.	Fair
14	"	No separation.	Slight separation. Debris tenacious.	Fair
16A	"	Spalls readily.	Spalls readily.	Good
16B	"	" "	" "	Poor

TABLE 8.   ASSESSMENT OF COATINGS AFTER 600 CYCLES IN SEAWATER
(1 hr immersed - 3 hr drying in moving air)

Specimen No.	Visual Appearance	Separation Resistance (Lifting with razor blade)	Spall Resistance (Scoring with penknife)	Gouge Resistance (Pointed tool)
3	Heavy brown bloom. No surface defects.	No separation at interfaces.	No spalling. Debris crumbly.	Good
6	No surface defects. Yellow stain.*	No separation at interfaces.	Spalls. Debris slightly crumbly.	Fair
7	" "	No separation at interface.	No spalling. Debris slightly crumbly.	Fair
9A	" "	No separation at interface.	No spalling. Debris tenacious.	Fair
9B	" "	No separation at interface.	No spalling. Debris tenacious.	Fair
11A	" "	Easy separation at mortar interface.	Spalling. Debris tenacious.	Fair
11B	" "	Separates with difficulty.	No spalling. Debris tenacious.	Fair
12A	" "	Separates fairly easily. Brittle.	Spalling. Debris tenacious.	Fair
12B	Coating lifted at one corner.	Top coat separates from primer as tough film.	No spalling. Debris tenacious.	Poor
14	No surface defects.	Separates with difficulty.	Spalls slightly. Debris tenacious.	Poor
16A	" " "	Separates with difficulty.	No spalling. Debris tenacious.	Fair
16B	" " "	Top coat separates from primer.	Spalling. Debris brittle.	Poor - spalls.

*All specimens exhibited yellow staining.

TABLE 9. ASSESSMENT OF COATINGS AFTER 1500 HR (6 YR) IN WEATHER-OMETER

Specimen No.	Visual Appearance	Separation Resistance (Lifting with razor blade)	Spall Resistance (Scoring with penknife)	Gouge Resistance (Pointed tool)
3	Fine craze cracking. Yellow encrustation.	No separation at interface.	No spalling. Debris crumbly.	Good
6	Fine craze cracking. Yellow stain.*	Separates cleanly from interface but with difficulty.	Spalls cleanly in flakes.	Fair
7	Fine craze cracking and flaking.	Separates easily and cleanly at interface.	Spalls easily and cleanly.	Poor
9A	Fine craze cracking. Clean surface.	No separation at interface.	Spalls slightly. Debris flaking and crumbly.	Fair
9B	Fine craze cracking. Clean surface.	No separation at interface.	Spalls slightly. Debris flaking and crumbly.	Fair
11A	Good surface.	Separates readily at interface in continuous film.	No spalling. Tough film.	Good
11B	Good surface.	Separates only with difficulty.	No spalling. Tough film.	Good
12A	Fine craze cracking.	Separates readily at interface with spalling.	Spalls easily and cleanly.	Fair
12B	A few craze cracks	No separation at interface.	Spalls slightly. Debris crumbly.	Fair
14	Fine craze cracking and flaking.	Separates easily and cleanly at interface.	Spalls easily and cleanly.	Poor
16A	A few fine craze cracks.	Separates easily and cleanly at interface.	No spalling. Tough film.	Good
16B	Craze cracking, curling, film loss.	Separates cleanly with difficulty at interface.	Spalls easily and cleanly.	Poor

*All specimens except 9A and 9B exhibited yellow stain.

42.
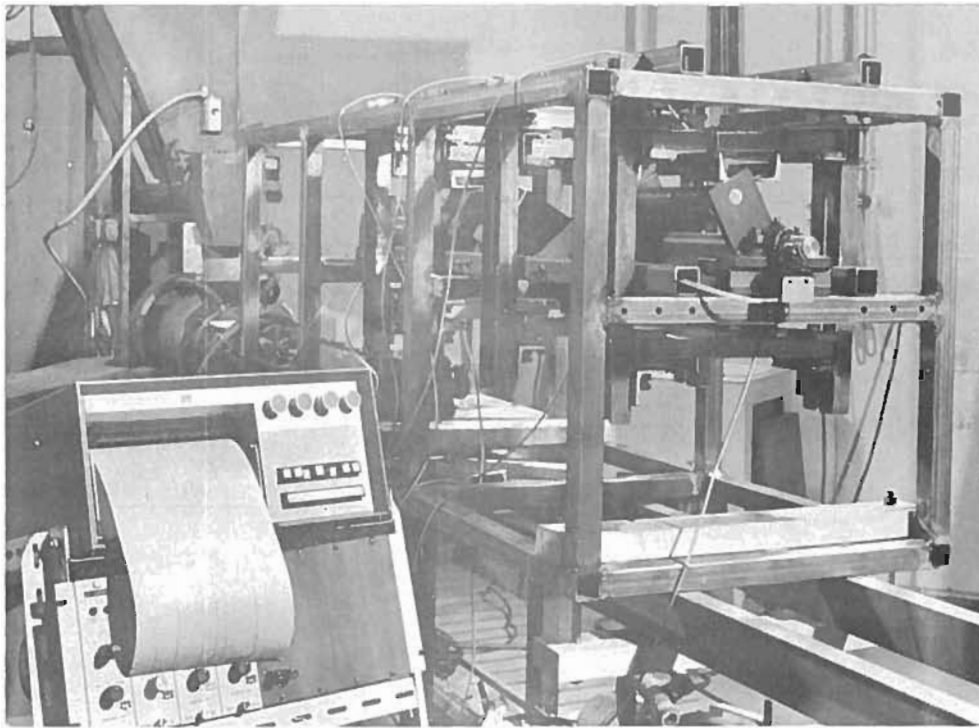
Fig. 1. Cyclic flexure test apparatus and recorder.
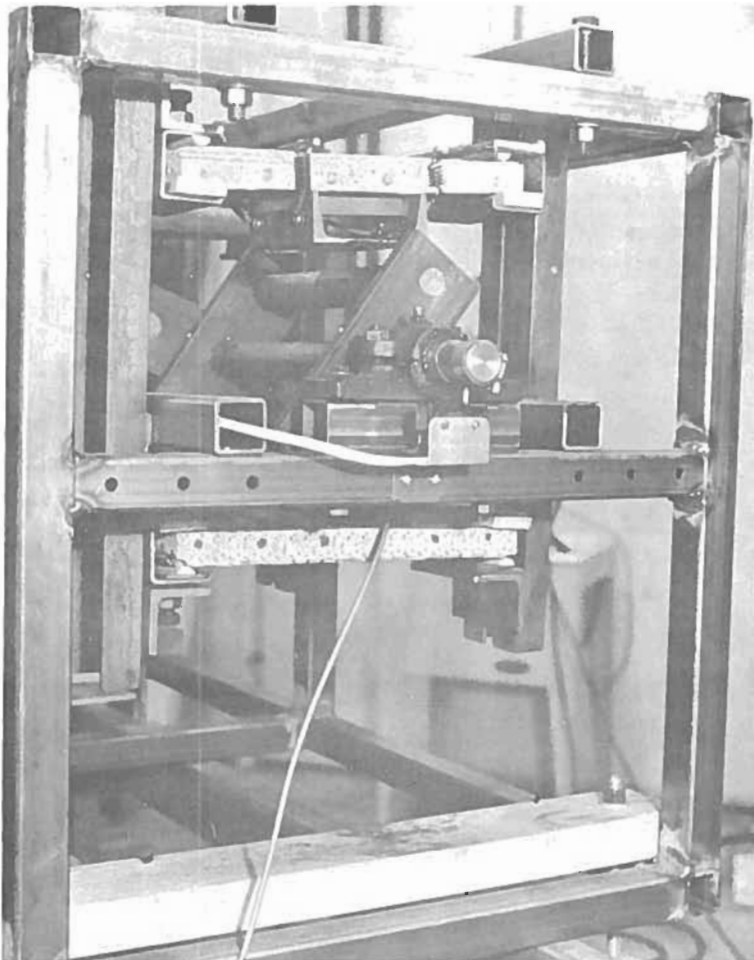
Fig. 2.

Beam specimens
and load cells
in cyclic flexure
test apparatus.
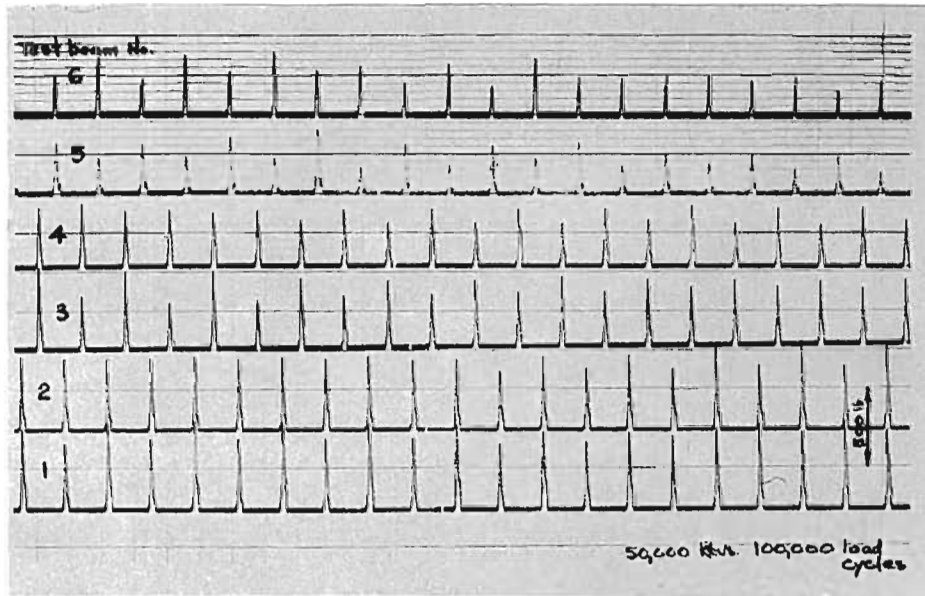
Fig. 3. Load-cell readout for six flexure beam specimens.
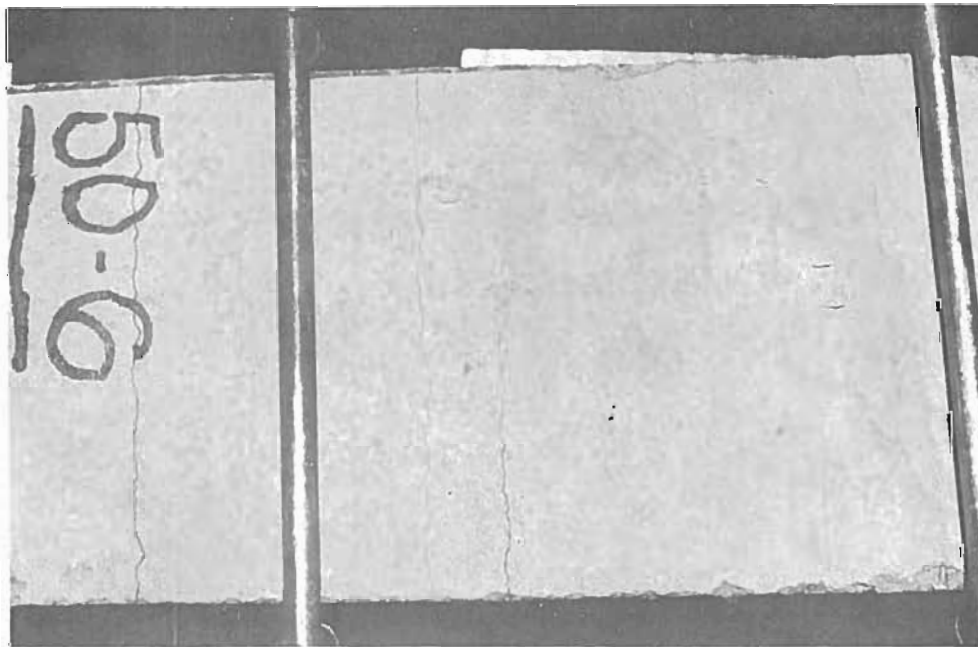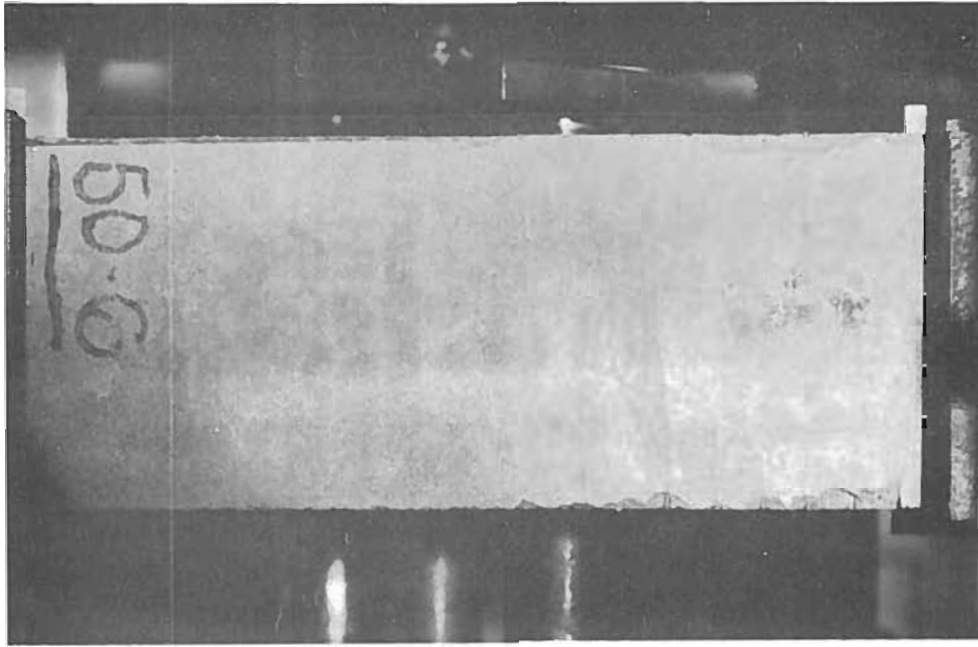
Fig. 4.   Specimen 50-6 before and after 6000 cycles
of bending loads.
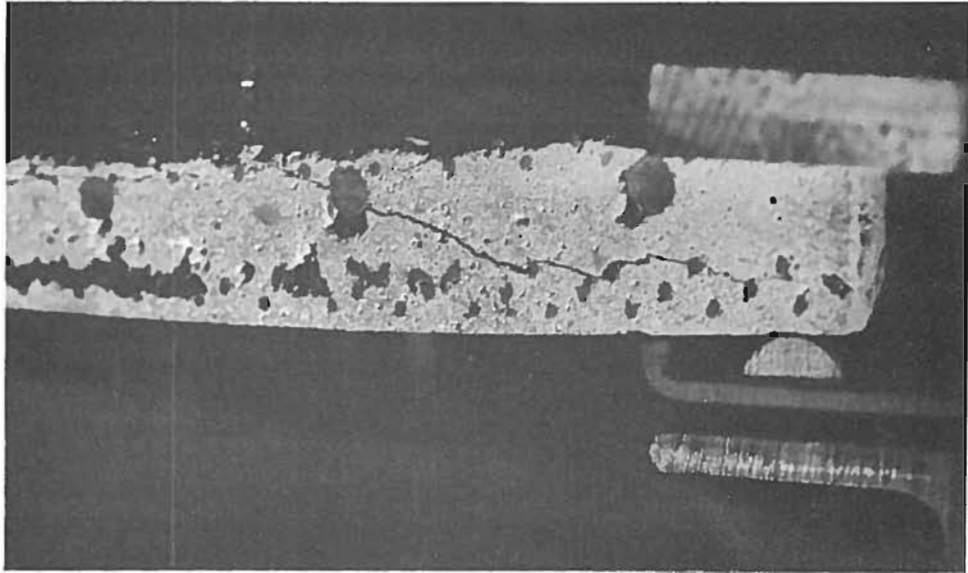
Fig. 5. Diagonal crack in specimen 50-5 between outside
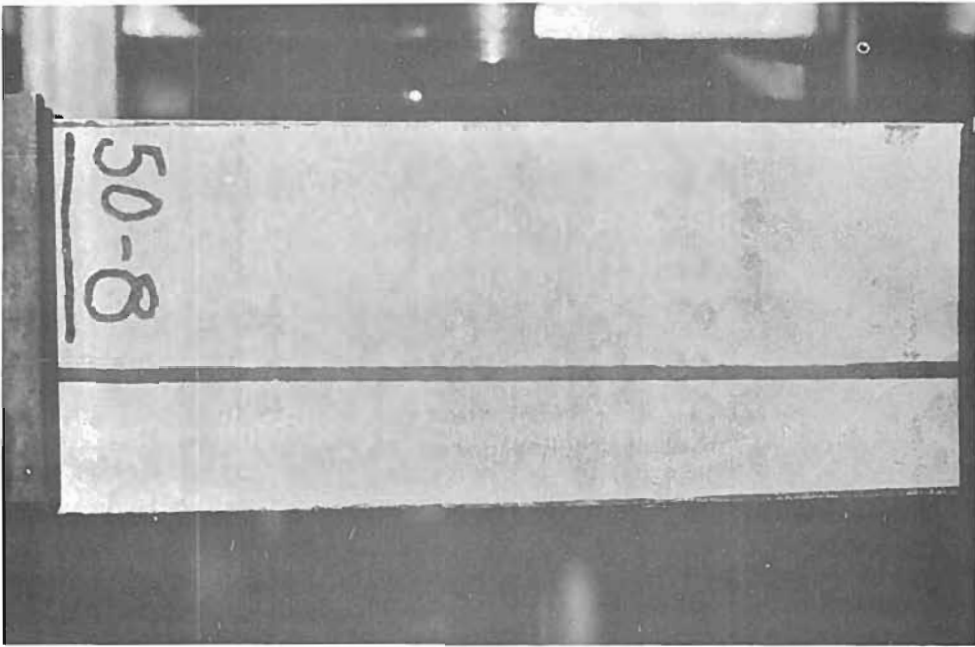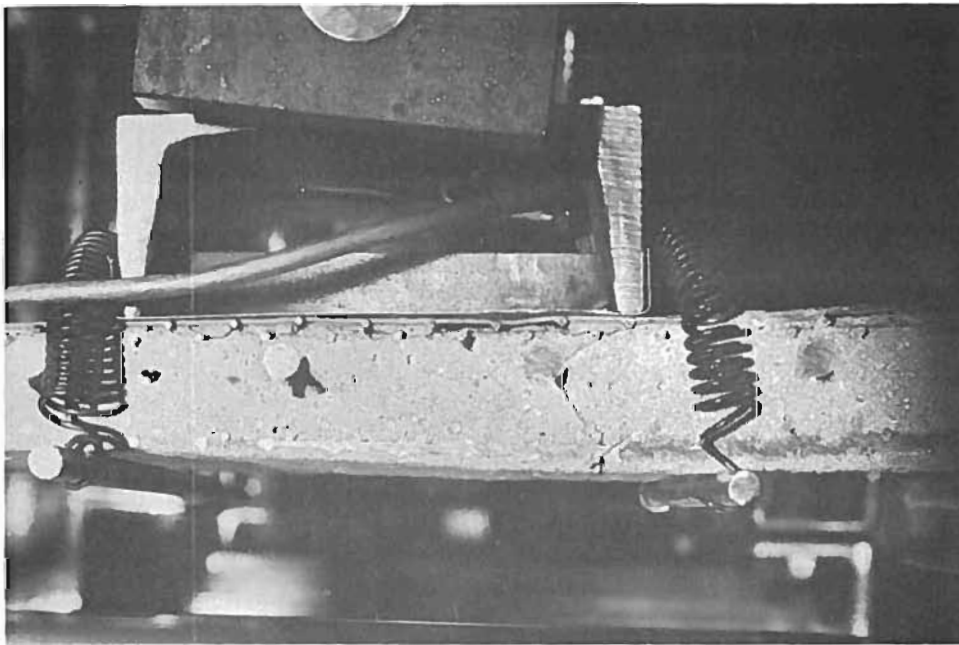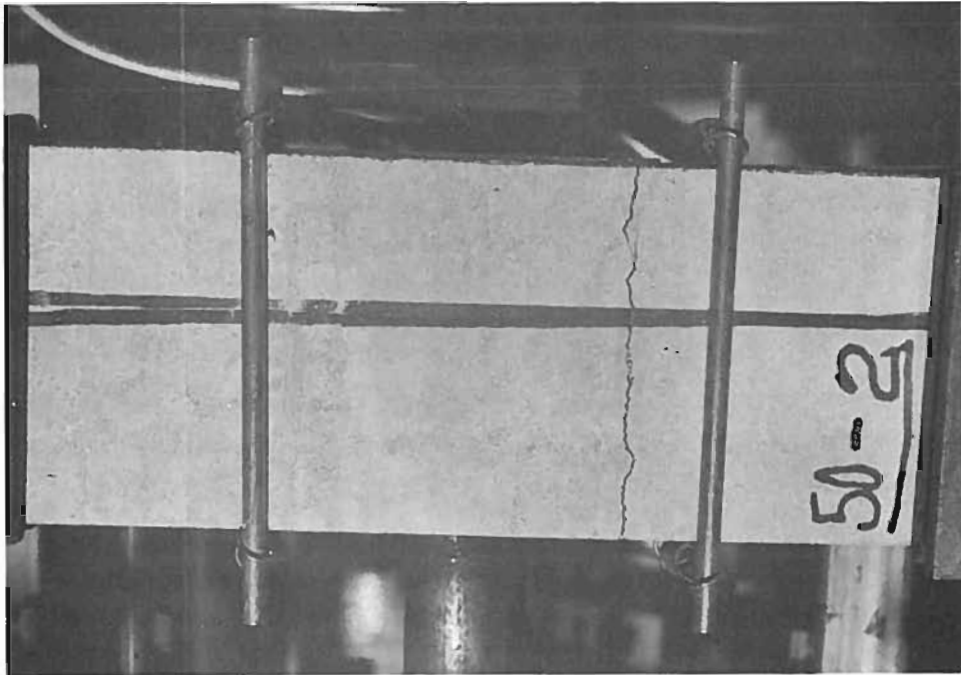load support and third point loading.
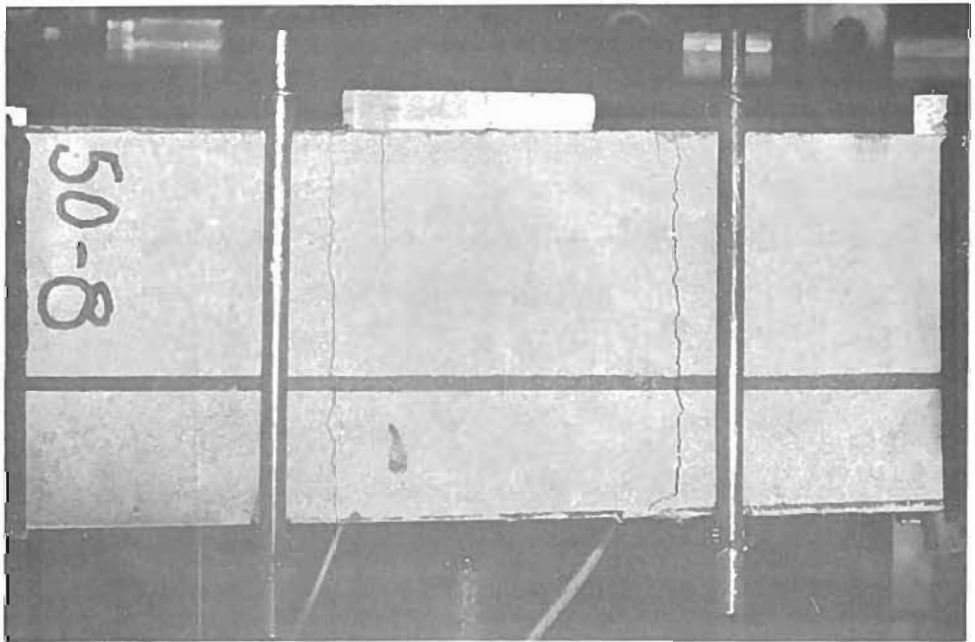
Fig. 6. Specimen 50-8 before and after 100,000 cycles of bending loads.

Figs. 7 and 8. Bottom crack in specimen 50-2 and deviation
towards transverse rod.
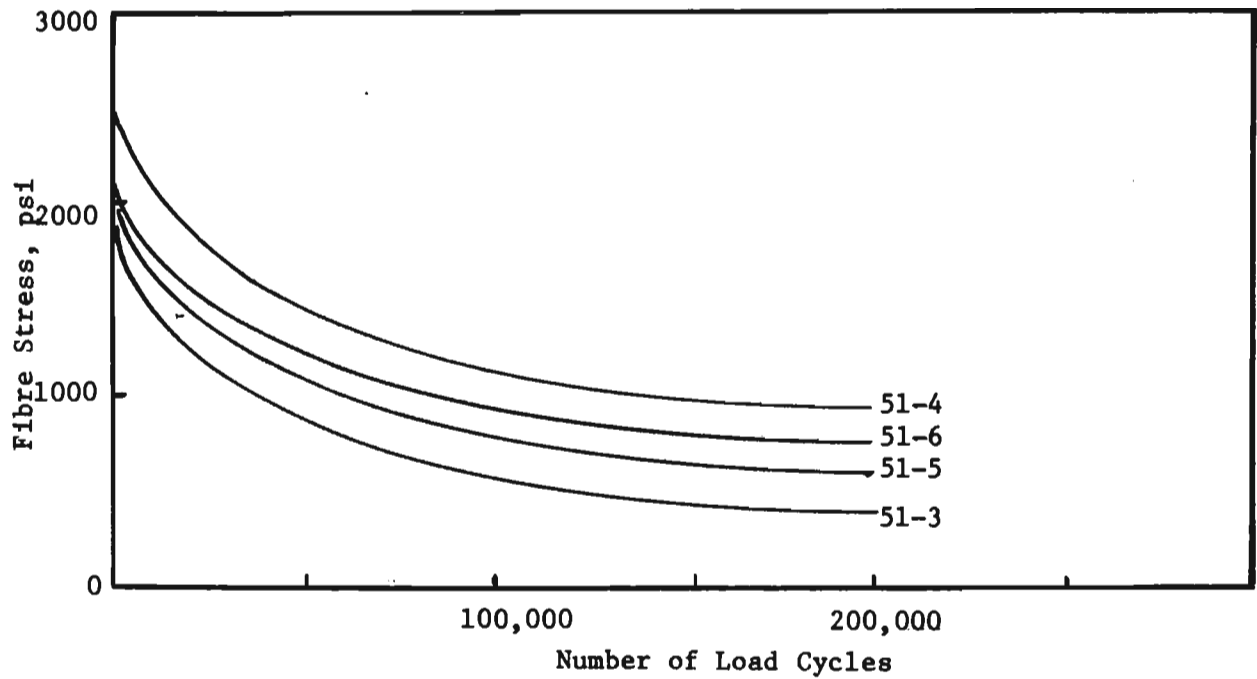
Fig. 9. Characteristic fibre stress curves for beams from panel 51 under repeated bending.
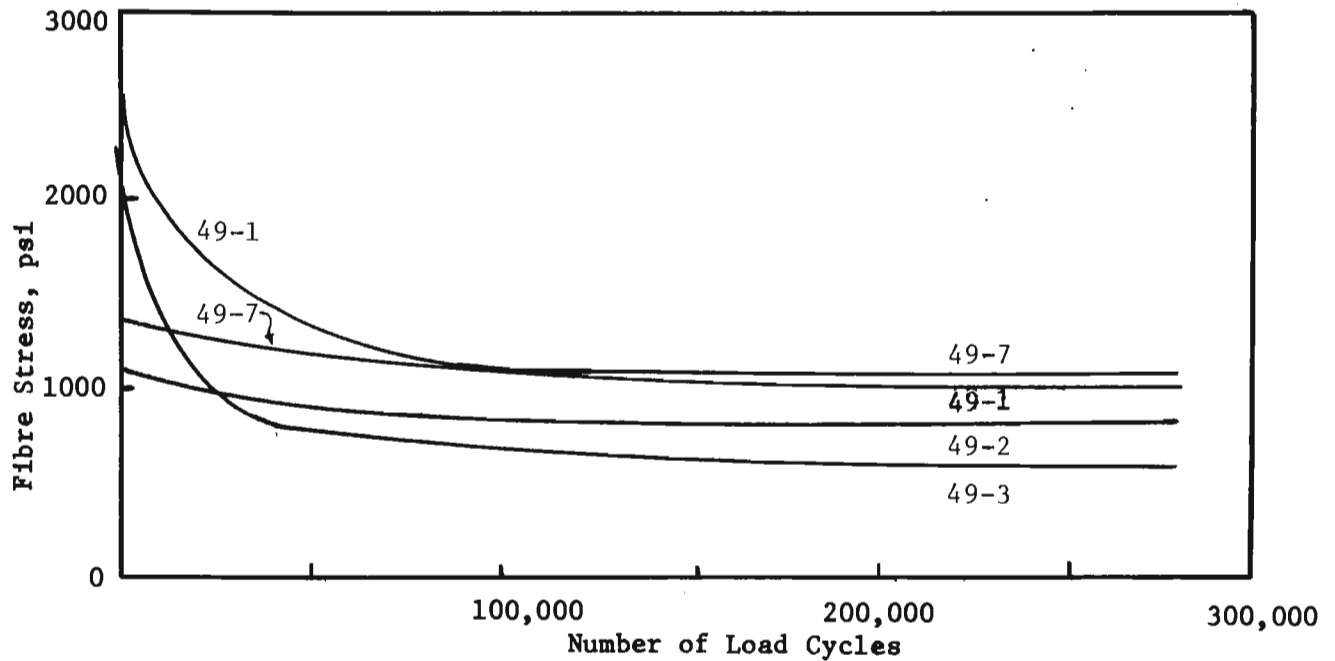
Fig. 10. Characteristic fibre stress curves for beams from panel 49 under repeated bending.
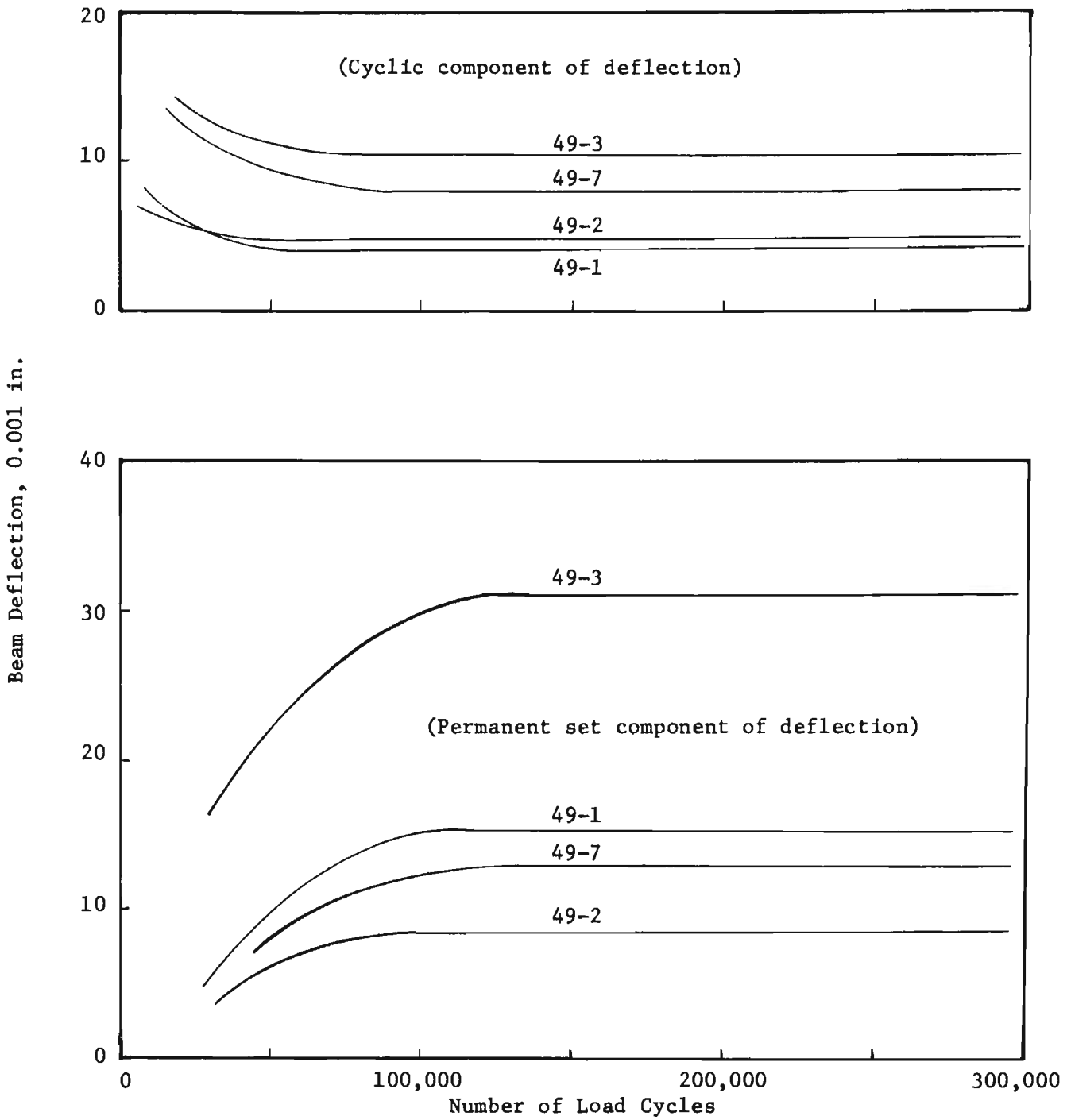
Fig. 11. Characteristic deflection curves for beams from panel 29 under
repeated bending loads.
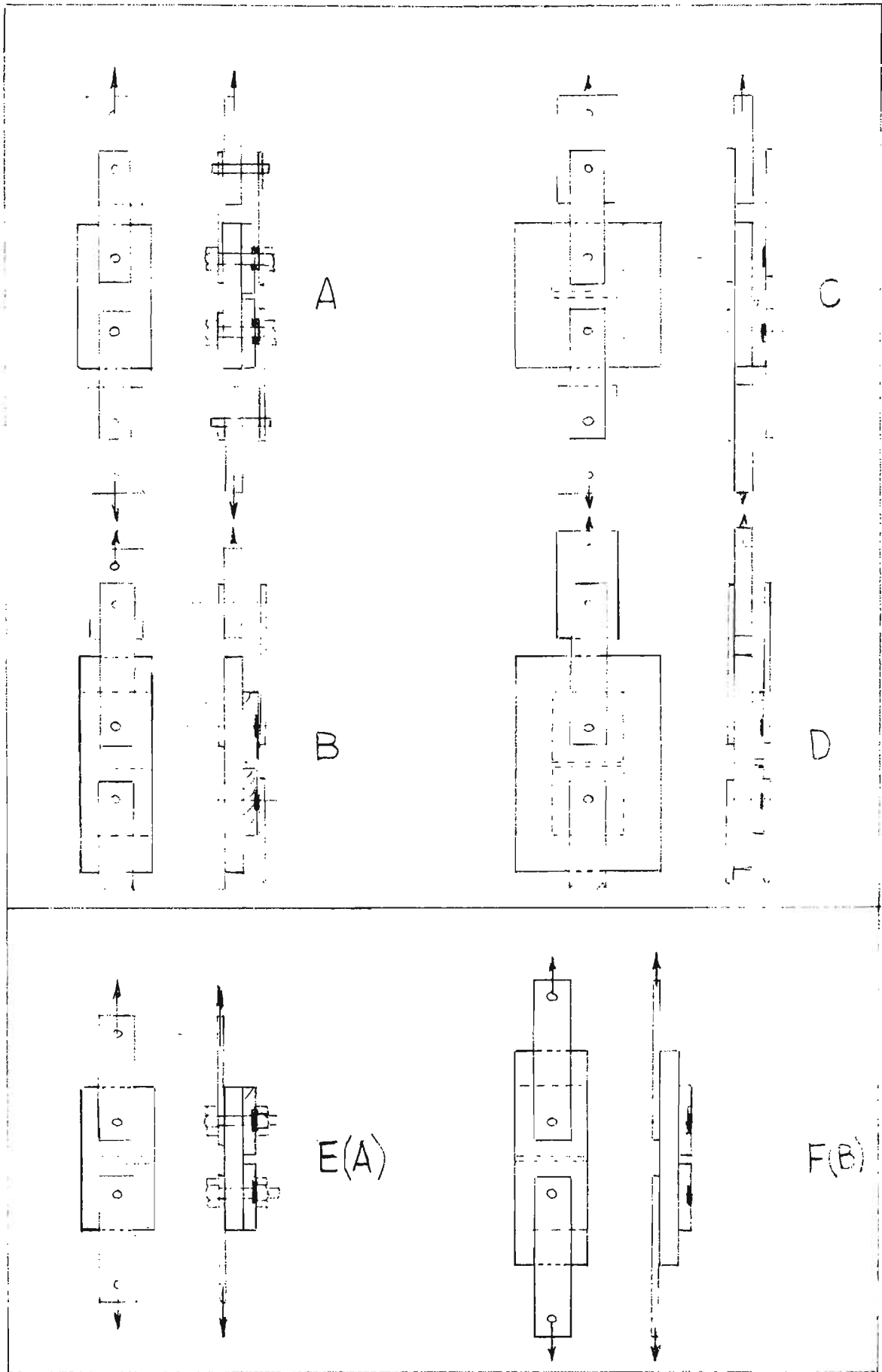
Fig. 12. Loading geometry for specimen types A, B, C, and D (approximately coplanar) and for specimen types E(A) and F(B) (offset).

Fig. 13.

Bolting failure in Type A specimens.
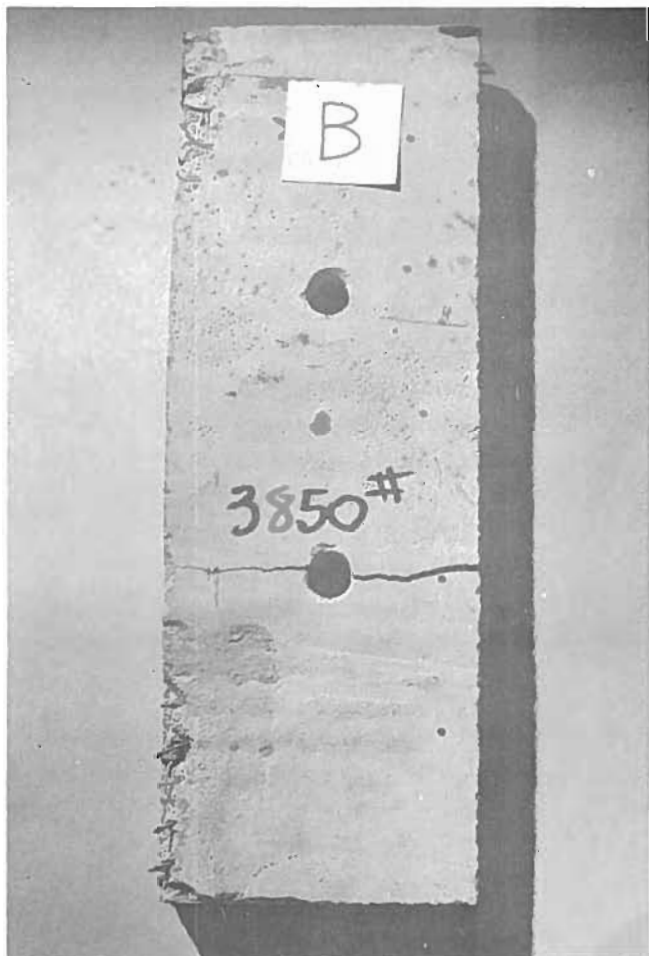
Fig. 14.

Bolting failure in Type B specimens.

Fig. 15.

Bolting failure in Type C specimens.

Fig. 16.

Bolting failure in Type D specimens.
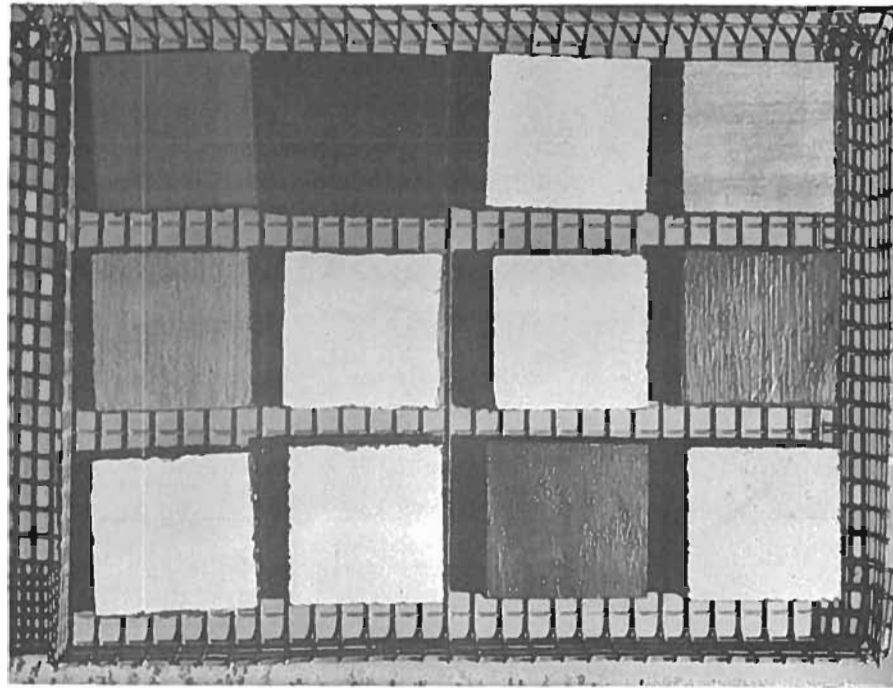
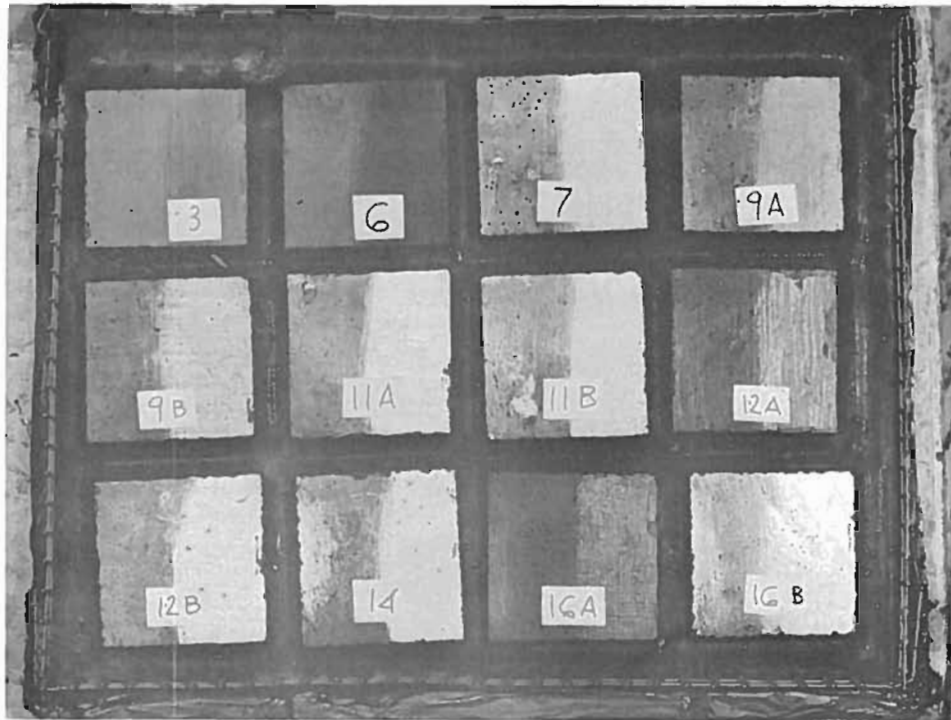Fig. 17. Original appearance of painted ferrocement specimens (use code of Fig. 18 and Table 5 to identify).

Fig. 18. Appearance of specimens after 84 days marine exposure at mean tide. Slime coat partly removed.
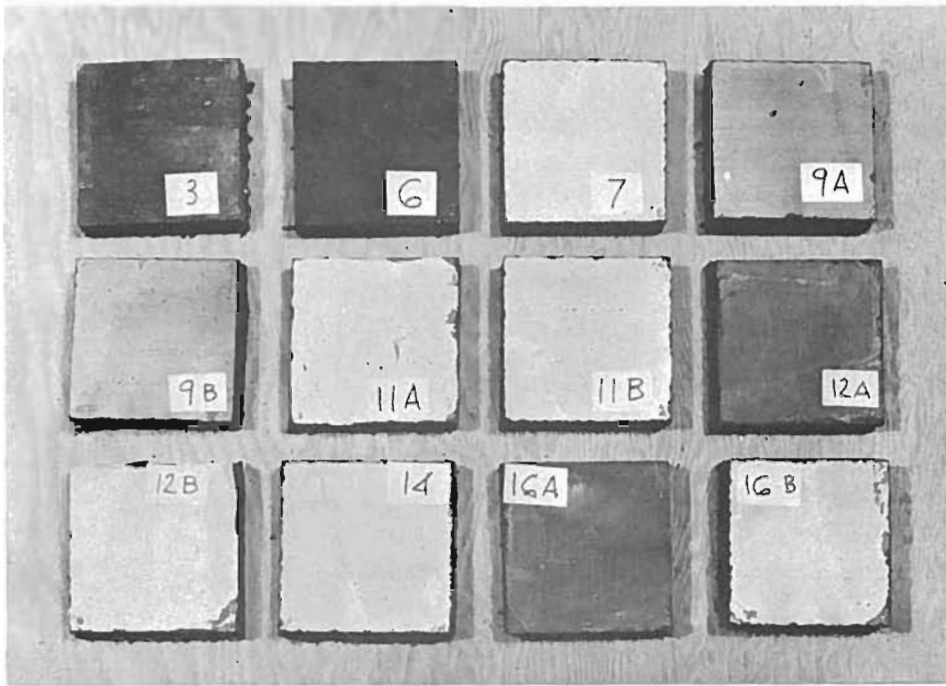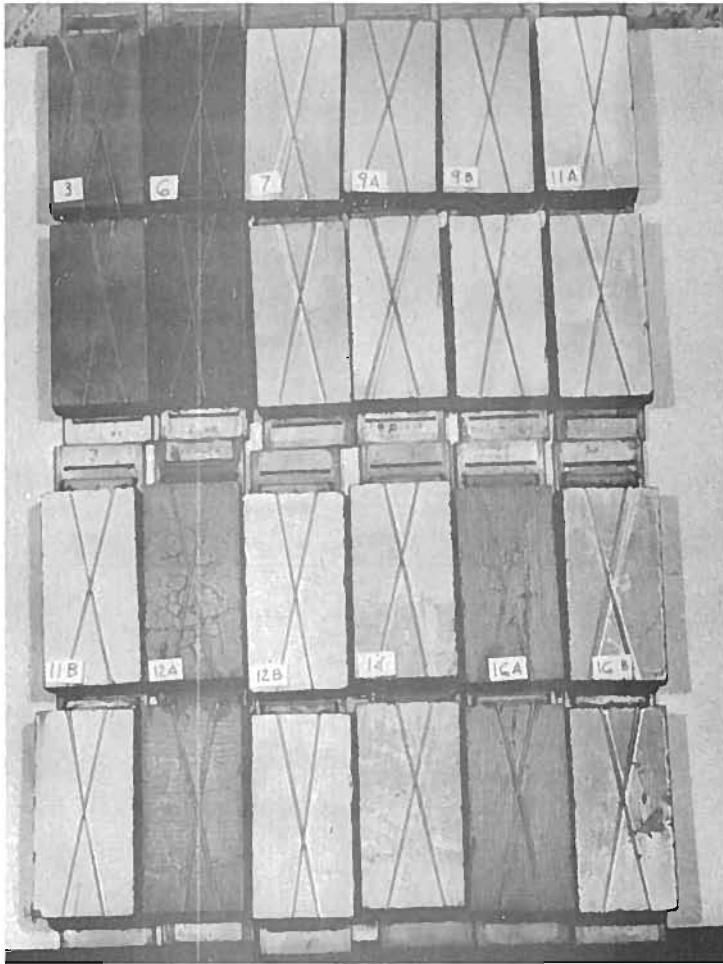
Fig. 19.   Appearance of specimens after 600 wet/dry cycles
           in a laboratory tank of seawater.

Figs. 20 and 21.

Specimens removed from Weather-Ometer after 1500 hrs exposure.