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ABOUT IFIC

The **International Ferocement Information Center (IFIC)** was founded in October 1976 at the **Asian Institute of Technology** under the joint sponsorship of The Institute's Division of Structural Engineering and Construction and Library and Regional Documentation Center. **IFIC** was established as a result of the recommendations made in 1972 by the U.S. National Academy of Sciences' Advisory Committee on Technological Innovation (ACTI). IFIC receives financial support from the **Canadian International Development Agency (CIDA)** and the **International Development Research Center (IDRC)** of Canada.

Basically, **IFIC** serves as a clearing house for information on ferrocement and related materials. In cooperation with national societies, universities, libraries, information centers, government agencies, research organizations, engineering and consulting firms all over the world, **IFIC** attempts to collect information on all forms of ferrocement applications either published or unpublished. This information is identified and sorted before it is repackaged and disseminated as widely as possible through **IFIC** 's publications, reference and reprographic services and technology transfer activities. All information collected by **IFIC** are entered into a computerized data base using ISIS system. These information are available on request. In addition, **IFIC** offers referral services.

A quarterly publication, the *Journal of Ferrocement*, is the main disseminating tool of **IFIC**. **IFIC** has also published the monograph *Ferrocement*, do it yourself booklets, slide presentation series, state-of-the-art reviews, ferrocement abstracts, bibliographies and reports. FOCUS, the information brochure of **IFIC**, is published in 19 languages as part of **IFIC**'s attempt to reach out to the rural areas of the developing countries. **IFIC** is compiling a directory of consultants and ferrocement experts. The first volume, *International Directory of Ferrocement Organizations and Experts 1982-1984*, is now being updated.

To transfer ferrocement technology to the rural areas of the developing countries, **IFIC** organizes training programs, seminars, study-tours, conferences and symposia. For these activities, **IFIC** acts as an initiator; identifying needs, soliciting funding, identifying experts, and bringing people together. So far, **IFIC** has successfully undertaken annual training programs since 1984; a regional symposium and training courses in India; a seminar to introduce ferocement in six countries in Asia and eight countries in Africa; study-tour in Thailand and Indonesia for African officials; the Second International Symposium on Ferrocement and a short Course on Design and Construction of Ferrocement Structures, and the Ferrocement Corrosion: An International Correspondance Symposium. **IFIC** has successfully established the Ferrocement Information Network (FIN), the **IFIC** Reference Centers Network and the **IFIC** Consultants network. **IFIC** has promoted the introduction of ferrocement technology in the engineering and architecture curricula of 144 universities in 51 countries. Currently, **IFIC** is involved to strengthen the outreach programs of the nodes of FIN.

EDITORIAL

IFIC and FIN are working towards effective technology transfer of ferrocement technology to developing countries. IFIC and FIN promote ferrocement as a technology that can be use for basic needs satisfaction and for employment generation. They stress that the process of technology transfer is linked with social - economic aspects and with the government organization appropriate to handle the technology. It is important to adapt the technology to local conditions and local technological level.

Transfer of technology is achieved through information dissemination and training courses. IFIC and FIN conducts training courses in three different levels : decision maker level; implementors level and end-user level. These training courses are undertaken with financial assistance from international aid organizations and from local government agencies. The experiences of FIN and IFIC in these training activities are now documented in the new publication entitled *Methodology for Training* . The guidelines have been prepared to help FIN nodes organize training courses effectively and efficiently. The guidelines also include tips for resource persons to enable them to upgrade their training and teaching capabilities.

Information dissemination is undertaken through personal contact, answer- to- queries and publications. The *Journal of Ferrocement* is the main disseminating tool of IFIC. In this issue, the experiences in Brazil to improve durability of ferrocement will help local engineers in other countries to adapt the technology to their environment. The authors stressed that technology transfer must consider social and cultural impacts to be effective.

The building up of a local technological capability is necessary and determines the effectiveness of the transfer. The papers on the behavior on the weldmesh ferrocement composite under cyclic loads; the flexural impact damage of ferrocement; ferrocement and replica ships; and the use of ferrocement panels in large span roofing system will be usefull in building up this technological capability.

We urge our reader to share with us and other users their experiences in the transfer of ferrocement technology.

The Editor

Behavior of Weldmesh Ferrocement Composite Under Flexural Cyclic Loads

Xiong, G. J.* and Singh, G.*

A new qualitative mechanistic model which is thought to reflect the behavior of ferrocement in flexural fatigue more realistically is presented. In light of this model and test results, the rectangular stress distribution assumption is found to be better for estimating steel stress when designing a weldmesh ferrocement against fatigue. This design is facilitated by using the fatigue behavior of only the reinforcement tested in the air. It is also shown that the dominating design criterion is not the crack width but the steel stress for all common structures.

INTRODUCTION

Under cyclic loading failure may be defined by the number of load repetitions to: (a) complete fracture or (b) allowable crack width or (c) allowable deflection. This paper deals with the first two.

Analytical models devised to predict cyclic behavior should be based on mechanistic approaches calibrated by phenomenological studies. Herein, a new qualitative mechanistic model of ferrocement in flexural fatigue is presented first. The significance of this model lies in the inclusion of the practical influence of increase of the height of cracks and the length of debonding of mortar-steel interface under cyclic loading. In light of this, it is found that the peaks of steel stress at cracked sections decrease with increase of the load cycles. The stress calculated by the existing elastic-cracked-section analysis is found to approach the theoretical upper boundary. The authors then propose a simple new method for predicting the value of the theoretical lower boundary of steel stress by the rectangular stress distribution assumption. This new method is thought to be more reliable and simpler for designing ferrocement using the fatigue behavior of only the wire tested in air.

Based on test data and theoretical analysis, it is also pointed out that for common structures the dominating design criterion is not the crack width but the steel stress.

This investigation endeavors to provide a clearer understanding of the interactions between the two phases of this composite. A simpler and reliable model has been developed which leads to an economical design of singly reinforced weldmesh ferrocement under flexural cyclic load.

THE NEW MECHANISTIC MODEL

It has been reported that the fatigue life of reinforcement in the air is lower than that calculated by elastic-cracked-section analysis of the ferrocement [1-3]. The authors take the view that one of the main reasons is that with the increase of load cycles, the actual peak steel stress at the cracked sections as against the calculated peak stress, decreases. This can be proved through the following study of the interaction between the wires and the mortar at different load cycles (Fig. 1).

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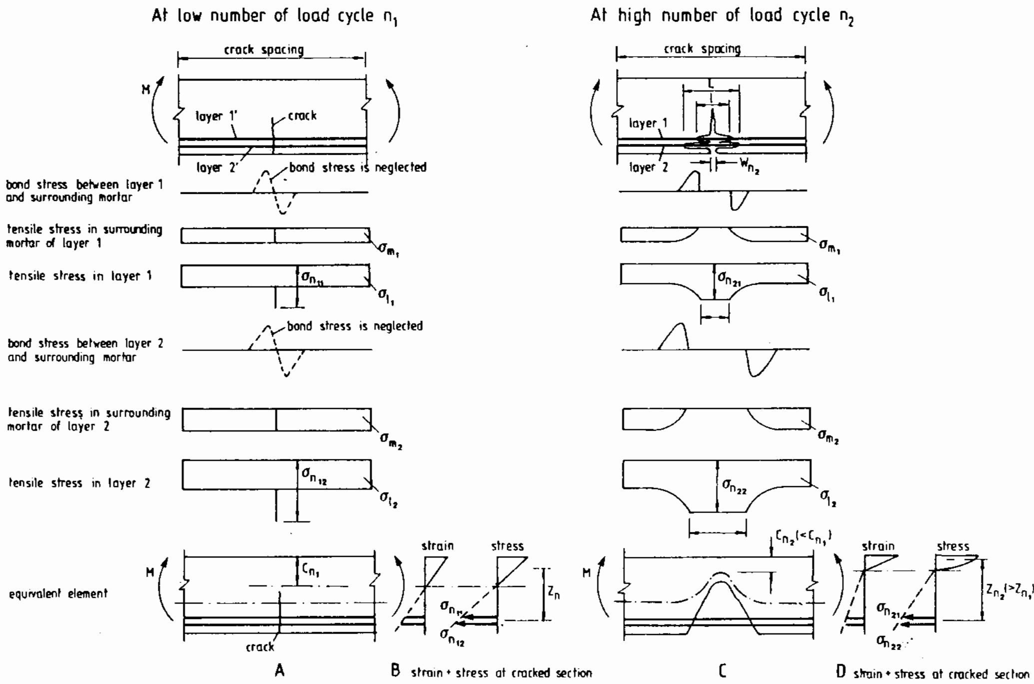

Fig. 1. Qualitative model of ferrocement in flexural cyclic load.

Crack Development Related to the Changes in Compressive Depth

As shown in Fig. 1A and 1B, the plane deformation assumption in which plane sections before bending remain plane after bending is assumed to be applicable to the beam when load cycle is equal to n_l. Plane deformation implies that the strains in mesh and in mortar are directly proportional to the distance from the neutral axis and that the mesh and mortar are bonded together perfectly; bond slip caused by bond stress is neglected. Crack width, therefore, tends to be zero and the steel stress reaches maximum value at cracked sections (σ_{nll} in layer 1 and σ_{nl2} in layer 2). The elastic-cracked-section analysis based on plane deformation assumption, therefore, is applicable for estimating the peak steel stress at cracked sections. In the figure C_{nl} is the compressive depth.

As shown in Fig.1C and 1D, when the load cycles increase to n_2 from n_l the crack develops both in width and height, the neutral axis moves up and the compressive depth decreases from C_{n1} to C_{n2} due to debonding between reinforcement and the surrounding mortar (debonding length of layer 1 and 2 are shown as l and L respectively). Plane deformation assumption is not applicable. The member, now, can be seen as an element with local regions of softening leading to a decrease in its overall stiffness (termed as equivalent element).

Stress Redistribution

Based on the above and the relationships between stress and strain for mortar and steel wire, the stress distribution can be estimated as shown in Fig. 1B and 1D at lower and higher load cycles. Because the height of the compressive zone decreases from C_{nl} to C_{n2}, the force lever increases from Z_{nl} to Z_{n2}..

Moment of Resistance

Assuming that the actual moment applied (in constant load test) is constant, and bearing in mind that force lever Z_{n2} at load cycle n_2 is longer than Z_{n1} at load cycle n_l, it can be concluded that the peak steel stresses (σ_{n2}) at load cycle n_2 are lower than those (σ_{nl}) at load cycle n_l

It means that the use of elastic-cracked-section analysis leads to over estimation of the peak steel stress, the actual stress in steel wire decreases with increase of cycles of load.

In fact the test results [1-6] show that the average strain of wire in tension zone increases and not decreases with load cycles. At first sight this may appear to contradict the above derivation. This is because the decrease in the steel stress occurs only in the region near the cracked sections. The stress in the wire outside this region increases with cycles of load because of the decrease of stiffness of mortar and bond failure of mortar-steel interface [4, 8] as shown in Fig. 2. In the figure the continuous and dashed curves express the stress distribution at cycle n_l and cycle n_2 respectively. This model leads us to improve the existing method for predicting the cyclic characteristics of the composite from the cyclic characteristics of the reinforcement and mortar.

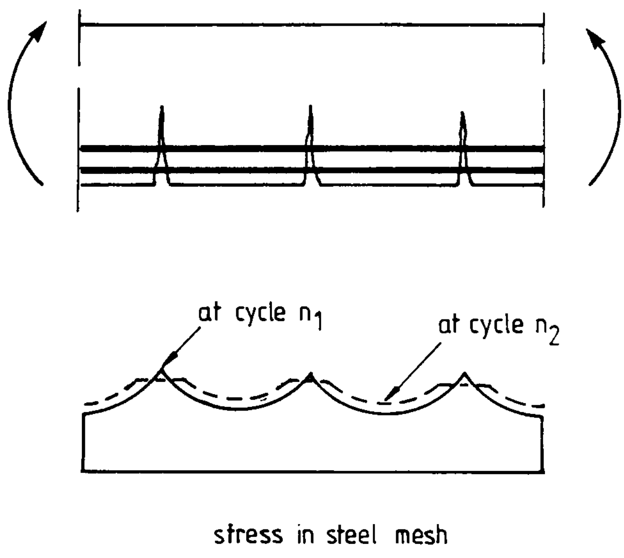

Fig. 2. Steel stress distribution at different load cycles.

THE NEW METHOD FOR PREDICTING STEEL STRESS IN FERROCEMENT

The stress in the outermost layer of steel mesh and in the extreme "fiber" of mortar have been calculated using elastic cracked section analysis by a number of researchers [1-6]. According to the presentation in the last section by the authors, it can be inferred that the steel stress values calculated from elastic cracked section analysis tend to approach the theoretical upper boundary. This is thought to be one of the main reasons why the fatigue life of reinforcement in composite appears to be higher than in the air.

In the following, by using the concept of rectangular stress distribution [8, 9] the authors consider a method for predicting the steel stress values at theoretical lower boundary. As shown in Fig. 3, the depth of compressive mortar, X, is calculated by the following equation first.

$$M = \sigma_c * X * b * \left(h - \frac{t_s}{2} - \frac{X}{2} - c\right) \qquad \text{........ (1)}$$

Where: M is the moment applied to the section,
σ_c is the mortar stress which is assumed to be equal to 0.55 x 0.67 x cube strength under static test according to ACI's suggestion [11],
b, h and c are width, depth and cover of the cross section respectively, and
t_s is the height of reinforced zone.

After the X value is determined, the steel stress σ_s can be easily calculated by the balance condition.

$$\sigma_s * A_s = \sigma_c * b * X \qquad \text{....... (2)}$$

Where: As is the area of steel.
The stress values calculated by this method tend to approach theoretical lower boundary.

For four groups of specimens (108 specimens) chosen deliberately [1- 3] to represent 2 and 4 layers of 12.6 mm x 12.6 mm x 1.6 mm diameter of galvanized and ungalvanized weldmesh, the maximum steel stress levels calculated by elastic cracked section analysis and rectangular stress distribution assumption are presented in Table 1. The stress levels calculated by rectangular stress distribution assumption are about 5 % to 17 % lower than those calculated by elastic-cracked section analysis . The minimum stress level calculated by elastic - cracked - section analysis is 12.5 % for all specimens. It is 0.8 % to 3.2 % higher than those calculated by rectangular stress distribution assumption. Because the differences in calculated minimum steel

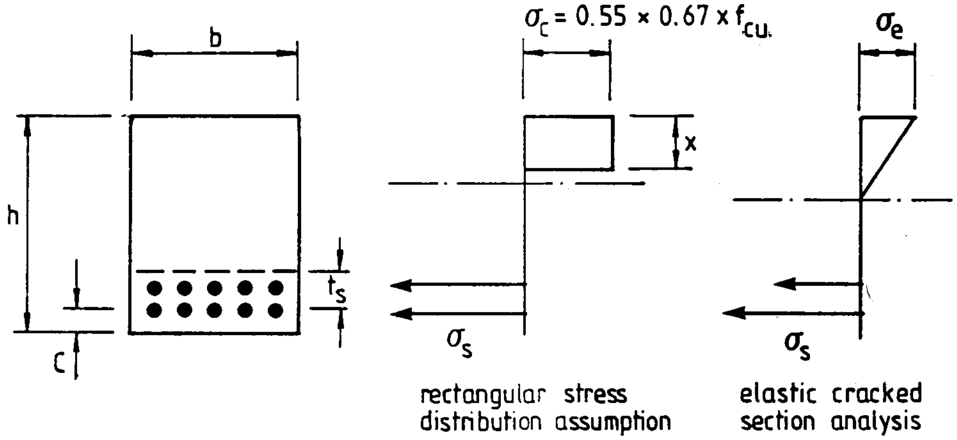

Fig. 3. Models for predicting steel stress.

stress levels between the two methods are small the minimum stress levels are assumed to be 12.5 % for convenience in the following analysis. This assumption does not influence the conclusion drawn on this presentation. The specimen size was 350 mm x 125 mm x 30 mm thick with a cover of 5 mm. The test rig consisted of a four point arrangement with a span of 300 mm

Table 1 Stress Levels (%) of Reinforcements in Ferrocement Calculated by Different Methods (presented as a percentage of ultimate strength)

Elastic Cracked Section	Rectangular Stress Distribution			
	2UG1.6*	4UG1.6	2G1.6	4G1.6
45		(32.4)	(37.5)	(31.6)
55	49.0	41.0	46.7	(39.4)
65	59.2	51.9	55.6	47.8
75	68.5	65.5	65.5	57.7
85	80.2		76.5	69.9

* First numeral denotes number of layers, Galvanized (G) or Ungalvanized (UG), Second numeral denotes wire diameter in mm.

Stress levels in "()" are not used for regression equations.

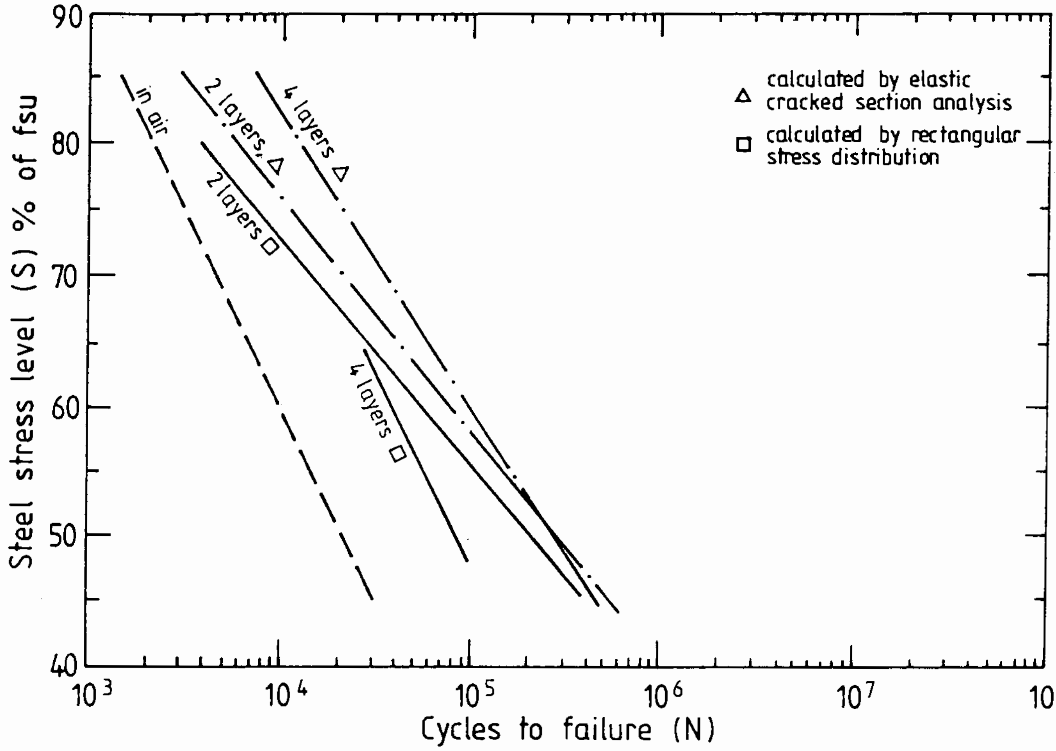

Fig. 4. *P-S-N* relationships, 1.6 mm ungalvanised weldmesh in air and in ferrocement [2 , 4]. $P = 5$ %.

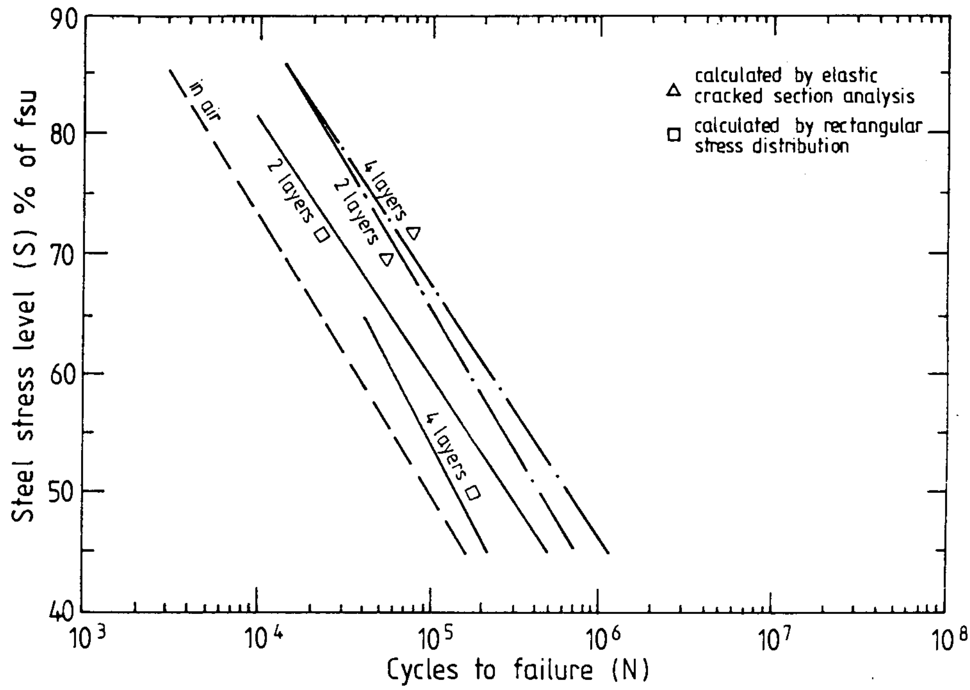

Fig. 5. *P-S-N* relationships, 1.6 mm ungalvanised weldmesh in air and in ferrocement [2, 4]. $P = 50$ %.

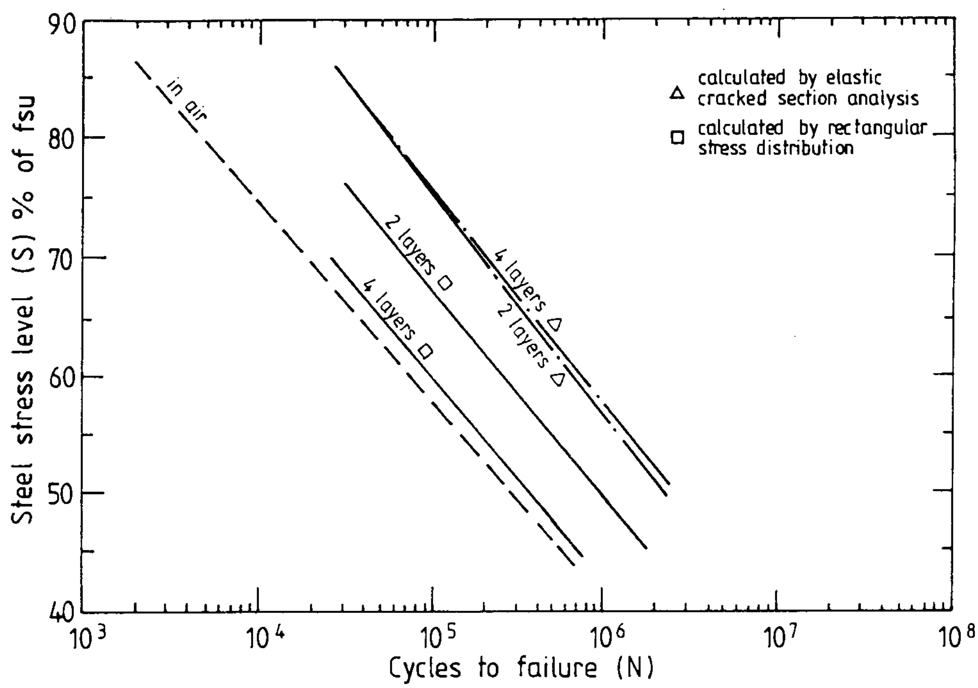

Fig. 6. *P-S-N* relationships, 1.6 mm galvanised weldmesh in air and in ferrocement [2, 4]. $P = 5$ %.

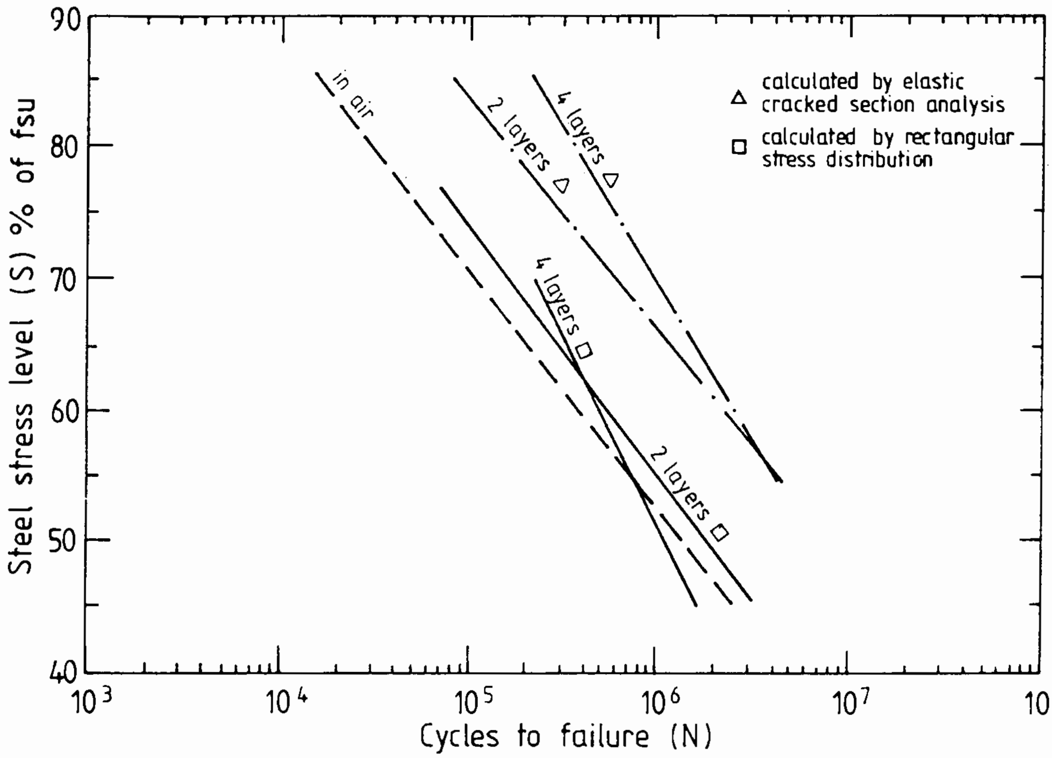

Fig. 7. *P-S-N* relationships, 1.6 mm galvanised weldmesh in air and in reinforcement [2, 4]. $P = 50\ \%$.

and a constant bending moment zone of 120 mm. The regression equation of P-S-N relationships of reinforcement in air and in composite are shown in Table 2. The corresponding plots can be observed in Figs. 4-7. Some calculated examples based on P-S-N relationships in Table 2 are presented in Table 3. It can be seen that even if the steel stress in ferrocement is underestimated (to be equal to the values of theoretical lower boundary) the fatigue life of reinforcement in composites is still higher than that in the air in most cases. This is because only a limited number wire sections, located at the cracks in mortar (Fig. 2), are subjected to the applied maximum stress [2].

A design example (Fig.3) in which the fatigue behavior of ferrocement is assumed to be the same as that of wire tested in air is presented in the following.

Design Example

Take the *P-Sa-Na* relationship of wire tested in air to be $S_a = 162.788 - 18.477 \, \mathrm{Log} Na$ (P=50%). Let the depth (h), width (b) and cover (c) of the member be 30 mm, 125 mm and 5 mm respectively, with 2 layers of 12.6 mm x 12.6 mm x 1.6 mm diameter galvanized weldmesh. Assume cube strength (f_{cu}) and the elastic modulus of mortar to be 50 N/mm^2 and 30 000 N/mm^2 respectively; ultimate steel stress (f_{su}) and elastic modulus of wire (E_s) to be 458 N/mm^2 and 16 8000 N/mm^2 respectively. Fatigue life of ferrocement is required to be $N_f > 1\,000\,000$. Determine the moment capacity of the ferrocement.

(1) Determine allowable maximum stress level.

By bringing $N_f = N_a = 1\,000\,000$ in the P - S_a - N_a equation given above, the steel stress level, $S\ (=S_a)$ is < 52.1%.

Table 2 Regression Equations of P-S-N Relationships

		Probability of Failure P			Average Fatigue Strength at10^6
	Series	5%	50%	(%)	(MN/m^2)
In air	UG1.6	Sa =180.191-29.971LogNa	Sa=165.825-23.124LogNa	27.1	156
	G1.6	Sa =141.930-16.754LogNa	Sa=162.788-18.477LogNa	52.1	239
In ferro-cement	2UG1.6	Se=147.423-17.818LogNe	Se=178.322-22.589LogNe	42.8246	
		Sr=141.239-16.988LogNr	Sr=162.591-20.258LogNr	41.0	236
	4UG1.6	Se=171.302-22.271LogNe	Se=171.486-20.860LogNe	46.3266	
		Sr=195.827-29.557LogNr	Sr=187.586-26.586LogNr	28.1	161
	2G1.6	Se=165.401-18.0-5LogNe	Se=173.883-18.098LogNe	65.3299	
		Sr=154.679-17.540LogNr	Sr=170.944-19.488LogNr	54.3	248
	4G1.6	Se=166.274-18.253LogNe	Se=216.135-24.572LogNe	68.8315	
		Sr=147.355-17.503LogNr	Sr=227.790-29.539LogNr	50.6	232

Note: Sa, Se and Sr are steel stresses as a percentage of ultimate static tensile strength. Among them, Se and Sr are calculated by elastic cracked section analysis and rectangle stress distribution respectively.

Table 3 Calculated Examples by using P-S-N Equations in Table 2

		P = 5 %			P = 50 %		
	S	Na	Ne	Nr	Na	Ne	Nr
2UG1.6	50	22075	293544	234850	102062	479435	361288
	55	15034	153836	119252	62035	287994	204662
	60	10239	80620	60554	37706	172996	115937
	65	6973	48078	30748	22919	103918	65676
4UG1.6	50	22075	279663	85853	102062	666612	149668
	55	15034	166774	58155	62035	383867	97065
	60	10239	99454	39394	37706	221048	62949
	65	6973	59308	26685	22919	127290	40825
2G1.6	55	306936	2465986	929001	1271270	7000370	1607221
	55	154387	1303296	481898	681759	3705505	890241
	60	77656	688806	249972	365615	1961434	493105
	65	39061	364041	129667	196073	1038246	273131
4G1.6	50	306936	2344936	364913	1271270	5769669	1044293
	55	154387	1247968	189027	681759	3611331	707220
	60	77656	664165	97917	365615	2260391	478947
	65	39061	353466	50722	196073	1414816	324354

(2) Determine moment capacity of ferrocement.
Bringing the allowable steel stress, $\sigma_s = f_{su} * 52.1\ \%$, and other parameters into the two moment capacity predicting models:

$M_e < 0.175$ kNm (by elastic cracked section analysis)

$M_r < 0.198$ kNm (by rectangular stress assumption)

Based on above, if the fatigue behavior of ferrocement is assumed to be the same as that of wire tested in air then for a simple and economical design (as well as a conservative design of higher reliability because the fatigue strength of the wire in the composite is higher than that of the wire in the air), the rectangular stress distribution method is recommended by the authors.

OBSERVATIONS ON THE PREDICTION OF CRACK WIDTH

The crack widths increase with cycles of load, because of the decrease of stiffness of mortar due partly to the growth of small internal cracks and partly to creep occurring outside of cracked zone, and the progressive deterioration of bond between the steel and the matrix [4-8].

It has been found that crack widths increase with cover thickness under static load [12, 13]. It can be inferred that a similar phenomenon would be observed under cyclic load. This can be

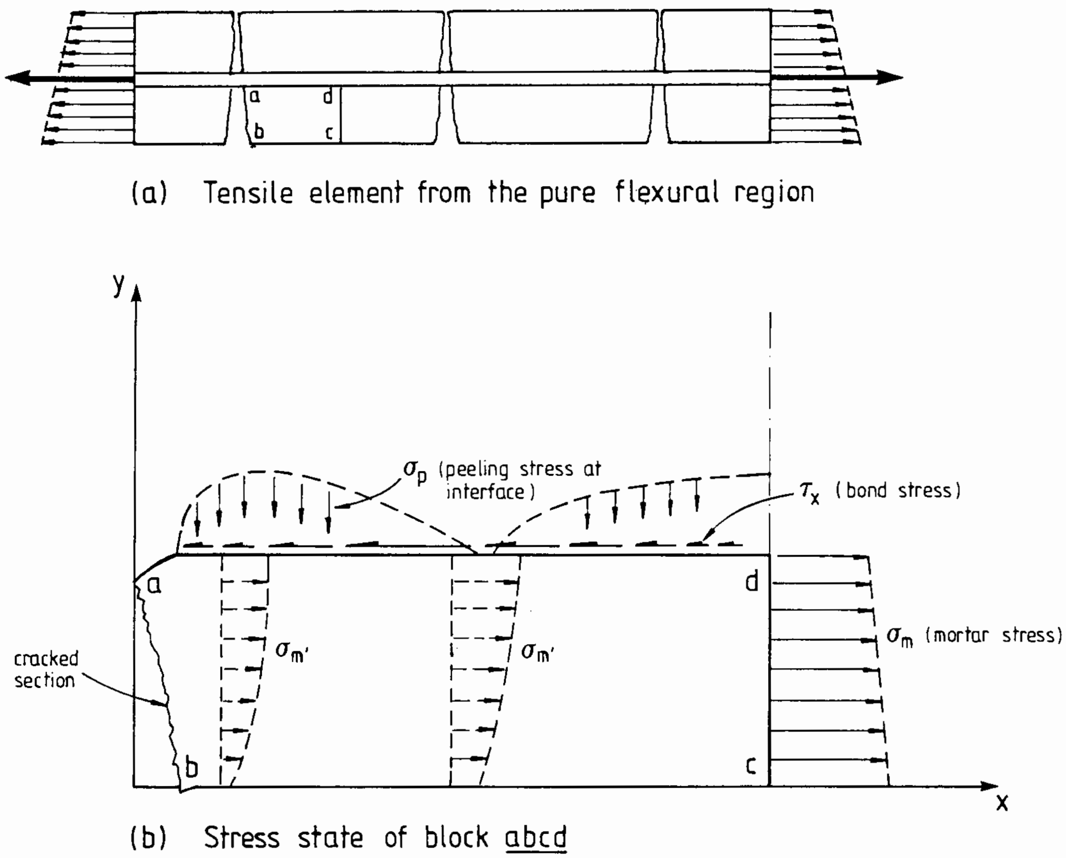

Fig. 8. Qualitative depiction of stress in a tensile region from a pure flexural zone.

proved by analyzing the stress state of block **abcd** (Fig. 8 B). The block is taken from an element in the tensile region of pure flexural zone shown in Fig. 8 A. The direct stress σ_m and bond stress τ_x on the block can only satisfy one balance condition, $\sum X = 0$. In order to keep the balance, the block needs another condition $\sum M = 0$. It leads us to conclude that there must be a peeling stress at boundary **ab**. Obviously, the thicker the cover is, the larger will be the peeling stress. The larger peeling stress results in longer debonding zone, consequently wider cracks. Because of the existence of peeling stress and uneven distribution of direct stress σ_m along Y axis, the crack pattern can be depicted (exaggerated) in Fig. 8 B.

Longer crack spacing results in wider crack width. It has been reported that crack spacings increase with the increase of the cover [1, 4]. When the cover is relatively thinner the crack spacing is nearly equal to the transverse wire spacing [4]. When the maximum value of the cover is 5 mm, as specified by ACI [14], then the crack spacing is about 13.8 mm to 23.6 mm (with mesh wire diameter of 1.6 mm and spacing of 12.6 mm) according to Fakhri [1].

In view of the large amount of effort put into the development of methods of predicting crack widths [4- 6] the authors have tried to establish if this effort is justified. They have come to the conclusion that the dominant criterion of design is that of steel stress, and not the crack width for all common structures. In the following the authors present an analytical proof of their view in which a method has been deliberately chosen which overestimates the crack width. The line of reasoning is that if the crack widths so calculated can still be shown to be dominated by the stress criterion then their view shall be verified. It will be assumed that under cyclic loading the two phases will be totally debonded and that the mortar will experience total stress relief and strain recovery. Therefore, the elongation of the reinforcement between two adjacent cracks will be equal to the crack width. According to Shah et al, the creep of weldmesh under cyclic loading is less than 3 % at a maximum stress of 57.6 % of ultimate strength. In light of the ACI formula for static flexure [14], the crack width can be modified to:

$$W = L * (f_{max} / Es) * 103\% * B \qquad \text{........ (3)}$$

Where: L is the crack spacing (instead of wire spacing, s, which is less than L).
f_{max} and E_s are the maximum cyclic stress and elastic modulus of steel respectively.
B is the ratio of distances to the neutral axis from the extreme tensile "fiber" and from the outermost layer of steel [15].

Consider the most unfavorable conditions:

Height of section = 20 mm
Cover thickness = 5 mm
Crack spacing = 23.6 mm (The longest one in Fakhri's test result (1))
Neutral axis is in the middle of the section.
Stress and elastic modulus of steel are equal to 270 N/mm^2 (allowable static stress) and 200 000 N/mm^2 respectively [14].

Oversttimate of crack width, W, obtained from Eq. 3 is 0.0656 mm.

Fakhri [1] measured the crack widths at the bottom on 4 groups of specimens(108 specimens) as stated before. Those specimens with minimum of two million cycles of fatigue life showed that the ratios of crack widths at two million cycles to those at 350 cycles were about 1.25 to 1.65. All final observed average crack widths are presented in Table 4. It should be noted that the elastic moduli of all kinds of meshes for these test are much lower than 200 kN/mm^2. As can be seen, both the over-estimate of crack width and the test results are smaller than the allowable value of 0.1mm for common structures as specified by ACI (14).

CONCLUSIONS

1. Peak steel stress at the cracked section decreases with the increase of load cycles.
2. When *S-N* relationship of ferrocement is assumed to be the same as that of wire in air then the rectangular stress distribution assumption appears to be better for a reliable and economical design.
3. The dominating design criterion under cyclic loading is not the crack width but the steel stress for all common structures.

Table 4 Cyclic Crack Data at Load Cycle of Two Million

	2UG1.6*	4UG1.6	2G1.6	4G1.6
Elastic modulus of steel (kN/mm^2)	168	168	168	168
Ultimate steel strength (N/mm^2)	527	527	458	458
Steel stress level Se* (%)	---	---	55	65
Steel stress level Sr (%)	---	---	46.7	47.8
Average crack width (mm)	---	---	0.046	0.059

* Se and Sr - Stress level calculated by elastic cracked section analysis and rectangular stress distribution assumption respectively.

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Flexural Impact Damage of Ferrocement

Y. Kobayashi *, Y. Tanaka * and M. Ono*

Lateral flexural impact tests of ferrocement were performed under three point bending. To understand the properties of impact, test results are discussed on the effect of striking velocity on impact load, strain, and deflection. Moreover, the relationships between face strain and deflection, and the absorbed energies obtained from load-deflection curves were studied.

The strain at first crack in impact tests was approximately equal to that in static tests. Localized damage occurred under the load right after impact. A linear relationship was observed between compressive strain and deflection after localized damage. The energy expended in impact damage could be assigned to localized damage, crack opening, compressive failure of mortar, and bending of reinforcement. The energy for localized damage was in proportion to the drop height of a striker and was 25 % of the input energy. For a specimen in which only cracks occurred, 85% of the input energy were spent for a cracked damage, and 15% remained as the recoverable energy of deflection.

INTRODUCTION

Ferrocement is superior in flexibility, but has a weakness in impact resistance because it is a thin plate. However, it is experimentally known that repairs are very easy because impact damage is localized to a limited area.

According to previous reports [1-3], the impact of ferrocement had been studied under single or cyclic drop impact tests and Charpy impact tests. Impact resistance is usually expressed by potential energy which is obtained by multiplying drop height by weight of a striker. This resistance increases with an increase of a volume fraction or a specific surface of reinforcement. This is mainly connected with improvements of cracking resistance or failure strength. Leak tests were also carried out after impact tests for the evaluation of the impact resistance. Shah et al. [4] confirmed that impact damage became lower by using a high specific surface and a high strength of mesh. However, Nimityongskul et al. [1] indicated that the impact resistance did not always increase with the increase in the specific surface of mesh under an almost practical maximum steel content condition; ferrocement has the optimum specific surface for a leak resistance after impact.

The evaluation of impact damage is very difficult. However, it is necessary to solve the impact problem of ferrocement to create standards for inspection of impact damage and thus it is important to understand the basic impact properties of ferrocement. In the present paper, the impact load was applied by drop weight under three point bending. The experimental strain rate was very slow because the striking velocity was less than 5 m/s. Test results are discussed on the response of impact waves, on the effect of the striking velocity, and on absorbed energies. Moreover, the failure process is studied in the compressive strain and deflection relationships.

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EXPERIMENTAL PROCEDURE

Materials and Specimen

A normal Portland cement and a river sand of 2 mm maximum sizes were used. The mix proportion of the mortar was water-cement ratio of 0.41, and sand-cement ratio of 2. In the strengths of the mortar, the compressive strength was 61MPa, the tensile strength was 5.2MPa, and the bending strength was 7 MPa. The elastic modulus of the mortar was 29.2GPa.

Square woven wire mesh (JIS G 3555, 1.0 mm x 10 mm x 10 mm) and steel bars (JIS G 3112, SR24, 5.6 mm in diameter) were used as reinforcement. The mechanical properties are given in Table 1. σ_{su}, $\sigma_{0.2}$, and E_s represent the ultimate tensile stress, the proof stress that corresponds to 0.2 % permanent set, and the elastic modulus of the reinforcement, respectively.

Table 1 Mechanical Properties of the Reinforcement

	σ_{SU} MPa	$\sigma_{0.2}$ MPa	E_S GPa	Remarks
Woven Wire Mesh	415	325	191	1 ϕ x 10 x 10
Reinforcing Bar	444	414	205	5.6 ϕ

A ferrocement specimen of 600 mm long, 300 mm wide, and 25 mm thick is shown in Fig.1. The cross section was reinforced with one layer of the reinforcing bars and three layers of the square woven wire mesh at each side of the bars. The bars were placed in the center at intervals of 50 mm. After placing the mortar, the cover for the tensile face was 3 mm thick, but that of a compressive face varied from 3 mm to 6 mm in thickness. The volume fraction of reinforcement was 0.038 to 0.04 in the longitudinal direction against flexural moment.

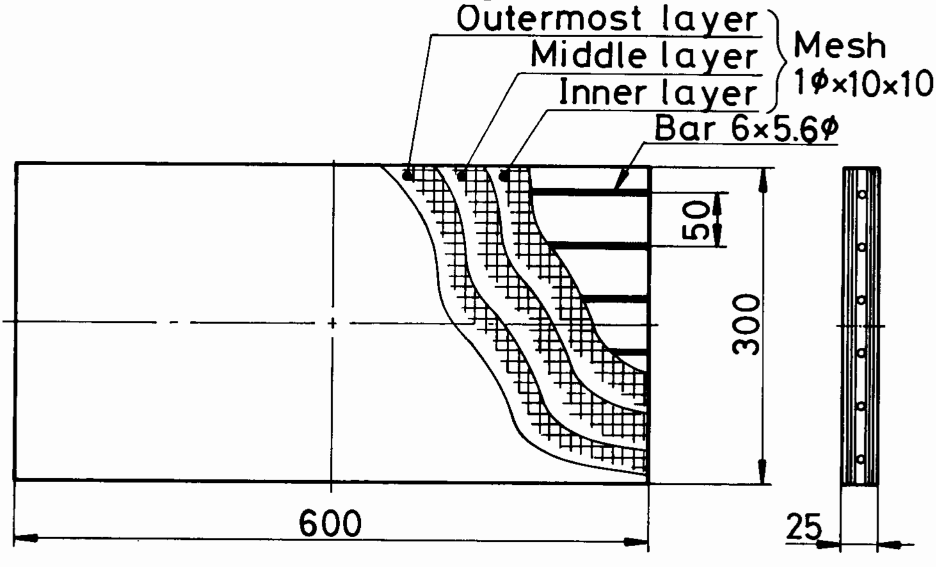

Fig. 1. Ferrocement specimen.

Testing Procedure

A schematic diagram of the flexural impact testing apparatus is shown in Fig.2. Single impact tests were performed under three point bending. The span was 500 mm. The striker was made of a steel rod; 100 mm diameter, 1000 mm long, and 61 kg in weight. A striking velocity was varied

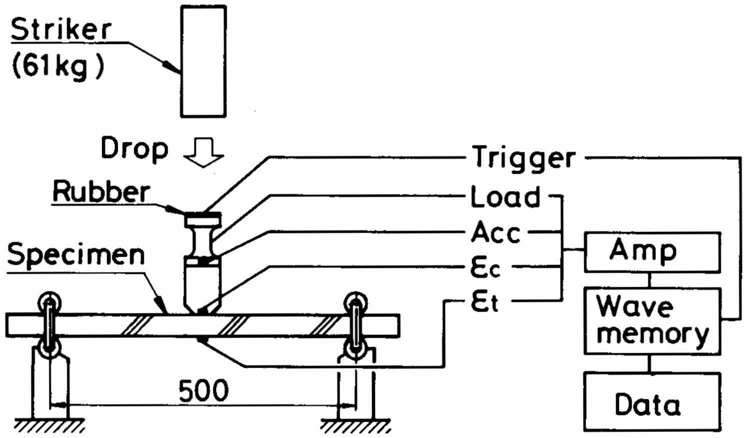

Fig. 2. Schematic diagram of flexural impact testing apparatus

by drop height of the striker. An impact load was transmitted to the specimen through a rubber sheet of 10 mm in thickness, a load cell, and a bending jig. The bending jig was fixed on the specimen by clamps. The specimen was also fixed on both supports to prevent it from jumping up, but was free to slide and to rotate due to flexure.

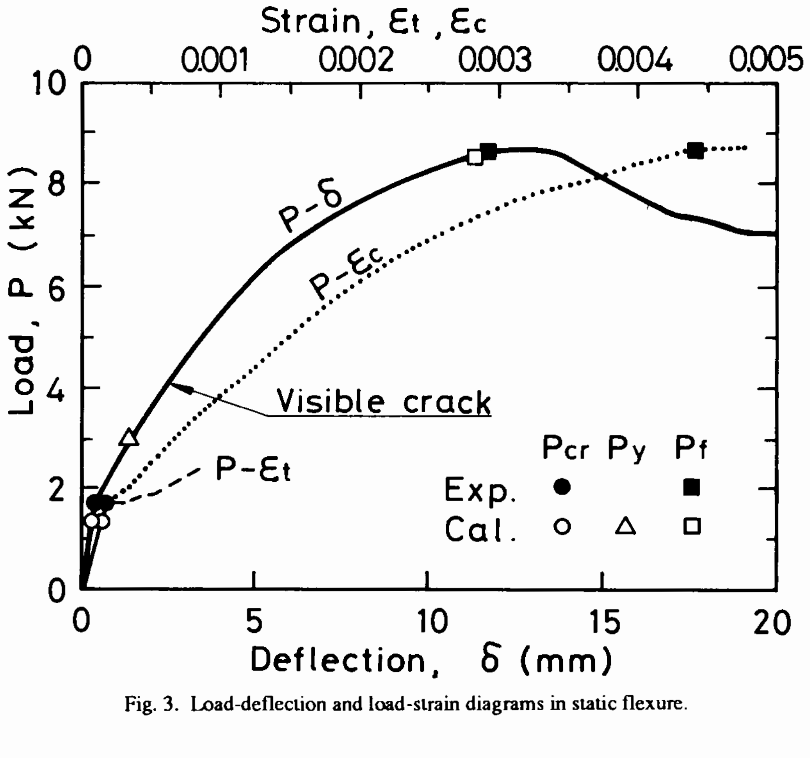

Fig. 3. Load-deflection and load-strain diagrams in static flexure.

The impact load was detected by a load cell (capacity: 20 kN) of strain gage type. The acceleration of the specimen was measured by a transducer(capacity: 500*g*) on the jig. A tensile and a compressive strain were measured by strain gages of 30 mm length. These analog data were transformed into the digital data with a sampling rate of 10 micro second by using an A/D translator of 12 bit resolving power. Finally the data were stored on a floppy disk in a personal computer.

RESULTS AND DISCUSSIONS

Flexural Strength in Test

A load-deflection curve and a load-compressive strain curve in flexural static test are shown in Fig.3. The first crack occurred at 180 $\times 10^{-6}$ on the tensile face. The experimental load at the first crack was higher than the estimated load. The visible crack load was beyond the theoretical yield load. The final failure occurred at a compressive strain of 4420 $\times 10^{-6}$. The estimated maximum load agreed approximately with the experimental load. The above failure stages were estimated by using the equations in Ref.[5]. The estimated equations were deduced from the assumption that the strain distribution was linear in the section of a specimen.

As the neutral axis at the first crack was located almost in the middle point of the section, the specimen itself behaved elastically up to the first crack. In proportion as a load approached the yield load, the neutral axis moved into the compressive zone, and therefore this made the flexural rigidity decrease markedly. However a linear relationship was observed in the range between the first crack and the yield load as seen in Fig.3.

Failure Stages in Static Flexure

The above failure stages are shown by a schematic diagram in Fig.4. The first crack is defined as the state when the tensile face stress reaches the tensile strength of the mortar. The mesh yield means the state when the tensile stress of the outermost mesh reaches the proof stress. The final failure occurs by breaking failure of the mesh or the compressive failure of the face mortar, and generally indicates the maximum load.

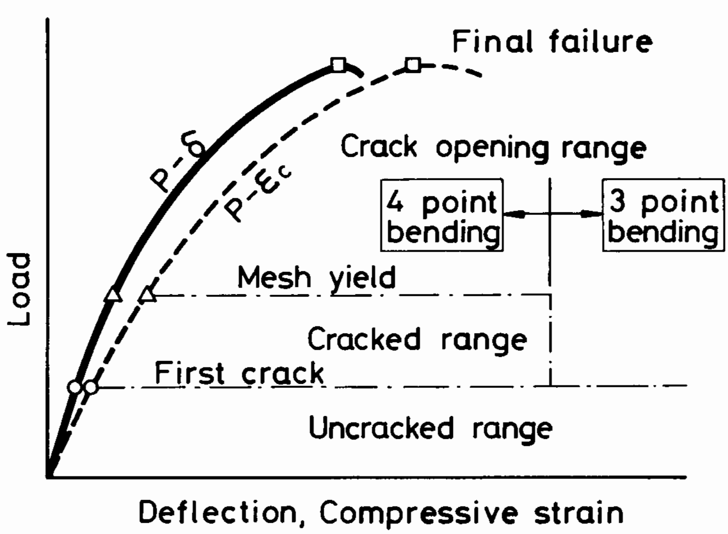

Fig. 4. Schematic diagram of failure stages in static flexure.

Under four point bending, cracks mainly begin to open after the mesh yield because fine cracks occur one after another in a region of uniform moment of a specimen. However, under a three point bending test, cracks begin to open immediately after the first crack. In case of a collision between ferrocement and another object, it is assumed that the loading condition is similar to three point bending.

It is necessary that ferrocement is watertight for utilization in a marine environment. So, crack observations can offer helpful information for potential damage. Visible cracks are approximately 0.02 mm to 0.025 mm wide in flexure[5], and generally are observed after yield of the outer most mesh. Therefore for convenience the load corresponding to visible cracking is considered to be the same as the yield load. In ferrocement under tension[6,7], water began to leak at a crack width of 0.025 mm. Visible cracks do not always leak in flexure if the crack does not penetrate through the section of the specimen. However, a visible crack provides useful data in the damage evaluation of impact.

Time Variation of Measured Impact Waves

An impact load, P, was corrected by subtracting an inertia force from the measured value detected by the load cell. The inertia force was calculated from multiplying the measured acceleration by the total mass of the jig and the load cell on the specimen. A flexural velocity, V, was obtained from the integral of the acceleration with respect to time. The deflection, δ, was also obtained from the integral of V. ε_t and ε_c represent the tensile and the compressive strain on the specimen respectively. The response of these strains can indicate the first crack and the final failure in the experiment. The striking velocity, V_s, is given by Eq.(1) on the assumption that all the potential energy is completely transformed into kinetic energy.

$$V_s = (2\,g\,h)^{1/2} \qquad \text{........ (1)}$$

where g is the gravity of acceleration and h is the drop height of the striker.

Fig.5 shows the time variation of measured waves when V_s=4.8 m/s. The peak load, P_p, was observed immediately after the impact. From the strain response on the specimen, it was observed

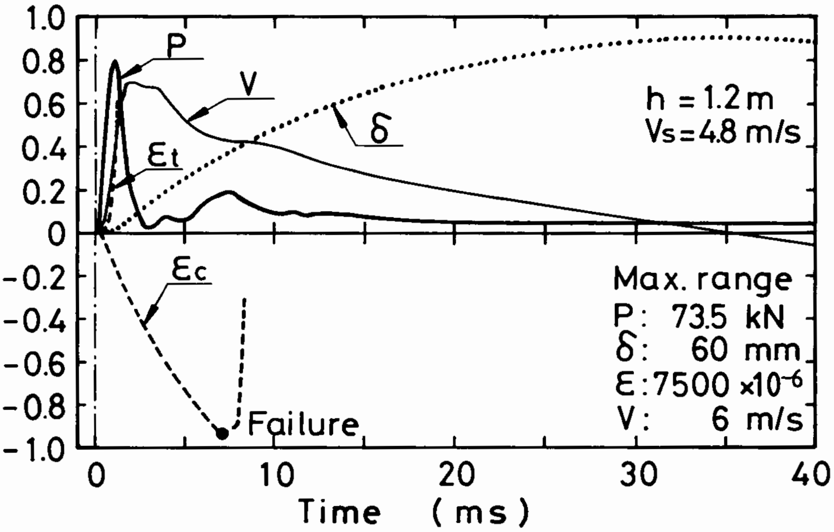

Fig. 5. Variation with time of impact waves.

that the first crack occurred right after impact and the compressive failure of the mortar occurred after the peak load. V reached approximately V_s right after impact, but decreased gradually with time. The response of the deflection was delayed in comparison with the response of the impact load. The deflection continued to increase after the compressive failure of the mortar and became a maximum at V =0. These variations of deflection are caused by not only the bending of reinforcement but also by the opening of cracks directly under the impact loading axis.

The impact damage was distinguished in two types by observation of the damaged specimen. One was damage where only tensile cracks occurred on the tensile face mortar and the other was damage where compressive failure occurred on the face mortar as the final failure in Fig.4. The first one is designated as a cracked-damage specimen and the other as a broken-damage specimen. Fig .5 shows an example of a broken-damage specimen.

Effect of Striking Velocity on Strain, Deflection, and Load

The measured values at the first crack, the peak load, and the final failure are discussed. The responses at the first crack are shown in Fig. 6. ε_{tcr}, δ_{cr} , and P_{cr} represent the tensile strain, the deflection, and the load at first crack respectively. Now it is necessary to estimate the time when the first crack occurred in impact. The time was estimated from the response of the tensile and compressive strain in the time variation curve or the load-strain curve. When a tensile crack occurred, the tensile strain increased markedly in comparison with the increase of the compressive strain because the neutral axis was shifted by crack initiation. Generally, the tensile strain is approximately equal to the compressive strain in the uncracked range. Therefore, ε_{tcr} was regarded as the strain at first crack in impact.

ε_{tcr} was independent of V_s and was approximately equal to the static data plotted on the vertical axis at V_s=0. P_{cr} increased as the striking velocity increased, but δ_{cr} decreased linearly. The above

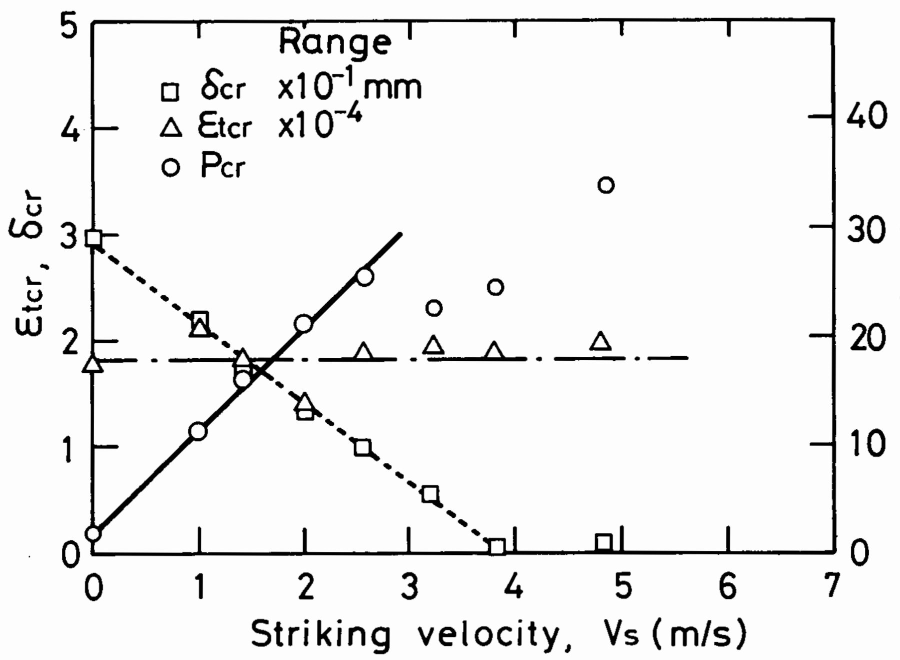

Fig. 6. Effect of striking velocity on first crack, load and deflection at first crack.

results indicated that the impact damage became more local with an increase of striking velocity. Moreover, the behavior of load and deflection are related with a difference of impact damage; that is, the cracked-damage specimen for V_s <3 m/s, and the broken-damage specimen for V_s >3 m/s.

Fig.7 shows the response at the peak load P_p, δ_p and ε_{cp} represent the deflection and the compressive strain at the peak load respectively. P_p was proportional to V_s and all the data lay on

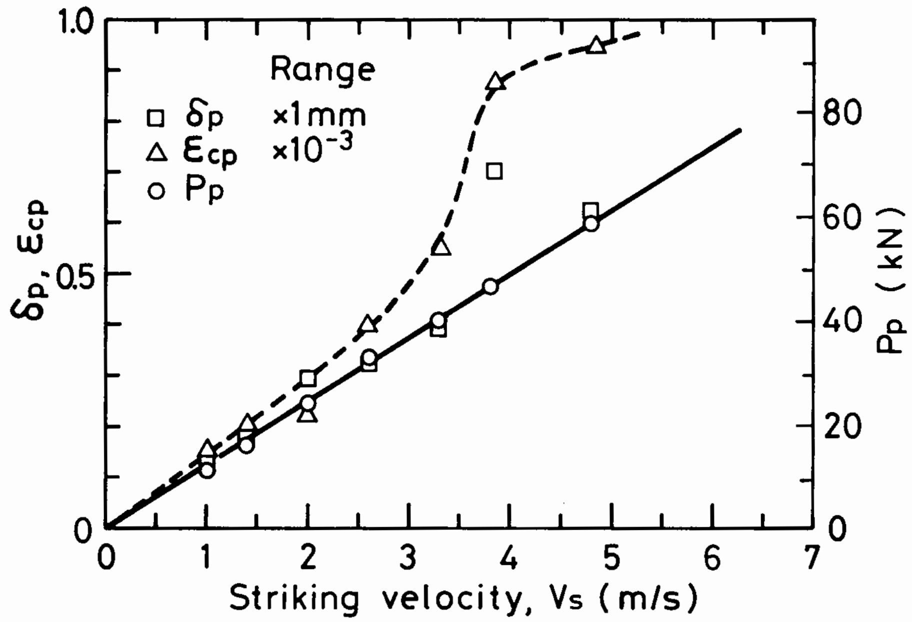

Fig. 7. Effect of striking velocity on peak load, compressive strain and deflection at peak load.

a straight line. Linear relationships were observed in δ_p and ε_{cp} for V_s<3 m/s. These are also related to the type of impact damage as mentioned above.

Fig.8 shows the response at final failure. ε_{cf} represents the compressive strain; that is, the maximum strain in cracked-damage specimens or the compressive failure strain in broken-damage specimens. P_f and δ_f represent the load and deflection at ε_{cf} respectively. Generally, δ_f became a maximum when ε_{cf} reached a maximum in the cracked-damage specimens. The value of ε_{cf} was greater than the static failure strain in the broken-damage specimens as seen by the solid triangles in Fig.8. The increase of failure strain is caused by the effect of strain rate on the strength of mortar. In connection with the effect of strain rate, impact compressive tests of another mortar were performed under a different striking velocity. As a result, the strain rate did not affect the elastic modulus, but the compressive strength increased considerably.

Relationships between Compressive Strain and Deflection

Fig.9 shows the relationships between compressive strain and deflection in experiments. These are useful to understand impact damage. For the static test specimen, the strain-deflection graph is combined by two lines with different slopes. In the impact specimens, the compressive strain increased rapidly when the deflection was small, but the strain-deflection relations were linear overall. Moreover, their slopes seem to be approximately the same irrespective of different values of striking velocity.

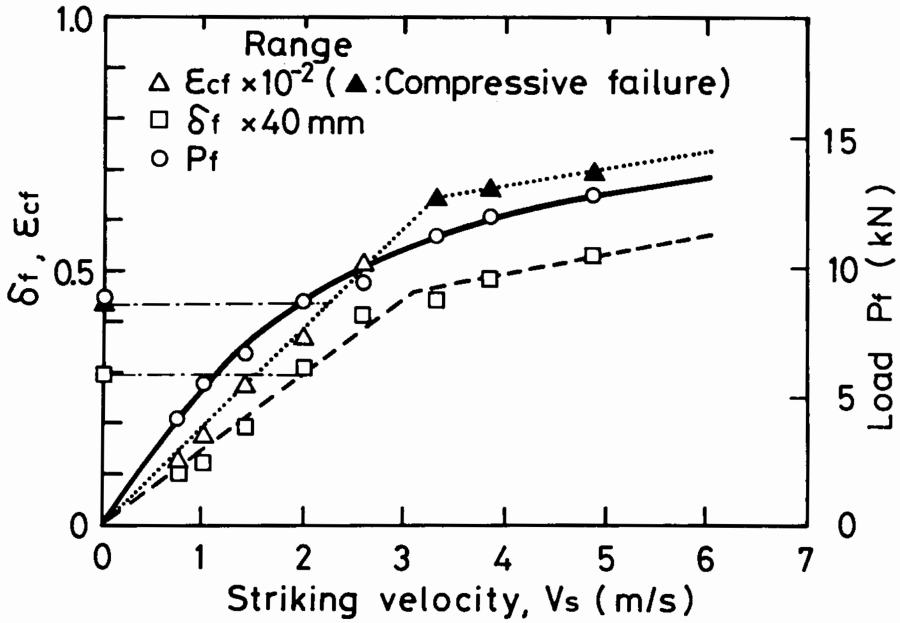

Fig. 8. Effect of striking velocity on compressive strain, load and deflection at compressive failure of face mortar.

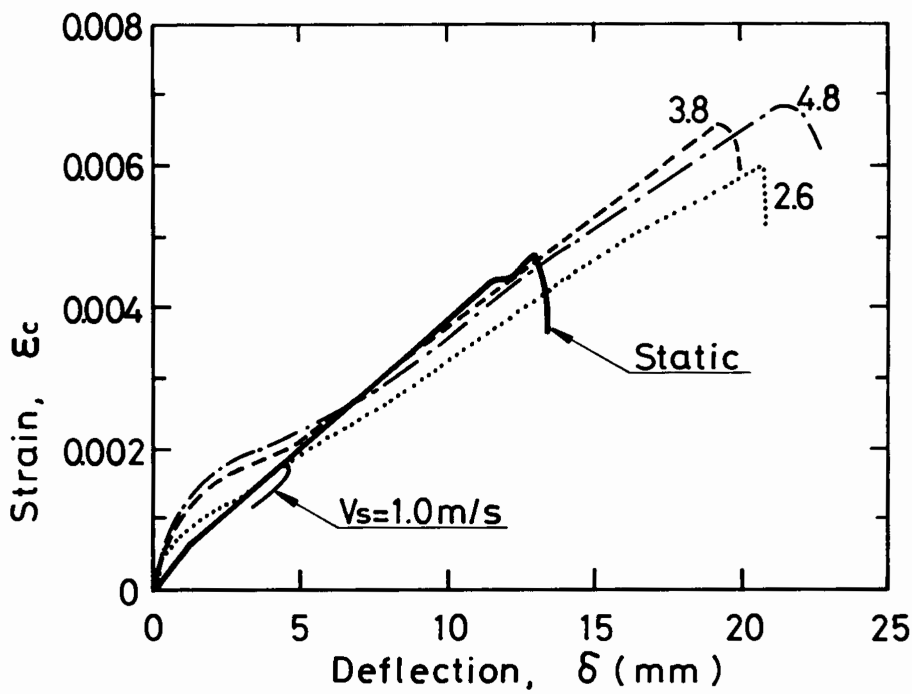

Fig.9. Relationships between compressive strain and deflection.

The above experimental relationships can be understood by using the schematic diagram of Fig. 10. In the case of a static specimen, the compressive strain reaches the point F by way of the points B and Y as shown by the solid line. This can be divided into two lines AC and CF intersecting at the point C. The points B, Y, and F correspond to the first crack, the mesh yield, and the compressive failure of the mortar respectively. ε_c is expressed by Eqs.(2) and (4) from the experimental fact that the points B and Y lay on the line AC and CF respectively. The slopes, α_1 and α_2, are expressed by Eqs.(3) and (5) respectively. Moreover, these slopes correspond to the behavior of the uncracked range and the crack opening range in the P - δ curve of Fig.4. Assuming that the load-deflection curve is bent at the first crack as seen in Fig.3, the value for estimated yield point, ε_{cy} and δ_y, are given by Eqs.(6) and (7) respectively. The constant, ε_0, becomes Eq.(8).

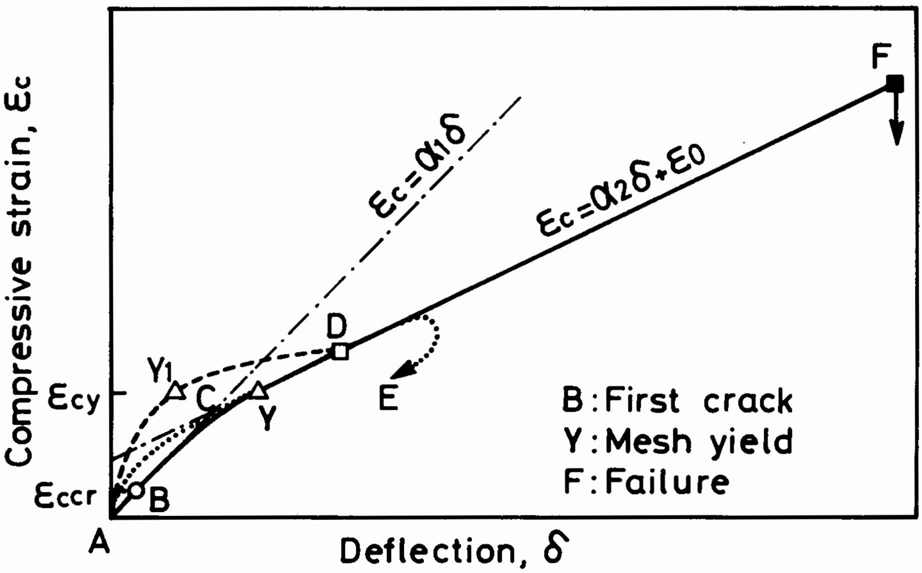

Fig. 10. Schematic diagrams of the relationships between compressive strain and deflection.

$$\varepsilon_c = \alpha_1 \delta \quad \text{........ (2)}$$

$$\alpha_1 = \frac{12}{l^2} e_{cr} \quad \text{........ (3)}$$

$$\varepsilon_c = \alpha_2 \delta + \varepsilon_0 \quad \text{........ (4)}$$

$$\alpha_2 = \frac{12}{l^2} e_y \quad \text{........ (5)}$$

$$\varepsilon_{cy} = \frac{e_y}{d_1 - e_y} \varepsilon_{sy} \quad \text{........ (6)}$$

$$\delta_y = \frac{l^2}{12} \left\{ \frac{\varepsilon_{cr}}{d - e_{cr}} + \left(1 - \frac{p_{cr}}{p_y}\right) \frac{\varepsilon_{cy}}{d_1 - e_y} \right\} \quad \text{........ (7)}$$

$$\varepsilon_0 = \frac{e_y \varepsilon_{cr}}{d - e_{cr}} + \left(1 - \frac{p_{cr}}{p_y}\right) \frac{e_y \varepsilon_{sy}}{d_r e_y} \qquad \text{........ (8)}$$

Where l is the span, e_{cr} and e_y are distances from the compressive face to a neutral axis at first crack and at mesh yield respectively, ε_{sy} is the yield strain of the mesh, d is the total thickness of specimen, d_l is the thickness excluding the tensile cover, P_{cr} and P_y are the estimated loads for the first crack and for the mesh yield respectively.

α_{1cal} and α_{2cal} represent the estimated values, and α_{1exp} and α_{2exp} represent the experimental values obtained from Fig.9. R_1 and R_2 denote the ratios of the experimental slope to the estimated slope. These data are given in Table 2. The static specimens, FCB9 and FCB10, have different thicknesses of cover, but the experimental slopes nearly agree with the estimated ones. The straight line YF indicates that cracks open with an increase of deflection.

Table 2 Slopes of the Strain-Deflection Relationships

Speci-men	Vs m/s	α_{1exp} x 10^{-6} mm^{-1}	α_{1cal} x 10^{-6} mm^{-1}	R_1	α_{2exp} x 10^{-6} mm^{-1}	α_{2cal} x 10^{-6} mm^{-1}	R_2	ε_{cy} x 10^{-6}	δ_y	ε_o mm	Remarks
FCB10	0	609	605	1.007	362	353	1.025	853	1.667	262	*
FCB9	0	598	586	1.020	290	300	0.967	843	1.551	377	*
FCB1	0.99	–	605	–	341	357	0.955	870	1.722	256	**
FCB2	1.40	–	600	–	345	349	0.989	868	1.720	268	**
FCB3	1.98	–	597	–	297	356	0.834	880	1.776	248	**
FCB4	2.62	–	578	–	287	327	0.878	886	1.802	296	**
FCB5	3.28	–	607	–	344	344	1.000	871	1.701	286	***
FCB6	3.83	–	603	–	319	346	0.922	869	1.708	278	***
FCB7	4.85	–	586	–	297	357	0.832	877	1.802	234	***

* Static test specimen, ** Cracked damage specimen , *** Broken damage specimen

In impact specimens, ε_c is classified into two courses. In the cracked-damage specimens, ε_c turns back to the point E by way of the point Y as the dotted line in Fig.10. In the broken-damage specimens, ε_c passes the points Y_1 and D, and reaches the point F as a broken line. The point Y_1 represents the strain corresponding to the yield strain of the mesh. The point D is the approach point that ε_c reaches the line YF. Therefore, it can be considered that cracks become visible at the point Y_1 and begin to open from the point D due to an increase of deflection. α_{2exp} was slightly less than α_{2cal} as seen in Table 2 because the flexural capacity was decreased by cracking after impact. However, the impact damage mechanism seems to be similar to the static one in the range beyond the strain of the point D.

Observing the behavior of the strain immediately after impact, the values are larger than for the line AC. Moreover, the type of impact damage is classified according as the strain passes across the extension of the line FC or not. The strain passes across the extension in a broken-damage specimen. Localized damage is formed in the range up to the point D.

Load-Strain and Load-Deflection Diagrams of impact

The load-compressive strain curves are shown in Fig.11. The circle, the triangle, and the open square correspond to the first crack, the mesh yield, and the point D of Fig.10 respectively. All first cracks occurred during an increase of load. The mesh yield was located over the peak load in the cracked-damage specimens, but was located near the peak load in the broken-damage specimens. The open squares are located at the end of the impact load or near the minimum load. The decrease of its impact load seems to be caused by yielding of the mesh. Moreover, cracks become visible at the triangles and begin to open from the open squares as mentioned above. Consequently, it is considered that localized damage occurs mostly by only the load immediately after impact. In addition to the above, it is suggested that improvement of yield load is very effective in improving impact resistance.

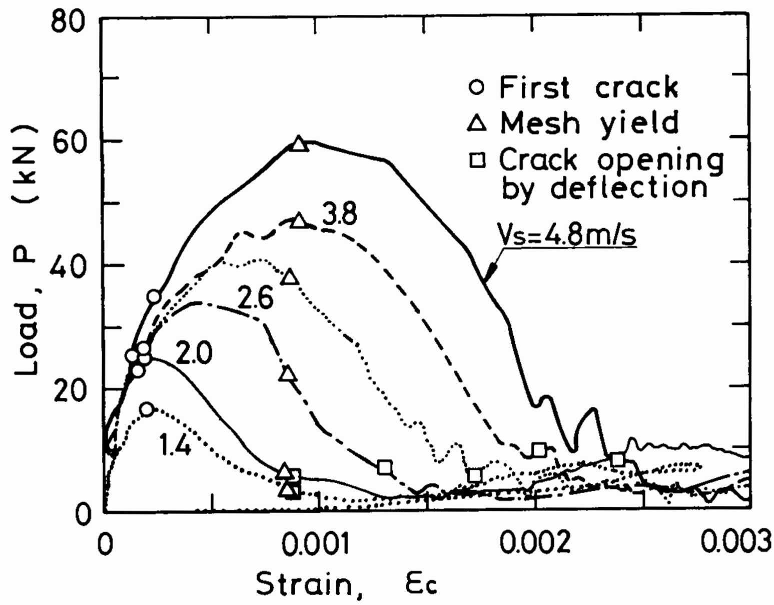

Fig. 11. Load-strain diagram in impact.

The load-deflection curves are shown in Fig.12. The triangles and the open squares also correspond to the mesh yield and the point D respectively. The solid squares represent the final failure, that is, the compressive failure of the mortar as shown by the point F in Fig.10. As mentioned above, localized damage is formed during the process up to the open squares. The value of deflection for localized damage is approximately a quarter of the deflection for final failure. Cracks open during an increase of deflection from the open squares to the solid squares. At the same time, the compressive failure of the face mortar occurred during an increase of load.

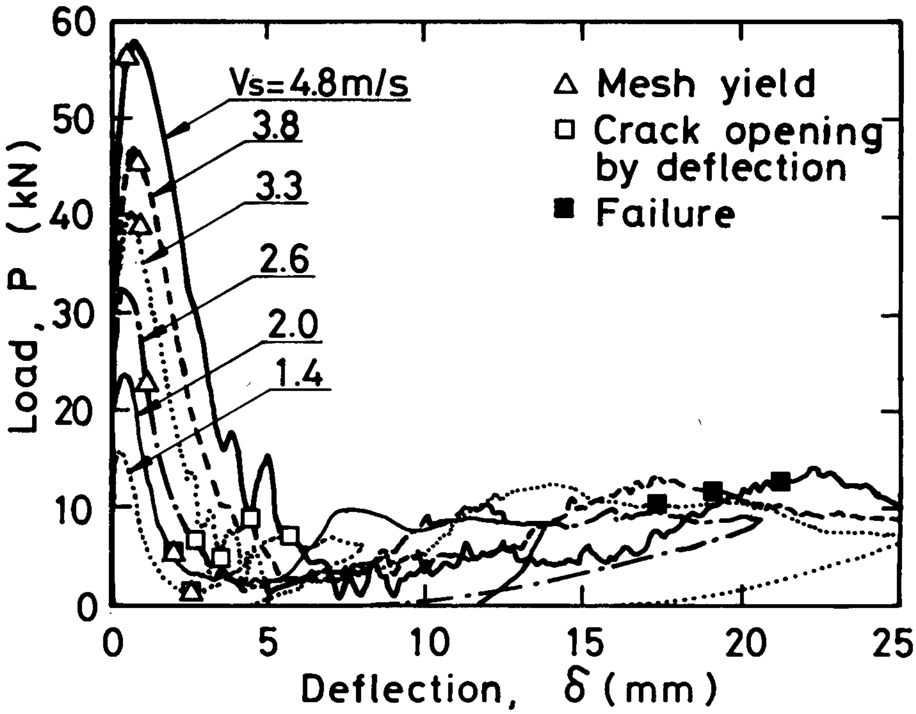

Fig. 12. Load-deflection diagram in impact.

Absorbed Energy in Impact

Each absorbed energy is defined in the schematic diagram of load-deflection curves in Fig.13. E_{cr}, E_y, and E_{ld} represent the energy of first crack, mesh yield, and localized damage respectively. The energy of the final failure, E_f, has two definitions with respect to the deflection range for integral. The integral range is until the maximum deflection in the cracked-damage specimen and until the deflection at the compressive failure of face mortar in the broken-damage specimen. The recoverable

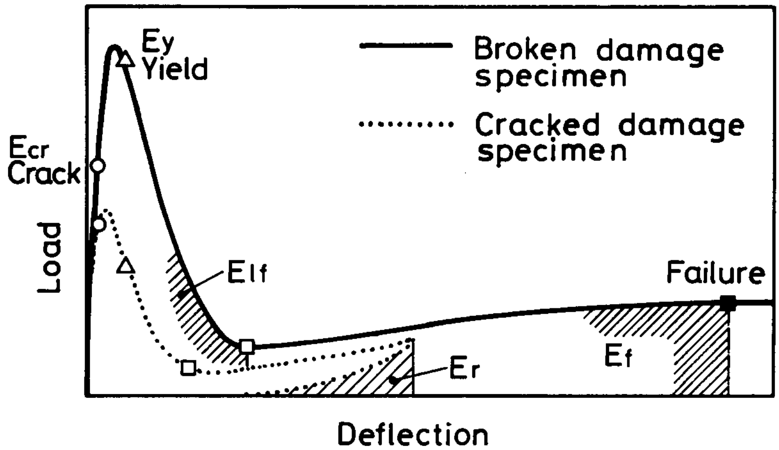

Fig. 13. Definition of absorbed energy.

energy, E_r, expresses until the maximum deflection returns to zero or free load in the cracked-damage specimen. These absorbed energies were obtained practically from the load-deflection curves of Fig.12 by using Eq.(9). To compare with the input energy "*mgh*," the obtained energy is expressed as Eq.(10) by using the effective coefficient η.

$$E = \int P \, d\delta \qquad \text{........ (9)}$$

$$E = \eta mgh \qquad \text{........ (10)}$$

The energies at first crack or mesh yield are plotted against drop height in Fig.14. The data in static test were laid on the vertical axis (h=0). E_{cr} gave approximately equal values irrespective of drop height. E_y reached the limited capacity, but decreased slightly for $h > 0.7$ m. According to the decrease of E_y, it is necessary to consider the effect of strain rate on the yield strength of mesh or the strength of mortar.

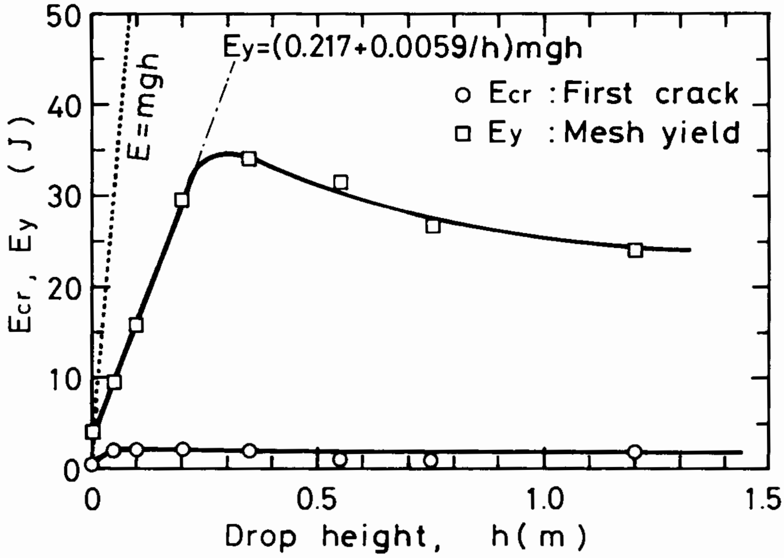

Fig. 14. Absorbed energy for first crack and mesh yield.

The energy of localized damage and final failure are shown in Fig.15. E_l increased in proportion to the drop height as given by Eq.(11), and therefore 25% of input energy was expended for localized damage. This result was not dependent on drop height or the type of impact damage.

$$E_l = 0.25 \, mgh \qquad \text{........ (11)}$$

In the cracked-damage specimen, the final failure energy increased in proportion to the drop height until $h = 0.4$ m. E_f was 85% of input energy as given by Eq.(12). E_r was 15% of input energy in each specimen. Therefore, it became clear that all the input energy was transferred to the cracked-damage and recoverable energy of deflection.

In the broken-damage specimen, E_f increased slightly with an increase of drop height as given by Eq.(13). This is also mainly caused by the effect of strain rate on the compressive strength of

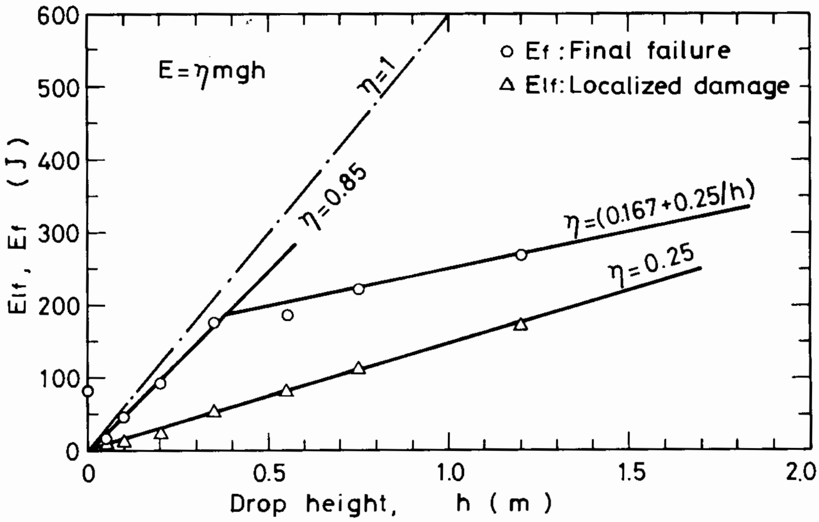

Fig. 15. Absorbed energy for localized damage and final failure.

mortar. However, the critical value of the final failure energy was decided to be approximately 190 J for $h = 0.4$ m from Eq.(13). Therefore the greater part of input energy was expended in the bending of the reinforcing bars and meshes.

For $h \leq 0.4 \quad E_f = 0.85\, mgh$ .... (12)

For $h > 0.4 \quad E_f = (0.167 + 0.25 / h)\, mgh$ .... (13)

Furthermore, E_{lf} reaches E_f at $h = 3$ m from Eq.(11) and Eq.(13). Therefore, all input energy will be spent for localized damage in $h \geq 3$ m, ($V_s \geq 7.7$ m/s). The specimen will fail by shearing force with a small deflection.

CONCLUSIONS

The properties of impact damage were obtained from the lateral single impact tests of ferrocement. The experimental results are as follows.

1. The strain at first crack in impact was approximately equal to that in static load, and was stable, irrespective of striking velocity. The deflection at first crack decreased as the striking velocity increased.
2. The peak load was proportional to the striking velocity. The value of the peak load is related to the yield of the mesh and the strength of mortar. Impact damage occurs mostly by only the impact load right after strike and became the localized damage without deflection.
3. The compressive strain-deflection relationship after localized damage was linear up to the compressive failure of the mortar. This is caused by crack opening due to an increase of deflection.
4. The reinforcing bars and laminated meshes were bent after compressive failure of the mortar, but spilling was not observed on the tensile cover mortar.

5. The absorbed energy of localized damage was in proportion to the drop height and was 25% of input energy. The 85% of input energy was spent for the cracked-damage and 15% remained as the recoverable energy of deflection in the cracked-damage specimen.
6. The specimen has a limited capacity to absorb energy in the final failure, but in practice the capacity becomes break in correspondence to the amount of striking velocity because of the effect of strain rate on the compressive strength of mortar.
7. Use of a higher yield mesh is effective in improving impact resistance.

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Ferrocement Durability : Some Recommendations For Design And Production

Jefferson B.L. Liborio* and Joao Bento de Hanai*

This paper presents some recommendations for design and production of ferrocement, in order to eliminate the source of many pathological problems.

These recommendations were derived from an exhaustive program of technical inspections of existing ferrocement structures in Brazil, that had been subjected to different climatic and environmental conditions.

Structures up to 30 years old were examined and the main durability factors, both the positive and the negative were identified. Design, production, maintenance and repair techniques were identified, classified and the most appropriate for ferrocement applications in civil construction were selected.

FIRST CONSIDERATIONS

The present paper was based on knowledge of pathology of concrete buildings, adapted to specific conditions of ferrocement considering the main production process being used in Brazil and results of observation of existing buildings [1].

Its objectives are to provide information about durability of ferrocement buildings and to present possible problems and solutions during project execution.

The durability of ferrocement buildings is a topic which deserves a special study considering the still relatively few experience accumulated. To better understand the related features and later technological development, it is also important to analyse some of the ferrocement implementation conditions in Brazil.

By the middle of 1989 the ABNT (Associacao Brasileira de Normas Tecnicas) through a national election approved the standardization process CE 18:05.14-001 " Ferrocement Project and Execution", registered at INMETRO under number NBR-11173(2), which includes among the provisions many features related to durability of ferrocement buildings.

This pattern specifies ferrocement as a particular type of reinforced concrete to be used in small thickness mortar within classical definitions , and steel wire mesh armature of limited mesh interlacing, distributed all over the cross section, having the conventional value of 40 mm as the small thickness upper limit.

The present study establishes other supplementary definitions and conditions of material applicability, execution techniques and project procedures, aiming to accurately evaluate the known field of ferrocement technology [3].

Its usage is commonly recommended for buildings and parts in which reduction of their own weight, permeability to water and cracks are essential.

Considering resistance, an idea of implementing general conditions, the current Brazilian Standard sets up the following:

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- at highly aggressive environments as well as at other adverse conditions related to durability of the mortar and armature, special protective measures have to be taken, of proved efficiency;
- the execution of ferrocement without a quality control can only be made in small structures.

The application field is then conditioned to take into account the durability of ferrocement building. It is however to be pointed out that many buildings had been constructed at highly aggressive environments and micro-climates and showed satisfactory performance.

RECOMMENDATIONS

From the information obtained from building inspection, experimental laboratory researches, project analysis and comparison with data from other technologies, recommendations were set up. Some of these are effective and some are on conditional basis, as a partial synthesis of the available data.

Such recommendations comprise questions about advantages and limits of material and components used in a building, besides the construction process.

That means that, the level of technological adequacy of the possible solution, that is, if the material and techniques chosen are able to meet the demands at each particular case must, be evaluated. The technological adequacy can then be extended based on a set of decisions made by the executor of a project or work, which must be sufficiently implemented in due time, on the decision level of the knowledge or domain of building technologies and the field where the intervention will take place. The competent technological adequacy exercise is not easy, and is usually obtained through trial and error. Therefore, the presented indicators are only a reminder on some aspects to be considered when planning and designing a ferrocement building, as in this specific case.

The serviceability of a building, in general, depends upon the building patterns and their functions, which also vary with the times; it is necessary then to evaluate the specific possibilities of ferrocement for the particular case in question.

It seems relevant however to emphasize that ferrocement has been successfully used in emergency building, for instance slums urbanizing.

To foresee the existence of slums and their existence for the next 50 years, for example, is an atittude to be faced with least resistance. However, the conscientious application of a technology to diminish the most adverse social and economical impact in the present, can be seen as a trial of its preservation for more effective future application

The project must be sufficiently detailed, including all the possible execution hypothesis, building usage and mantainance, at several identified phases. For instance, before a serviceable life expectation a sensible evaluation of the micro-environmental and environmental characteristics will give an answer to the following aspects: type of cement, armature characteristic, thickness of cover, additional protective measure, building characteristics, maintenance program, level, and so on.

The recommendations presented below are not in priority order but as a partial list of items to be verified:

Planning the Project

- The projects must be detailed as to contain all information for a perfect understanding of all the productive process, inclusively from the presupposition that ferrocement technology has no tradition in the professional applications.

• It is necessary to be able to integrate the projects: structure, protections, hydraulic, electric, and so on, preventing all possible interferences.
• Once production process has been tested and refined, preferably in an interactive mode between project and execution teams, the executive phase will have to perform its role in transposing "abstract to real", without valuable interferences.
• Descriptive elaborate details of the project as well as complete description of all production steps of the basic system must be included.
• All critical situations should be exhaustively studied until it is completely resolved.
• All hyphotesis must be rationalized to turn the project and its integrated parts into an easily feasible project with good quality control.
• It is necessary to have complete technical specifications; it is necessary specially to have complete details with new and special procedures.
• It is necessary to maintain a data bank, documenting all experiences and collecting information about present tendencies and new advices.

Armature

• Effective characteristics of the employed steel and the commercial forms [panels, bobbins, (spool)etc.] should be verified, for adoption in the standard procedures for quality control.
• In any situation, it is necessary to evaluate possible existing faults in wire mesh and bobbins, the stretching out, the inspection, fixing and straightening needed before its usage.
• The several armature arrangements have to be tested, preventing possible unfavorable situations.
• The fringes should be considered in armature arrangements, preventing more unfavorable situations when adopting a thickness for the elements.
• According to the Brazilian Standards - NBR 11173, arrangements with a unique steel wire mesh are allowed in parts up to 20 mm thick; for thicker parts, two or more layers of wire mesh must be used.
• Wire and bar diameters (besides wire mesh) which make the armature should not be more than 1/4 of the part thickness or larger than 8 mm.
• The flexing plan and armature assembly sequence should be established.
• Flexing should be avoided in regions with low ductility and attention must be given to the minimum radius of the curve allowed in each case.
• All corners and extremities should contain, even constructively, at least a minimum 3 mm diameter wire.
• It is necessary to adopt constructively a skin armature when $h > 10\ t$ (height related to considered element thickness) composed by a minimum 4.2 mm diameter wires and distributed along the cross section of the elements, according to NBR 6118, until researches and other scientific data are obtained concerned with ferrocement.
• The reduction to the extreme of element dimensions through excessively complex armature arrangements should be carefully studied; this could not be constructively possible, for it is better to elaborate moulds to ease armature positioning and adjustment, observing lack of precision.
• Gaps are to be allowed in strategic position of the elements so as to absorb the effect of lack of precision during execution.
• Certain problems could be diminished, by increasing thickness of the ferrocement in critical regions giving better armature protection, which would allow the principal objective of permitting a larger gap at armature production and assembly.
• Care should be taken when positioning the armature, considering its little stiffness and little tolerances required.

- According to NBR-11173, positioning armature tolerance, in relation to protection, is 2 mm.
- Armature preparation should be accomplished, generally with the aid of gauges.
- The usage of interlaced wire mesh, such as chicken wire mesh and a "sieve" type should be wisely verified and its resistance should be established by experiments.
- The usage of expanded wire mesh can be carried out carefully, considering that there are still little data about the subject, despite indications of good future outlook.
- Intermittent steel fibers, although there are few works related to ferrocement technology, present good usage perspectives, considering that there is an enormous range of works in the international literature. But their usage as well as their virtues and limits must be carefully evaluated and previously tested as to justify the choice.
- Armature with different characteristics should be avoided in the same element in order to prevent differences of electrical potential.
- When galvanized armature is used, zinc corrosion chemical inhibitor should be used in previously analysed amounts.
- The fixing of all armature compounds can be made with overbolted wire, which will be a part of the armature that should be protected taking into account the protuberances added to the armature.
- In any situation a geometric rate of minimum armature of 0.60% should be adopted, considering the two directions.
- Store skeletal steel, protecting them from adverse climate.
- The precast armature must be carefully checked with suitable gauges for each case.
- Other dispositions about armature; like type and dimension of steel wire mesh ,armature wires and bars , data about bond and anchoring, as well as other construction dispositions, are given by NBR11173.

Cement and Aggregate

- The aggregates must be graded according to technical standards.
- Composition, quality, reactivity, etc, have also to be observed.
- In some cases use 40% of large aggregate, in relation with the total mass of aggregate, with characteristic maximum dimension up to 9.5 mm, according to limitations of armature arrangements and the formworks.
- Recommendations made by Tezuka (4), in "Practical Guide for the usage of Hydraulic Cements" ("Guia Pratico para o Uso de Cimentos Hidraulicos"), from Portland Cement Brazilian Association/ SP, can be used to choose cement type.
- ARI (High Initial Resistance) and ARI-E-MRS cement types can be used for ferrocement elements, for quick demoulding.
- Lafuma's advice, reported by Canovas (5), established the cement amount in relation to aggregate dimension by Eq. 1 , but observe the minimum value of 450 kg/m³ and other specific reccomendations.

$$P_c = \left(\frac{700}{\sqrt[5]{D_{max}}} \right) \qquad \text{........ (1)}$$

Where, D_{max} = maximum dimension of aggregate (mm);
P_c = cement mass, minimum (kg);

• The ratio between dry aggregate and cement mass can be up to 3.5, this is necessary to consider all involved parameters in the production and usage of elements and systems;

Water

• The water/cement ratio should not be more than 0.45, by volume but should not exceed 250 kg/m^3, in usual cases; however in aggressive environments this value should be close to 200 kg/m^3.

• Water must conform to the requirements of NBR-6118 and NBR-11173.

Additives and Admixtures

• The usage of additives and/or admixtures can bring benefits when improving the mortar, and mixed according to instructions from suppliers since they know the interactions between additive / cement and additive/additive, when it is the case.

• Trials should be accomplished for the usage of admixtures and additives, to predict actual behavior (temperature, air relative humidity, mortar manipulation conditions, etc).

• Usage of admixtures and additives should be foreseen in project, as well as practical orientations for previous analysis and usage.

• . Additives containing chloride should be avoided or carefully used.

• Some admixture with goodreaction in reinforced concrete can have opposite reaction in ferrocement therefore it is necessary to check beforehand all instants of element fabrication.

Curing

• The curing of ferrocement elements should be done carefully, according to proportions and specifications of the usual procedures of reinforced concrete technology.

• . Curing by immersion, chiefly when using metallic double formworks with minimum requirements give satisfactory results.

• . In case of short-time demoulding, in general, the curing procedures should continue for a period of 7 to 10 days in saturated environment, or until the mortar reaches, at least, 70% of its characteristic resistance to compression foreseen in the project, taking other conditions into account, as for instance the usage of slow hardening cement, special requirements established in project, etc.

• . Thermal curing is another possibility to be considered of special advantage in ferrocement production, although there is not enough experiences to establish objective recommendations. Until enough studies are available, one can initially adopt the procedure in the concrete field.

Formwork

• In case of usage of vibrating tables and easels, the formworks must be rigid enough to transmit vibration to all points in the most possible uniform way.

• The formwork must be duly engaged and watertight, preventing emptying of cement paste mainly at junctions.

• The formworks must be rigid enough as to prevent dislocations and deformations when placing and thickening of mortar.

- It should take account and provide for the position of holes, insertions, raising handle, cuttings, protuberances and the like, as well as respective dimensions and tolerances.

Quality Control

- In general, specifications of NBR-11173 should be followed.
- In the major part of the cases, it is necessary to adopt, as control, that mortar mix should be according to standards, and should not be more than $1m^3$ to $3\ m^3$, considering that this amount is enough to generate a large quantity of elements with variable characteristics.
- The time difference between the mortar mixing and its final application should not exceed 20 minutes.
- In case additives are used, this time difference should be reevaluated.
- When temperature variation is higher than 10°C, a new consistency test must be undertaken.
- Moulding of ferrocement elements must be performed in protected environment.
- Before the production it is necessary to choose which plastering technique brings better result.
- Good workmanship in ferrocement buildings can be obtained with suitable training, with emphasis on controls and techniques.
- It is desirable to have professional supervision in several areas, in order to obtain the many techniques and respective control requirements: e.g. mixing of mortar, armature production, formwork , plastering, curing,storage, assembly, etc.
- For definition of inspection parameters in relation to corners, colour, burrs, texture, bas-relief and the like, the manufacturer or the contractor must show representative samples of the specified quality, which must be checked by the owner and used for comparison for the quality control of the finished work.
- The part of the building in ferrocement are considered acceptable if the project specifications, execution and quality control conformed to accepted standards.

Environment

- According to NBR11172, in highly aggressive environments as well as in other conditions adverse to durability of mortar and armature, special measures for protection of proved efficiency must be used.
- Flat covers, beams-gutters, etc, even in an environment with rural characteristics can be considered in "medium aggressive" environment up to "highly aggressive" environment, depending on circumstance.
- Reservoir covers and closed galleries upper parts must be considered as aggressive environment.
- Consideration of galleries which receive only fluvial affluents must be carefully made, considering that the situation, at least in Brazil, has been constantly changed by clandestine drain.
- Any exposed surface of ferrocement must be considered as in a least medium aggressive environment.
- Protected elements but exposed to water should also be considered as in a least medium aggressive environment, varying the degree according to function of usage (example: kitchens, bathrooms, toiletes, places of public crowd, etc.).
- In regions with great humidity , such as maritime and industrial regions, structures must

be considered as in an aggressive environment.

- Armature cover in compressed regions can reach upper value of 6 mm and in case of medium aggressive environments this value must be at least 10mm, unless a protective cover is used, as well as regular maintenance.
- In case of tractioned regions, the cover should not exceed 6 mm, considering that, in highly aggressive environments, it should get additional protection.

Protective Measures

- The adoption of protective measures, as covers and painting, must be based from experiences or according to tests of the specified product performance potential.
- The literature has reported that the incorporation of hydrorepellent in concrete has proved inefficient in more severe cases, but they can be applied as complementary surface protection in less severe situations.
- Protective covers must be applied according to specific recommendations, taking into considerations relative dislocation occurrence between the cover and mortar surface or between cover layers.
- One cannot think of adopting extra protections, considering initially a doubtful quality mortar and ferrocement element.

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Ferrocement and Replica Ships

Lawrence M. Mahan *

Future maintenance and repairs for wooden replica ships can sometimes cost as much, if not more, than the original construction budget. If the hull portion of the building process was constructed using ferrocement as the building medium, future maintenance and repair expenditures would be lessened to a great degree.

Ferrocement as a hull material can reduce this high maintenance overhead to a minimum and the savings can be used to keep exposed woodwork and trim in proper condition. An example of this type of hull material is described in this paper. By adhering to traditional wood building methods for all the train and railings, the look of a period is maintained with the benefit of a no row and low maintenence hull and deck structure.

INTRODUCTION

During the past twenty years, many maritime organizations and museums have elected to construct period replica vessels. Some of the motives have been historical but most of the vessels were built either for publicity or revenue generation. Authenticity was not always the main theme as some of the ships were to be used for sail training or other programs. This meant certain current government regulations had to be met for safety reasons resulting that many original aspects of construction of design needed to be changed to conform to the regulations.

Most of the changes needed caused little concern as the visual look of a period vessel did not seem to be seriously effected. These structural and safety modifications also apparently do not alter a person's opinion as to whether or not the historical vision of the ship has changed. Most people think not. Since the previously stated changes do not basically effect a person's idea of historic ships why not use more practical building materials for hulls and decks?

Every naval architect knows the severe wood deterioration problems a wooden vessel endures throughout its life. The following group of historic ship replicas, with the exception of the Sea Lion, have had severe and costly repairs done to planking, frames and decks.

The May flower, launched in 1956, has had numerous planking repairs along with extensive other work about every fifteen years since it was built (Fig. 1). Even now, several deck beams have developed serious rot to the extent that on a recent sail, the first in many years, one of the deck beams collapsed due to sailing strains.

A typical rot section shows how serious this problem can be (Fig. 2). This area has broken apart and other wood has been fitted to carry the load. Repairs to structural knees become very involved and are labor intensive. All of this ongoing repair work appears on a regular basis at considerable expense. Recently, a ship called the Rose, launched in 1969, has had over 1.5 million dollars spent to repair upper hull planking and frames.

Taking all this into account, it is proposed that some future period vessels be constructed utilizing ferrocement as a hull and deck material. A low maintenance and rot free hull and decks would be

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Fig. 1. The Mayflower.

the result. Applying careful consideration to trim work and deck structures, the authentic look of history would be preserved. Once painted the ship would, for all practical purpose, appear realistic yet could be sailed with complete structural integrity and safety.

The money saved on initial construction could be carried over to the ongoing maintenance program and the outside trim and other woodwork would be kept in proper shape.

Fig. 2. A typical rot section.

CONSTRUCTION PROCEDURE

The standard removeable pipe mould method was used and all mesh and rod layers were formed around it. Alternately, between each mould pipe, web frames were fabricated in order to provide strength and structural members for later internal framing and bulkheads. Window and port openings were made from stainless flat stock and provided a corrosion free screeded edge to the

Fig. 3. The standard removeable pipe mould method.

cement work. The hull was plastered from in to out and work starting at seven in the morning, was completed by six in the evening. The hull surface amounted to 312.5 m^2 (3200 ft^2) and 130 people were required to complete the plastering phase.

The decks were individully done at later stages with small groups of plasterers. Carrying over the hull cement onto the deck by one foot realizes a tremendous structureal advantage. Using this idea, the weak hull/deck intersection of separate cementings was avoided.

Inside the hull, web frames(Fig.4) were also cemented at the same time as the hull and at the deck/hull intersection, great strength was derived this way. Starter mesh and rods were left protruding to connect with the future deck and deck web frames. Lower deep floors, or short bulkheads, were plastered later along with the full size bulkheads.

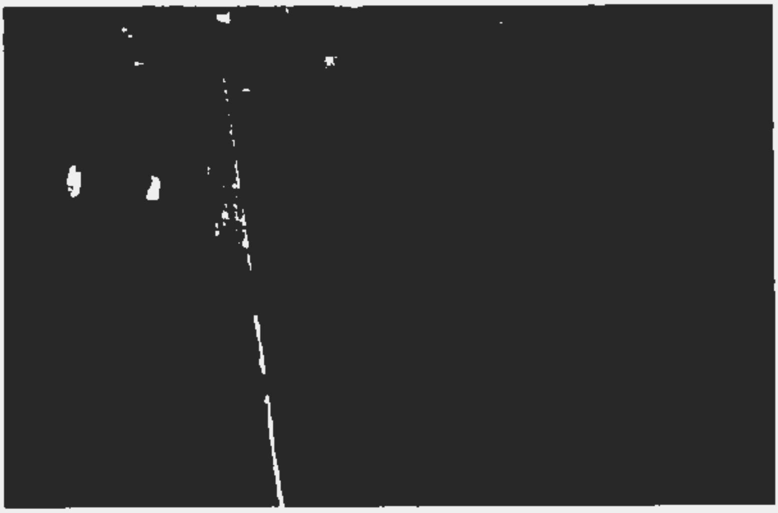

Fig. 4. Plastering of web frames.

Once the hull was cured and all traces of lime ceased to leach out of the cement, the whole structure was epoxied and painted. At this point the first outside woodwork could commence.

A temporary moulding method consisting ofwood slats and plywood was formed and held in place by ordinary rope tensioned with mechanical devices. The plywood forms the fair backing needed for subsequent application of multiple layers of thin hard pine planks. Epoxy was used to glue all the layers together.

After the laminated coaming was completed, deck covering boards were fitted for the coaming to fit and fastened against. While the coaming board was being fastened, wood trim was being attached

Fig. 5. Moulding method consisting of wood slats and plywood.

to the ferrocement coamings around each hatch opening (Fig. 6). Now that the finished coaming is a permannent part of the boat the railing support posts can be let into the covering boards and bolted in place.

A large number of posts were fashioned and in addition to providing support for the bow railings, others were bolted in place to support and give strength to the center deck bulwarks. At this time all of the finished hatch coamings were fitted with their respective covers.

Instead of the usual glass plate for the hatch lights, a large sheet of 9.525 mm (3/8 in.) thick lexan was used for strength and safety. This material can even be hammered without breaking yet is easily drilled or cut.

After the bulwark posts were properly bedded and fastened to the deck, the bulwark side became the next phase. Standard wooden boat-building techniques were used in this area and each plank was sprung to the curve of the deck. Where the planks touch the deck and meet the hull, epoxy filler material was used as a fairing in compound. Once smoothed off by grinding and sanding, the cement and wood joint cannot be detected.

Continuing back to the bow railings, other structural members were bolted into position. The cathead beam was used as a structural knee and was bolted through the deck at the inboard end and also bolted approximately at the center of the railing. The purpose of the cathead was to provide a strong support for the tackle used in raising the anchor up to the railing, where it was tied off when sailing.

Fig. 6. Wood trim attached to the ferrocement coamings.

Hard pine laminates were used in all areas of construction where extreme curves were found. A good example of this method was shown in the forward section of the rub rail. Each layer varies in thickness from 12.7 mm to 19.05 mm (0.5 in. to .75 in.). The gluing process involved temporarily bolting the sections in place until the epoxy glue had cured. Only the forward sections of the rub rail needed this laminating method (Fig. 7). The rub rail areas with less curve were placed with steam bent solid planking.

The ornamental head rails were built using the laminating systems and a mould was constructed giving the proper form. After the finished rails were set up, laminated wood knees were bolted through the deck for support. Where the rails terminated at the stem, wood side pieces

Fig. 7. Laminating method.

were fitted. Holes were drilled through the ferrocement stem for fastening the side pieces.

Planning the aft deck bulwarks involved making a temporary mould system in order to plot the correct angles and curves for posts and planking. All of the posts, with the exception of a through deck post at the forward end of the aft deck, were fitted and set up as done on the forward deck. The through deck post was fitted to maintain needed strength at the rail end. Holes cut in the deck were made by drilling multiple holes along marked lines and knocking out the resulting plug. A small impact drill was used for all cement drilling. Any steel that was met was cut out with an oxygen/acetylene torch or cut-off wheels. All openings were then smoothed off with various sized grinding stones.

After the post was fitted and bolted in place the rest of the bulwark posts were made and drilled for through bolting to the deck. A special drill system was set up in order that the long holes needed could be accurately drilled.

All of the deck covering boards were bedded in an epoxy compound for a perfect fit to the deck surface. When the boards were bolted in place the bulwark posts were set. Every other post was short and these provide a mounting point for the cap rail that fits over the side planking.

The large railing posts were bolted in position and the bulwark rail cap can be lowered into position and through bolted to the short posts. Side planking can now be fitted similar to the center deck.

The quarter window opening edges were filled to provide a flat surface for mounting the lexan light. An epoxy filler was made from small sawdust particles mixed with epoxy to the required density for non-sagging placement. A temporary mould was made and taped in position using duct tape. The same saw dust epoxy mix is also used to fill in the area at the hull, deck and bulwark joint. Several coats were needed to realize a smooth invisible joint.

The final coat was done using an epoxy filler without sawdust. Paint was then applied to completely hide the dissimilar surfaces. Wood trim for the quarter windows was fitted and through bolted over the lexan lights.

Side trim wood work was cut to fit and bent into position. It was then bedded in caulking compounds and through bolted to the hull. Small 9.525 mm (3/8 in) galvanized carriage boltswere used for the trim with 15.875 mm (5/8 in) carriage bolts holding the rub rail in position. Large backing blocks of hard pine were used inside the hull for reinforcement of the rub rail bolts.

Final color paints were added to visualize overall effect. The planking seams on the bulwarks were filled and after sanding and paint, the complete surface appeared as one.

Because of the large size timber used in the construction of the transom framing, correct size bolts have to be placed in strongly reinforced areas. Inside the ferrocement hull, hard pine backing of similar size was cut to shape and bedded in an epoxy compound to help reinforce this area.

All of the wood framing was contour fitted to the shape of the hull. By marking the hull with coloring crayons, a wood to hull contact pattern was made. All the high spots on the wood were sanded or planed off and by repeatedly doing this, accurate fits against the hull were produced.

Filler blocks were cut to fit between the verticle framing and inside, backup blocking was set in place for through bolting.

All of the outside framing was also bedded in an epoxy compound. The compound gives perfect hull contact and also prevents water from entering this joint.

The top curve was defined with a long clear wood batten and filler blocks were cut to fit, epoxied and bolted in position.

The transom frame, now complete, requires that the steam bent cypress planks be fastened into position. The cypress planking comes from an old water tank and is of perfectly clear stock.

The transom rail cap was fabricated by laminating multiple layers of 6.35 mm (1/4 in) hard pine planks. Epoxy glue holds all of the layers together and the finished piece was fastened onto the transom frame. Transom trim and decorations (Fig. 8) were carved from hard pine and add personality to the project. After these pieces were in place the boat seems to come alive.

Below, inside the boat, floor timber support beams along with an engine bed were set and bolted in place. The deep floors, or short bulkheads, provide excellent mounting positions for this framework.

Fig. 8. Transom trim and decorations of the ship.

The ferrocement web frames allow easy mounting of interior framework for future joinery. Between the web frames, secondary wood frames were glued into position for attatchment of the wood ceiling. Floors were laid down over steel fuel tanks and are bolted to the floor framing system.

All verticle posts were connected to the hull with steel weldments. Another bracket was attached overhead to one of the deck web frames. These steel fabrications were coated with epoxy and were bedded with an epoxy saw dust mixture. Companion back up steel was on the opposite side of the ferrocement web frames and deep floors.

Interior joinery follows standard boat building practices with all connections to web frames or other ferrocement supports being through bolted. The lexan skylight hatches let natural light down below and give a warm pleasing look to the woodwork.

The aft cabin was designed to fit the complex shape of this stern area, while the galley utilizes the available light from one of the large quarter windows. Overhead web frames were boxed in and appeared as wood beams.

Forward, the curved staircase leads down into the crew quarters and main head area. Often the inside wood backings for the rub rail and other trim could be used for interior attatchment. Overhead wood trim can be epoxied in place and as usual, the overhead web frames are convenient connecting points.

Back up on the aft deck, openings were cut to install large posts for the pilot house frame. The main posts pass through the deck and fasten to steel weldments below. Additional steel fabrications were set in the deck openings and provide strong support at this important joint. Where posts are not required to pass through the deck, steel brackets and framing were built and fastened to the deck. The smaller posts were then fitted to these brackets and bolted in place.

All of the hard pine timber used in this construction was salvaged from locally demolished factories. Most of it was 75 to 100 years old and was of a much better stock than wood cut today. Each beam as salvaged, was resawn with a portable chainsaw mill. Once cut to rough size it was run through a large power planer and trimmed to the proper size. The chainsaw mill could be used anywhere and locally cut trees could be used to produce lumber. This would have to be seasoned of course before placement in the boat.

The old hard pine has proved to be an excellant wood for this project (Fig. 9) and it was highly rot resistant because of the natural resins it contains. It holds fastened well and has the strength needed in critical members.

Maintaining the top side wood rails (Fig. 10) and trim consists of three yearly applications of a substance called pine tar. A person can easily see if a rot problem develops by using this oil as the wood grain is always visible. A side benefit is the original look it gives a period replica.

The decks can be painted or, if a more traditional look is desired, a wood overlay could be installed. This would add considerably to the cost of the boat and with heavy foot traffic, would require constant upkeep.

Fig. 9. Ship made of old hard pine wood.

In conclusion, based on structural and cost effect , this method of building a workable copy of a period vessel has much merit. Early designs are easily constructed in ferrocement because of the heavy displacement style that early designers used. Once the hull is constructed and the wood trim and deck structures finished, painting and other cosmetics give the visitor the appearance and feel of the period vessel it represents. Future maintenance and upkeep costs will be at a minimum and certainly no extensive hull rebuilding will be required.

Several personal visits to all the vessels cited in the text for first hand inspection of deterioration problems were included in the research.

Fig. 10. Topside wood rails of the ship.

CONCLUSIONS

The author's own personal experiences with ferrocement date back to 1968 when he was asked to construct a simple steel framework over an existing wooden boat for later application of a ferrocement covering up to the deck line. Since that time he had worked on many other ferrocement boats as well as redesigning and constructing the schooner featured in this paper.

Over the years he has been contacted to perform survey work for people planning to purchase ferrocement boats that were either completed or in various other stages of construction. All of this experience has helped him understand and promote this unique construction method of utilizing cement and steel.

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Use of Ferrocement Panels in Large Span Roofing System

S. F. Ahmed* and H. Dawood**

Prefabricated ferrocement panels offers a variety of possibilities to be used in many locations where economy, ease of construction and aesthetics are of prime importance. The objective of this paper is to discuss the design, fabrication, erection and construction technique for shell-type ferrocement units used to cover a large span gymnasium, to form a composite roof.

Considerable saving in material cost, about 20 % and a substantial reduction of construction time can be realized by employing ferrocement.

Based on the procedure described here, one industrial roof has already been built in Jamshoro, Pakistan.

INTRODUCTION

For large span industrial buildings, it is often difficult to select the most efficient roofing system based on commonly available materials and technology. Normally mechanized prefabricated systems, such as asbestos cement, galvanized iron sheets, and reinforced cement panels, resting on main structural elements are considered. This calls for sophisticated water proofing and drainage details along with vigilant supervision.

Ferrocement panels, because of their low cost, durability, and crack resistance (by virtue of shape) can effectively be used in such cases. The desired strength can be derived through undulating shapes. Additionally if these are made composite with in situ concrete, it would lead to many advantages which may not be offered by other flooring systems particularly asbestos cement and galvanized iron sheets.

Simplified prefabricating system for ferrocement panels, using labor intensive process may lead to more economy than normal large scale prefabrication system in areas where labor is very cheap.

Based on these considerations a composite system using prefabricated shell-type ferrocement panels was developed for a gymnasium building of square plan (36.5 m x 36.5 m).

REVIEW OF FERROCEMENT ROOFING SYSTEMS

A number of studies were carried out on the flexural analysis and design of ferrocement members [1-5]. A variety of forms has been investigated for ultimate strength, flexural rigidity, crack propagation and cracking resistance including V-U shaped folded plates, channels, I and hollow box sections, and shells [6-9]. Typical ferrocement elements in use have been reported [10-12]. Elements that can only be used in roofs are shown in Fig.1. Other shapes, shown in Fig.2 can be used as floors. The ductility of such units, sometimes limit their use for large spans in case of floors. However, these

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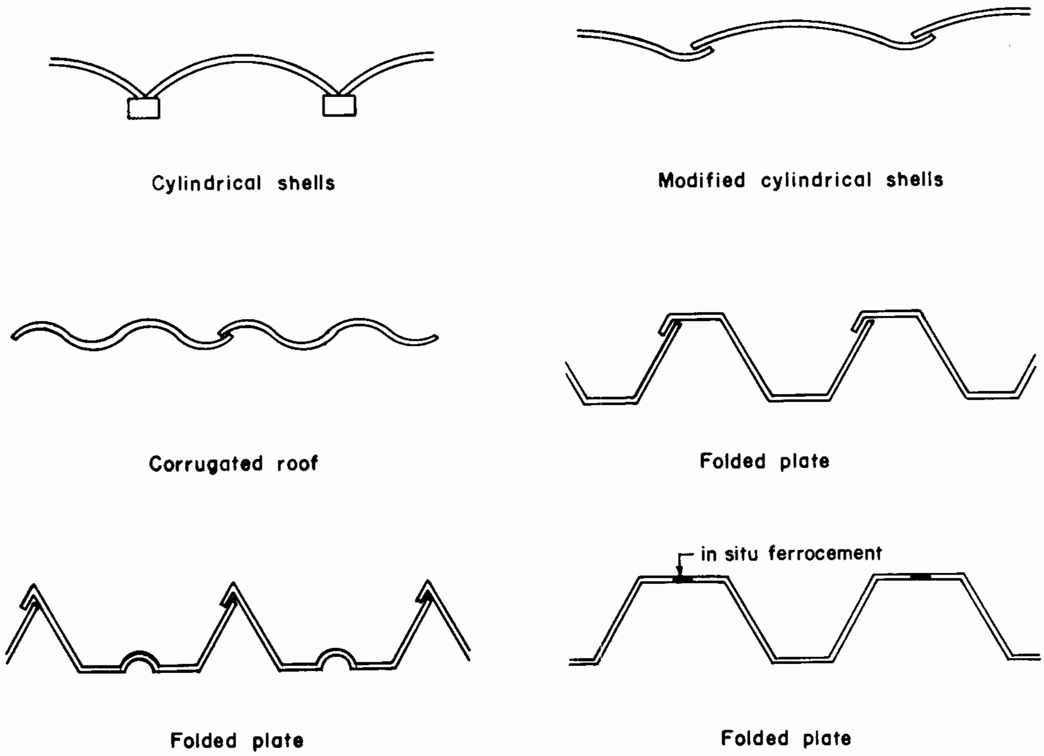

Fig. 1. Typical ferrocement roofing elements.

thin elements may not offer adequate thermal protection for roofing. The cost of services thus increases substantially. The composite system shown in Fig.3 may eliminate the above-cited deficiencies but is likely to increase the weight to strength ratio. In addition, hollow core, I-sections and ribbed slabs also pose difficulties in fabrication. It is therefore desirable to develop a system which is simple to fabricate and easy to handle which still contains all the merits desired for certain buildings. A composite system using a partial prefabricating technique, which satisfies all the requirements, has been developed.

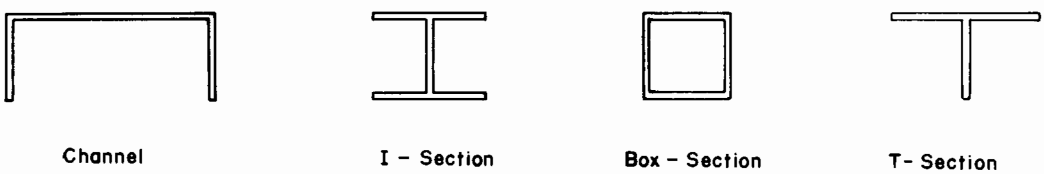

Fig. 2. Typical ferrocement floor elements.

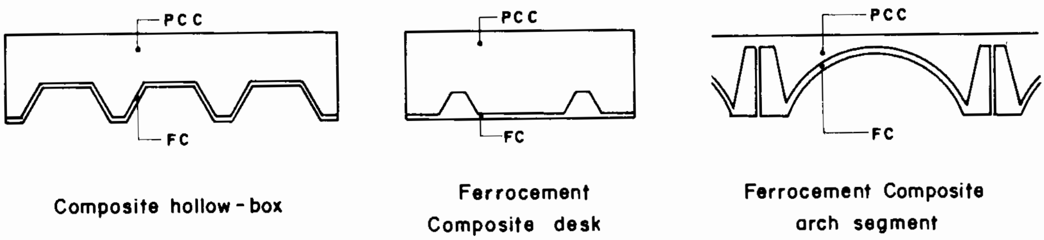

Fig. 3. Ferrocement composite system.

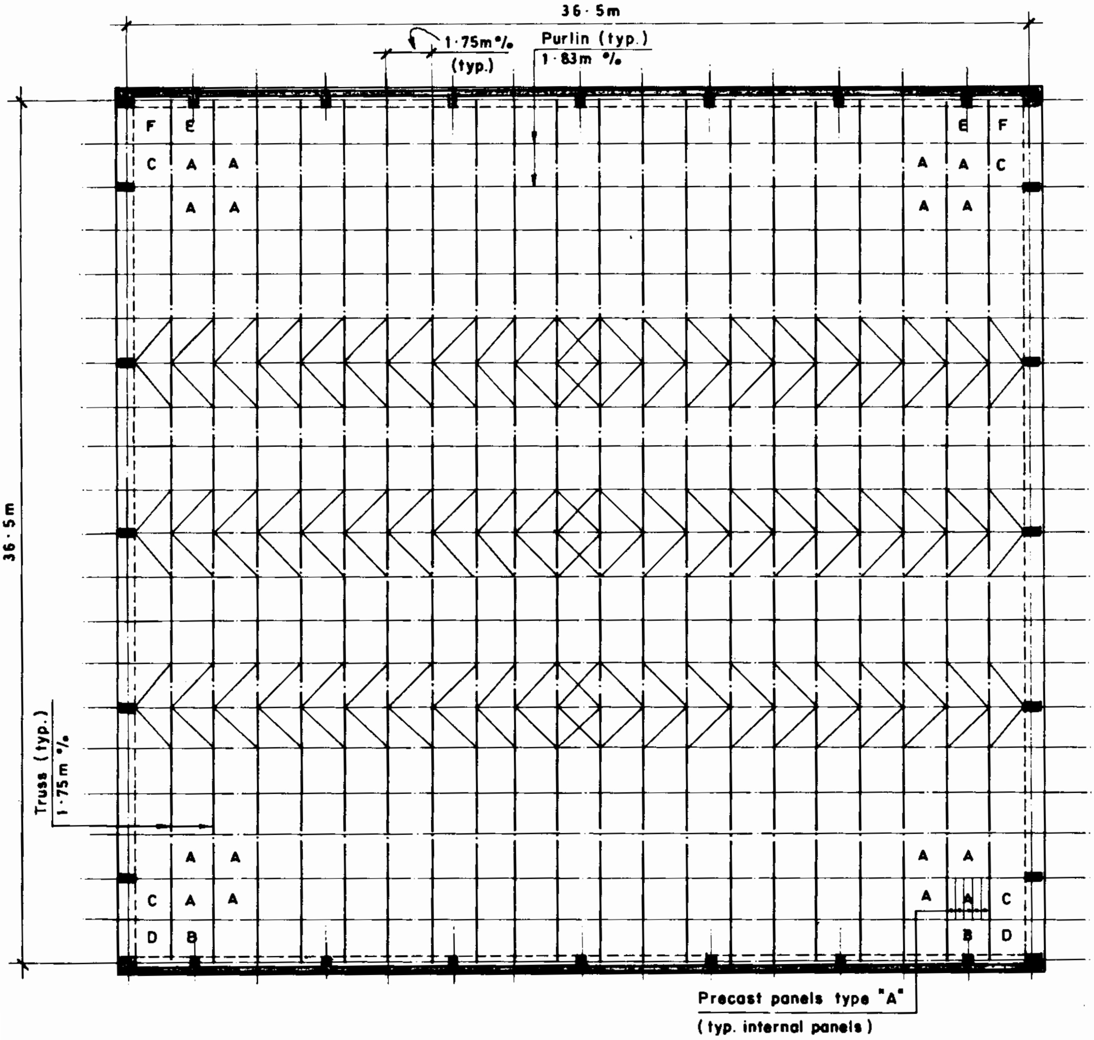

Fig. 4. Roof plan for gymnasium.

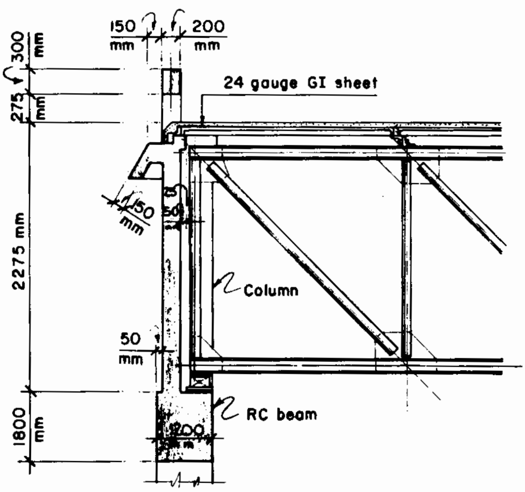

Fig. 5. Section showing truss support drainage and water proofing.

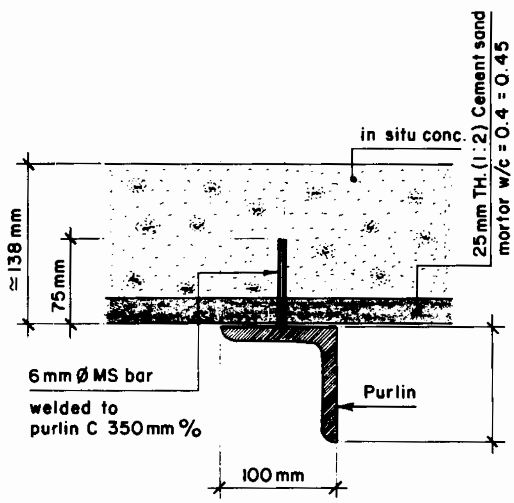

Fig. 6. Section showing shear connector welded to purlin.

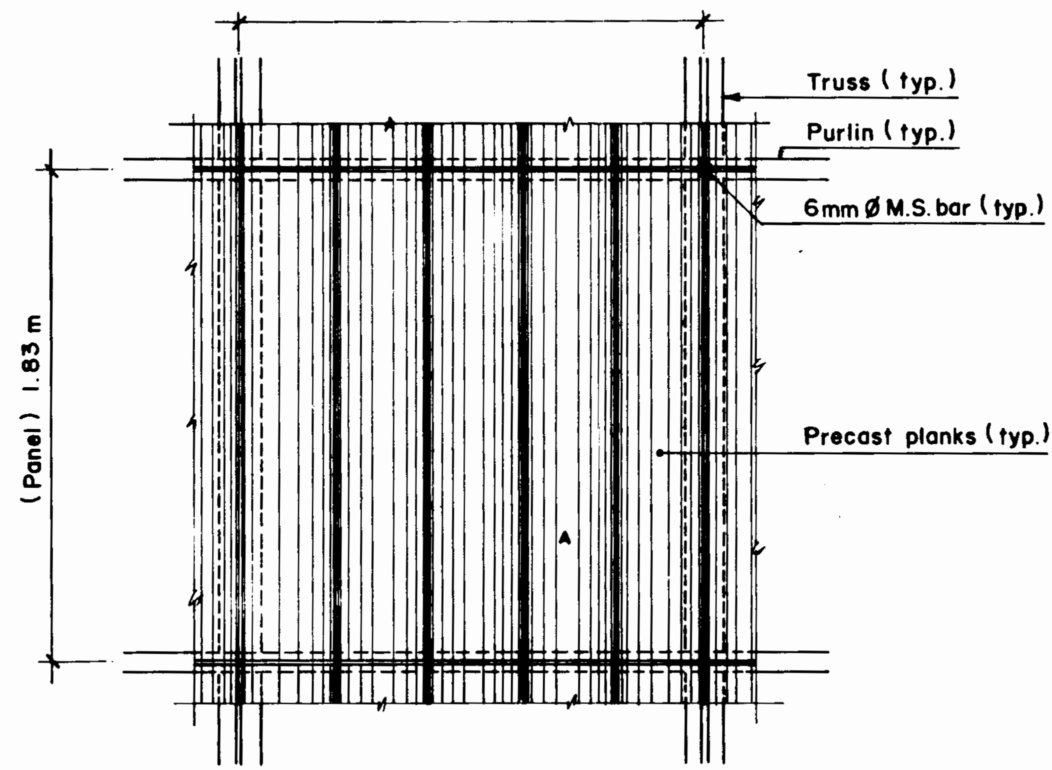

Fig. 7. Typical interior panel for precast ferrocement units.

DETAILS OF THE DEVELOPED SYSTEM

The roof plan of the gymnasium building for University of Sind, Jamshoro is shown in Fig.4. The main framing system of the 36.5 m span steel trusses spaced at 1.75 m, consists of reinforced concrete beams framed to reinforced concrete columns at the periphery (Figs. 4-5). Angle section steel purlins were spaced at 1.83 m on center with 6 mm mild steel bars welded at 350 mm as shear connectors (Fig.6). Shell type individual ferrocement units were placed side by side on these purlins. A typical interior panel of the precast unit is shown in Figs.7-9.

SEQUENCE OF CONSTRUCTION.

Individual precast ferrocement units were cast in steel moulds using 1:2 cement- sand mortar with water-cement ratio of 0.4 - 0.45. The 25 mm thick ferrocement shell panels were prefabricated with 2 layers of 0.5 mm diameter chicken wire mesh and 100 mm x 100 mm of 3.25 mm diameter wire fabric as shown in Figs.7-9. The shell units were kept under water for 7 to 10 days and air dried before placing on purlins.

Scaffoldings was erected on the sides of the building to place the first row of precast units, one10 mm diameter mild steel bar was placed from purlin to purlin in the valley between two precast units as shown in Fig.8. The placing of units progressed ahead in the same fashion to reach the center. The in situ concrete was placed from periphery towards center thus forming a composite system to act as an effective diaphragm to resist transverse and lateral loads.

The 1.75 m deep main trusses were cambered by 375 mm at the center for drainage of water. Fig.5 further clarifies the drainage details at the ends.

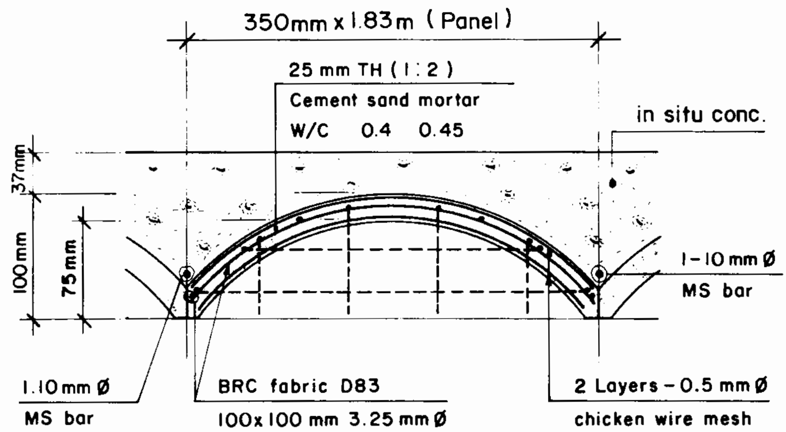

Fig. 8 . Section showing details of composite panel.

This system eliminated the vast formwork as the precast skin elements were capable of resisting the load of fresh concrete and live load during construction, finally giving extra strength and rigidity once the concrete hardened, when the system behaves as a composite.

PREFERENCE FOR SHELL FORM

For the ferrocement precast unit a shell form was preferred as alternative sections because of its near perfect suitability, ease of fabrication, handling and rigidity. The smooth curved surface gave a neat and pleasing appearance. A more efficient cross section also provided economy in materials.

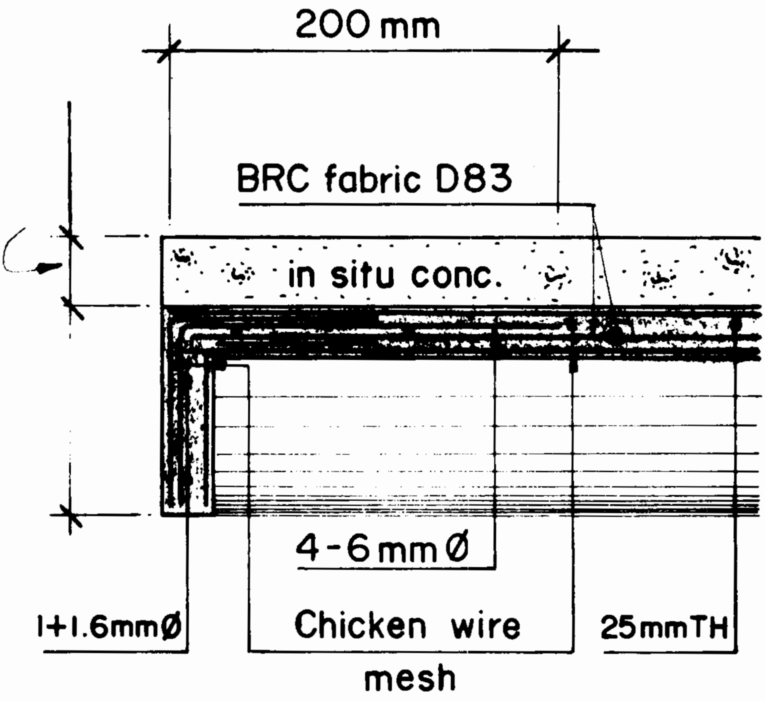

Fig. 9. Cross section of composite exposing details for end diaphram of the panel.

CONCLUSIONS.

The following conclusions and advantages are drawn from the application of the composite developed:

1. The composite roof acts as a diaphragm and results in a reduced column section.
2. The composite is more durable than the conventional asbestos cement or galvanized iron sheets.
3. Quality control is not difficult.

4. Major formwork is totally eliminated.
5. The form chosen gives a neat and clean appearance.
6. Drainage and water proofing details are simple.

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slurry infiltrated fibre concrete / cement slurry / cyclic loading

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building / structures / construction management / physical performance

NOTES TO AUTHORS

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Provide atleast five keywords that best describe the contents of your paper.

IFIC DATABASE

The INFC database will save your time and effort in finding current information on ferrocement and related construction materials. This database is created and maintained by the International Ferrocement Information Center (IFIC), Asian Institute of Technology, Bangkok, Thailand using UNESCO's Computerized Documentation Service/Integrated Set of Information Systems (CDS/ ISIS). It covers ferrocement, the form of reinforced concrete which uses hydraulic cement mortar, and closely spaced layers of continuous and relatively small diameter wire mesh reinforcements; and related construction materials, such as steel fiber composites, bamboo fiber composites, natural and organic fiber composites, and polymer composites.

IFIC regularly reviews over 100 journals, magazines, newsletters, digests and bulletins, in addition to numerous monographs, reports, conference proceedings, theses, and materials supplied directly by ferrocement builders and researchers. From these publications, articles on ferrocement and related construction materials are identified, abstracted, indexed, and entered into the bibliographic database. Each record contains the following primary information: author, title, source, abstract and keywords; and secondary information: availability, date, language and type of publication. INFC database is expanding at the rate of 300 records per year. From these records, IFIC provides computerized bibliographic search services for requests on particular aspects of ferrocement technology and related materials at the following rates:

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NEWS AND NOTES

ETHIOPIA

Roughing Filters in Ethiopia

As population grows, pollution of the rivers increases and this then calls for water treatment facilities. In most cases, conventional water treatment systems are very expensive to construct and run due to the complexity of the systems, their need for imported chemicals, sophisticated electrical and mechanical equipment as well as the need for highly trained personnel. To overcome these problems, a system of water treatment incorporating plain sedimentation, horizontal roughing filtration and slow sand filtration (SSF) with a capacity of 92 m^3/hr has been designed and constructed in Ethiopia

Horizontal roughing filters (HRF) are chambers filled with gravel ranging from 20 mm-40 mm in diameter. The presence of these gravels give the filter a large surface area to obtain a very high turbidity removal. The walls and floors of the HRF and sedimentation tanks are constructed with 0.40 m thick stone masonry. The top surface is plastered with a cement rich mix on the chicken wire (ferrocement).

A surface loading rate of 20 m^3/m^2/day for the sedimentation tanks, a horizontal flow rate of 2 m/hr for the HRF while a filtration rate of 0.15 m^3/m^2 hr for the SSF units has been adopted.

The chemical quality of the final effluent of the SSF satisfies the WHO guidelines for drinking water. Bacteriologically the influent showed a faecal coliform count of greater than 16 MPN in all seasons while the effluent from the SSF was as low as 2 MPN after 90 days of operation and nil after 122 days. The entire system is being run and administered by locally trained personnel.

(Senkut, M. 1991. Roughing Filters in Ethiopia. Water and Sanitation News, NETWAS, AMREF 1(5) : 5-6.)

Guder Water Supply Project

The Ethiopian Government, through the Water Supply and Sewerage Division, is undertaking the Guder Water Supply Project. Recepients of the project are the factory and factory workers in the town of Guder, 130 km west of Addis Ababa. Under this project the following ferrocement structures have been constructed : two sedimentation tanks, six horizontal roughing filters and four slow sand filters. The project is under the supervision of Mr. Mesfin Shenkut, head, Water Supply and Sewerage Division, EWWCA. Mr. Shenkut was a participant of the 1983 Seminar cum study tour for African officials sponsored by the UNESCO, Division of Water Resources and organised by IFIC and the Continuing Education Center of the Asian Institute of Technology.

(For further information about this project contact : Mr. Mesfin Shenkut, Head, Water Supply and Sewerage Division, EWWCA. P.O. Box 30504., Addis Ababa, Ethiopia.)

TANZANIA

Water Tank

A locally made water tank is of great importance in rural as well as urban areas for drinking and other purposes, where availability of water is uncertain and where tap water is not provided. The size of the tank depends on the the availability of water and demand for water. This water tank is very simple in construction and also less expensive, so that village people will be able to use it without much difficulty. The major feature of the tank is a dome shape structure. It is provided with an inlet pipe with ball valve for letting in and out water when it is full. Outlet pipe can be conveyed to other places such as kitchen, toilet etc., and an overflow pipe to flow out water when it exceed the maximum limit. The tank can be install temporary or permanently depending on the roof condition.

Water Tank

Top view of the Tank

The tank was constructed with a sack mould filled with rice husk and mortar with a ratio of 1:2 dry mix. A total of 20 kg cement and 40 kg of washed sand was used. The construction time for the tank was approximately one month.

(Information from Tanzania Railways Corporation, Tanzania)

UNITED KINGDOM

British Standards for Cements 1991

Revised cement standards have just been published (November 1991). This include BS 12 " Portland Cements", BS 146 " Portland Blastfurnace Cements " and BS 6588 " Portland pulverised fuel ash cements". In addition, BS 4027 " Surfate - resisting Portland cements ", BS 4246 " Low heat portland blastfurnace cement and BS 6610 " Pozzolanic pulverised - fuel ash cement " have been revised similarly to achieve uniformity, BS 4246 being redesignated " High slag blastfurnace cement".

The most important compositional changes to BS 12 are the adoption of the general European practice of permitting the addition of minor additional constituents, to most cements upto 5 % level and the incorporation of additives to improve the manufacture or the properties of the cement up to 1.0 %, ground granulated blastfurnace slag(GGBS), ground limestone or cement - making raw meal. The requirements are specified as characteristics values and conformity is assesed by means of a statistical procedure for continuous inspection operated by the cement manufacturer. All the changes made in the revision of each of the mix standards are clearly indicated in the respective foreworks.

(Concrete, November / December 1991)

Secret Additive set to Boost Epoxy's Appeal

Epoxy coating of reinforcing bars could become cheaper, tougher and more effective with the inclusion of a 'special ingredient', according to researchers at Essex University.

Alfred Tseung, professor of electrochemistry, has found that the addition of his special ingredient, which he calls Celcote, increases the time before a corrosion current starts to flow by 5 - 10 times compared with straight forward epoxy coating.

Professor Tseung did these comparisons by carrying out accelerated testing on coated samples immersed in a 3 % salt solution - nearly as concentrated as sea water. These show that epoxy resins are not completely immune to the diffusion of chloride ions and water - they just slow it down. This secret ingredient not only slows down this process further, but since it should replace up to half the epoxy used, it should considerably cut the cost of coating. This material improves the fracture toughness of the coating, so that the risk of chipping on site is lessened.

(Concrete, November / December 1991)

General purpose mortar for all seasons

A general purpose mortar developed by BRE and originally put into production by ICI is to be marketed as Limebond by Buxton Lime Industries.

In producing Limebond ICI brought together in one pack its own Portland cement and "Limebux" hydrated lime with a mortar plasticizer. By incorporating all these components in one bag the need for admixtures is avoided, a high quality mortar is obtainable by mixing Limebond with sand and water only. BRE tests have shown that Limebond is suitable where previously sulphate resistant portland cement is used.

Mortars and bricks attacked by sulphates

For some years it has been thought desirable that there should be a ' General Purpose Mortar Mix" for inner and outer walls of all low - rise buildings. The view was strongly supported by the National House Building Council and the Department of the Environment because such a mortar would reduce confusion and make quality control on site easier. A general purpose mortar, easy to mix and largely factory controlled would solve some of the problems of site practice and quality control.

(BRE, News of Construction Research, February 1992.)

U.S.A.

Laminated Process for Ferrocement

Laminating process for ferrocement consists of embedding the mesh in preplaced layers of mortar. This permits high concentration of reinforcing, eliminates voids, and all of the labor needed to tie the mesh in former methods. As a result it is now possible to place a 20 mm thick layer of ferrocement containing 20 % or more of steel mesh by weight on low- cost floating waterboard panels for less than $ 20 a square meter.

Laminated ferrocement can also be vertically or horizontally slip formed afloat in circular or rectangular cross section to produce OTEC pipes and concrete oil rigs or floating bridges and immersed tunnels for less than one million dollars a kilometer for each traffic lane. The concept of building directly on the water is difficult for traditional engineers and venture capitalists to accept, however, so it may be some time before the technology is accepted for this application even though it has been in use for over twenty years for smaller structures.

(Information from Martin E. Iorns, 1512 Lakeward Drive, West Sacramento, CA 95691. USA))

New 37 m, 4 section placing boom and full hydraulic pump sequencing

At the Atlanta world of Concrete, MORGEN manufacturing introduced a 37 m, 4 section placing boom with 36.58 m vertical reach, 33 m horizontal reach and continuous rotation. The first stage has a 90 degree articulation range, the second and third stages 180 degrees, and the fourth stage has a 245 degree articulation range. This is the largest placing boom made in the United States.

All pump and boom functions may be remotely controlled from the pour site. The Morgen proportional controls allow the operator to operate three boom functions simultaneously. Each function speed of operation is infinitely adjustable.

The new 37 m boom is mounted on a three axle truck. The boom is available with either the 115 or 140 SV Morgen swing valve pump. The new full hydraulic sequencing of these pumps utilizes state - of - the - art poppet valves for extremely fast cycle times.

MORGEN placing boom.

For more information contact MORGEN Manufacturing Company, P.O. Box 160, Yankton, SD 57078-0160. Telephone 605-665-9654. Telex 910-6683601. Fax 605-665-7017.

IFIC REFERENCE CENTERS

Ferrocement basic reference collection is available in the following IFIC Reference Centers. Each Center has a resource person who will entertain queries on ferrocement.

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BOOK REVIEW

ANALYSIS OF CONCRETE STRUCTURES BY FRACTURE MECHANICS

Edited by L. Elfgren and S. P. Shah

Proceedings of the International RILEM Workshop , Abisko, Sweden, June 28 - 30, 1989.
Published by Chapman and Hall, 2-6 Boundary Row, London SE1 8HN, England.

This book contain 21 papers, presented at the International RILEM Workshop on Analysis of Concrete Structures by Fracture Mechanics, organized by RILEM Technical Committee, held in Abisko, Sweden on June 28 - 30, 1989. The papers have been divided into five sessions, covering separately such topics as behavior of concrete, structural modelling, bending, shear, bond, punching and anchorage. The workshop was dedicated to Professor Arne Hillerberg in recognition of his many outstanding contributions to this field. In addition to the presentations and discussions during the workshop, as summarized by the authors, the volume also contains some papers contributed by colleagues and friends of Arne Hillerberg.

305 + XIV *160 x 240 mm; Hard bound* *1991*
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ABSTRACTS

FP 186 BEHAVIOR OF WELDMESH FERROCEMENT COMPOSITE UNDER FLEXURAL CYCLIC LOADS

KEY WORDS : design, fatigue, ferrocement, flexure, weldmesh

ABSTRACT : A new qualitative mechanistic model which is thought to reflect the behavior of ferrocement in flexural fatigue more realistically is presented. In light of this model and test results, the rectangular stress distribution assumption is found to be better for estimating steel stress when designing a weldmesh ferrocement against fatigue. This design is facilitated by using the fatigue behavior of only the reinforcement tested in the air. It is also shown that the dominating design criterion is not the crack width but the steel stress for all common structures.

REFERENCES : Xiong, G. J. and Singh, G. 1992. Behavior of weldmesh ferrocement composite under flexural cyclic loads. *Journal of Ferrocement* 22(3) : 237 - 248.

FP 187 FLEXURAL IMPACT DAMAGE OF FERROCEMENT

KEY WORDS : ferrocement, flexural, impact, damage, strain, deflection

ABSTRACT: Lateral flexural impact tests of ferrocement were performed under three point loading. To understand the properties of impact, test results are discussed on the effect of striking velocity on impact load, strain, and deflection. Moreover, we studied the relationships between face strain and deflection, and the absorbed energies obtained from load- deflection curves.

The strain at first crack in impact tests was approximately equal to that in static tests. Localized damage occurred under the load right after impact. A linear relationship was observed between compressive strain and deflection after localized damage. The energy expended in impact damage could be assigned to localized damage, crack opening, compressive failure of mortar, and bending of reinforcement. The energy of localized damage was in proportion to the drop height of a striker and was 25 % of the input energy. For a specimen in which only cracks occurred, 85 % of the input energy were spent for a cracked damage, and 15 % remained as the recoverable energy of deflection.

REFERENCE : Kobayashi, Y.; Tanaka, Y.; and Ono, M. 1992. Flexural impact damage of ferrocement. *Journal of Ferrocement* 22(3) : 249 - 264.

FP 188 FERROCEMENT DURABILITY : SOME RECOMMENDATIONS FOR DESIGN AND PRODUCTION

KEY WORDS : ferrocement, durability, design, production, recommendation

ABSTRACT : This paper presents some recommendations for design and production of ferrocement in order to eliminate the source of many pathological problems. These recommendations were derived from an exhaustive program of technical inspections of existing ferrocement structures in Brazil, that had been submitted to different climatic and environmental conditions.

Structures upto 30 years old, were examined and the main durability factors, both the positive and negative were identified. Design, production, maintenance and repair techniques were identified, classified and the most appropriate for ferrocement applications in civil construction were selected.

REFERENCES : Liborio, J. B. L. and Hanai, J. B. 1992. Ferrocement durability : Some recommendations for design and production. *Journal of Ferrocement* 22(3) : 265 - 271.

FP 189 FERROCEMENT AND REPLICA SHIPS

KEY WORDS : ferrocement, replica, ship, wood, hull, material

ABSTRACT : Future maintenance and repairs for wooden replica ships can sometimes cost as much, if not more, than the original construction budget. If the hull portion of the building process was constructed using ferrocement as the building medium, future maintenence and repair expenditure would be lessened to a great degree.

Ferrocement as a hull material can reduce this high maintenenc overhead to a minimum and short funding can be used to keep exposed woodwork and trim in proper condition.

REFERENCE : Mahan, L. M. 1992. Ferrocement and replica ships. *Journal of Ferrocement* 22(3) : 273 - 281.

FP 190 USE OF FERROCEMENT PANELS IN LARGE SPAN ROOFING SYSTEM

KEY WORDS : ferrocement, panel, large span, roof, building, low-cost

ABSTRACT : Prefabricated ferrocement panels offers a variety of possibilities to be used in many locations where economy, ease of construction and aesthetics are of prime importance. The objective of this paper is to discuss the design, fabrication, erection and construction technique for shell-type ferrocement units to cover a large span gymnasium, to form a composite roof.

Considerable saving in material cost about 20 % and a substantial reduction of construction time can be realized by employing ferrocement.

Based on the procedure described here, one industrial roof has already been built in Jamshoro Pakistan.

REFERENCES : Ahmed, S. F. and Dawood, H. 1992. Use of ferrocement panels in large span roofing system. *Journal of Ferocement* 22 (3): 283 - 289.

INTERNATIONAL MEETINGS

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March 10 1993 : **International Symposium on Cement and Concrete in the Global Environment, Chicago, Illinois, USA.** Contact : Mr. Steven H. Kosmatka, Portland Cement Association, 5420 Old Orchard Road, Skokie, Illinois 60077-1083, USA. Tel : 708-966-6200. Fax : 708-966-8389.

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B.K. Paul and R.P. Pama

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